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Chapter 1

Fundamentals of materials

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The long and still visible history of civil engineering shows how materials influence construction practice and design. Modern materials are diverse in composition and in internal microstructure. Broadly, we recognise three major families: metals, ceramics and polymers. Scientifically, material properties are controlled by the atomic, molecular and microstructural organisation, including pore structure. There is increasing emphasis on sustainability in the selection and performance of materials, not least because of the prodigious quantities consumed in civil engineering and building throughout the world.

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Introduction

Materials are at the heart of all branches of engineering. It can be argued – and with not much exaggeration – that engineering *is* the creative and rational use of materials for practical purposes. *Homo faber*, man the maker, works with what the material world permits and makes possible. Engineers are better engineers if they have a good understanding of the properties of the materials which they use. In the long past this came from hands-on familiarity rooted in tradition and craft practice. Now the range and variety of materials is so great that this is hardly feasible. The challenge to the engineer is also made greater by the pace at which new materials are being developed. But, at the same time, scientific knowledge of materials – of their composition, behaviour and properties – is advancing rapidly. Science now provides a framework for understanding materials, both new and old. Within this framework we can include all types of materials, thus to some extent unifying the treatment of the metals, ceramics, plastics and composites. This is the discipline of *materials science*.

The purpose of the first two chapters is to describe these generic aspects of materials, emphasising well-defined engineering properties and sketching our scientific understanding of how some of these properties arise. In this chapter we discuss materials from a broad social and historical perspective; and mention two important contingent issues: sustainability, and health and safety. We then describe the composition and internal architecture of materials as a basis for understanding the engineering properties which are the subject of Chapter 2.

Evolution and innovation in materials for civil engineering

No branch of engineering can provide such a long historical perspective on the significance of materials as civil construction. The survival of buildings, bridges and roads from ancient to modern times shows clearly the methods of construction in every era and region, and demonstrates repeatedly the close relation between available materials and structural and architectural design.

It is often noted that over several millenia until, say, the eighteenth century, substantial buildings (pyramids, temples, cathedrals, palaces, large houses and villas) and also bridges were necessarily constructed in some form of masonry, using ‘as-found’ materials: first quarried stone, later supplemented by fired clay brick. Thus the mechanical basis of most structures lay in compressive loading, which eventually found its sublime expression in the arch, the vault and the dome. Nonetheless, wood, another as-found material, was used widely in construction. Being less durable, such structures survive less frequently. Timber beams resistant to bending carried tensile loads from earliest times.

Masonry structures may be built without jointing mortars but only at greater cost, requiring shaped stones; and jointing materials such as gypsum, resins, bitumens, and, later, limes have been of value since Egyptian times. The role of lime as a binder and the eventual emergence of recognisably modern Portland cements deserve more detailed discussion (**Box 1**).

The metallic tradition

The metallic tradition (a phrase of J. E. Gordon) has been dominant throughout engineering for the last two centuries. It is from physical metallurgy that have emerged the combustion engine, the jet engine, the oil tanker, the turbine, the suspension bridge, the skyscraper (and indeed the elevator), the petrochemical plant and the machines of war. We can add to this list countless innovations in electrical power engineering and in electrical machines, including the engineering of building services. Metals, of

Box 1 Cementitious materials

Binders in the earliest eras, notably in ancient Mesopotamia and Egypt, were based on gypsum, clays and bitumens. These required at most modest heating to make them workable, but they conspicuously lacked durability and strength. Greek and Roman construction made use of lime produced by burning limestone (at around 700°C). Lime mortars and lime concretes harden slowly by reaction with carbon dioxide from the air. However, in Roman building there was a skilful understanding of how excellent binders could be made by combining lime with various siliceous materials (both natural, such as volcanic ashes and tuffs; and man-made, such as powdered brick and tile). These rather reactive materials combine chemically with the wet lime to form cementive minerals in what is now called a *pozzolanic* hardening process.

Recognisably modern *Portland* cements began to appear in the middle of the nineteenth century in Britain, France and Germany. This was the result of new methods of firing lime and clay to higher temperatures, sufficiently high not only to convert the limestone to lime but also to drive complex chemical reactions between the lime and the clay. The calcium silicates formed were *hydraulic*, reacting with water to form cohesive solid materials: liquid stone. These first cements were slower to develop strength and perhaps five times weaker than modern Portland cements. Nonetheless, their emergence marks the beginning of the modern cement industry, and of modern concrete construction. Today's industry produces some 2.6 billion tonnes of cement each year (van Oss, 2007), by far the largest quantity of any manufactured material. The technology is mature, but still capable of substantial if incremental improvement; for example, in achieving high compressive strength in concrete by using low water/binder ratio; or improving ductility by the use of short polymer or metallic fibres.

Figure 1 Terracotta pipework from Roman Ephesus. Pipes are 20 cm internal diameter and with socketed joints (reprinted from C. R. Ortloff and D. P. Crouch, Journal of Archeological Science ©2001 Elsevier)

course, have a much longer history than that, but their dominance has depended on cheap energy and an industrial scale of production. Above all, it was the emergence of ironmaking and steelmaking in the eighteenth and nineteenth centuries that stimulated and sustained the metallic tradition in heavy engineering. Before that, metals (gold, silver, bronze, steel, lead . . .) played mostly supporting roles in building and construction as decorative elements or as critical but minor functional components (nails, hinges, bars), or in making handtools for the working of wood or stone. Innovative architectural uses of unusual metals such as titanium depend initially on the existence of larger markets in aerospace and biomedical engineering.

Pipes

Dependable pipes and pipework systems are a prerequisite for hydraulic engineering. For complex small-diameter pipework, capable of fusion jointing, lead has been used since Roman times. So have moulded fired clay ceramics, **Figure 1**. Vitruvius *Ten books on architecture* comments on both and notes the difficulty of jointing in ceramic pipe systems. For large-diameter pipes, fired clay (vitrified or salt-glazed to reduce permeability) was the workhorse material throughout the nineteenth century well into the twentieth, with some continuing use at the present time. Wrought iron pipes came into use, and later steel. The radical innovation has been in the rapid and general substitution of polymer pipes over the last 20–30 years, primarily high-density polyethylene (HDPE). The advantages of good durability, low permeability and at least some flexibility are clear. Flexibility largely eliminates the problems of fracture caused by ground settlement. The long runs (even continuous reeled pipe for smaller diameters) makes trenchless installation possible; manufacturing technology allows for effective jointing details to be incorporated, even in some cases polymer welding to be applied. Other polymer-based materials may also be used, for example using glass-fibre reinforced polyester. Polypropylene pipes (and couplings) complement the use of copper in small-diameter plumbing systems.

Glass

The float process for manufacturing sheet glass (Pilkington, 1969), (**Figure 2**) made flat glass for windows widely available throughout the world. The process, in which a thin layer of molten glass floats on the surface of a tank of molten tin, produces a continuous ribbon of glass sheet of high surface quality and free of distortions. Before the float process, glass of such quality could only be produced by the laborious and costly grinding and polishing of plate glass. The availability of window glass strongly influences façade design and environmental performance in buildings of all kinds. Here the innovation was in manufacturing method rather than material composition, but this was critical in making glass available in a form, of a quality and at a cost which stimulated new engineering.

In turn, the widespread use of sheet glass has demanded improvements in glass technology to meet deficiencies of

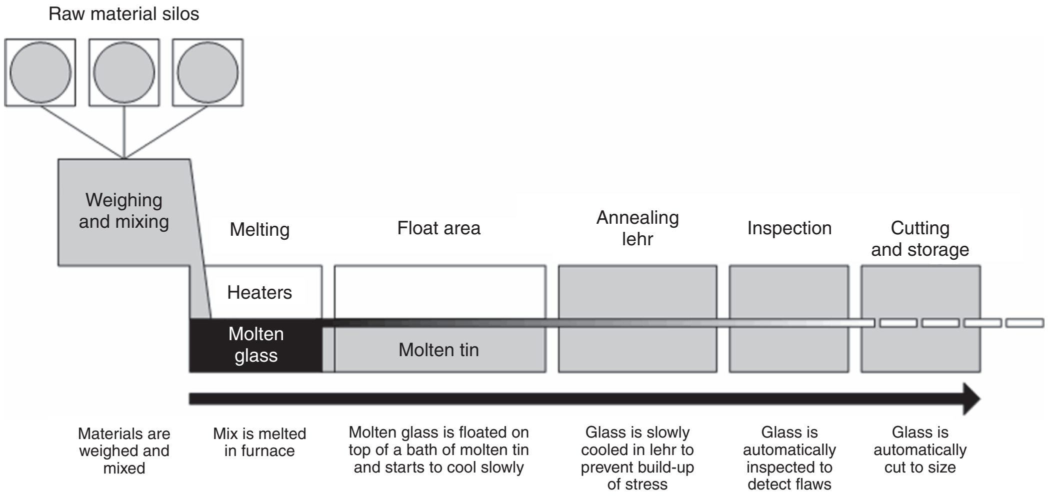

Figure 2 The float glass manufacturing process (courtesy of Tangram Technology Ltd)

the standard material. Notably, laminated safety glass sheet has been developed, most commonly by sandwiching one or two layers of a transparent polymer film, usually poly(vinyl butyral), between two or three glass sheets. Surface treatments of sheet glass, in which extremely thin coatings of transparent materials (such as titanium dioxide) allow the glass to act as a selective filter for various components of sunlight can provide solar control in the building envelope.

In parallel with advances in flat glass have been numerous innovations in materials which depend on the remarkable properties of glass fibres. These range from short glass fibres for incorporation into cement and concrete (for which zirconia glasses were developed to avoid alkali attack); glass fibre in the form of rovings and fabrics for the manufacture of polymer-based composites; and, of course, highly engineered optical fibres for communications.

Innovation in materials

Given the vast quantities of materials consumed in civil engineering, it seems unlikely that radically new primary or commodity materials will emerge in the near future. There are profound differences between the economics, needs and practices of civil engineering, and of sectors such as information storage, communications and biomedicine. While new high value-added materials produced in small volumes using advanced manufacturing processes may offer unique benefits, we are unlikely to see radical displacement of the main materials: concrete, steel, brick, stone, glass and polymers.

Nevertheless, civil engineering benefits in many ways from developments in materials technology in roles which are less apparent than as primary structural materials. Concrete technology has exploited numerous chemical additives which modify, for example, workability or allow air-entrainment to improve frost resistance. Civil engineering and construction now are among the largest end-uses of polymers (plastics and rubbers), mostly in non-structural applications such as membranes, geotextiles, fibres, coatings, adhesives and sealants. Sometimes, however, such materials come to the fore and find striking architectural expression as, for example, in tension roof structures using polymer/glass textiles.

There is also no doubt that the rate of innovation in materials science and engineering as a whole is rapid. These advances are on many fronts. Among the most remarkable in the last two decades is the unexpected discovery of new forms of pure carbon materials: the fullerenes, carbon nanotubes and most recently graphene. These materials, made from one of the commonest of the chemical elements, have extraordinary electrical, chemical and mechanical properties. Carbon nanotubes, **Figure 3**, are the stiffest materials yet known (stiffer than diamond), and at least on the small scale the strongest. They demonstrate the possibility (if not yet the sure prospect) of ultra-stiff and ultra-strong bulk materials for structural use based on a sustainable and abundant raw material of intrinsically low cost. Of course, manufacturing processes need to be developed but it is clear that proof of concept has now been attained.

Other innovations in materials from many fields will also diffuse into civil engineering over time. Examples can be found in energy technology (for instance, in photoactive materials for solar energy conversion); sensor materials for structural health monitoring; and in superhydrophilic photocatalytic coatings for self-cleaning surfaces such as flat glass or superhydrophobic surfaces to shed water (**Figure 4**).

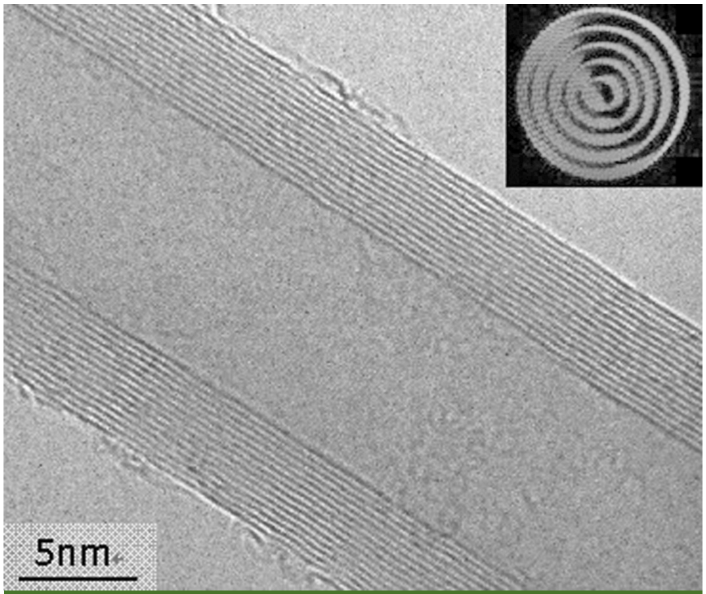

Figure 3 Multi-wall carbon nanotube (courtesy of Professor M. Endo, Shinshu University, Japan)

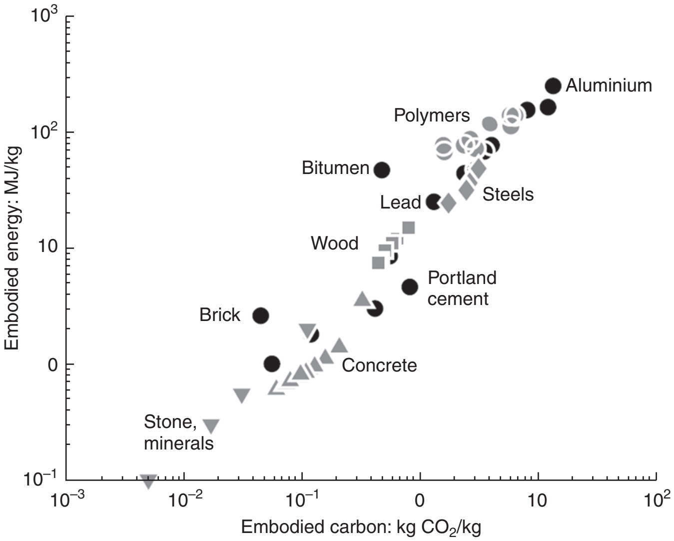

Figure 5 Embodied energy of materials (based on data from Hammond and Jones, 2008)

Sustainability of materials

All materials have their origin in the natural resources of the planet; all consume energy in primary manufacture, in conversion to components and products, and subsequently in transportation through the supply chain to the user. During service life, the processes of deterioration and cumulative damage entail further expenditures in maintenance and conservation; and ultimately in replacement and disposal. From the perspective of *sustainability*, we consider how to analyse the entire life-cycle costs of materials to provide a basis for rational choices in materials selection and usage.

One important aspect of this analysis concerns the energy costs of material manufacture (sometimes called the *embodied energy*). Thus there are large differences in the energy required to transform raw materials (metal ores, other minerals, oil and gas feedstocks) to primary materials such as iron and steel, aluminium, cement, plastics and rubbers. These primary energy costs are of course reflected in the prices of the materials, but it is useful to identify the embodied energy unambiguously. In **Figure 5** we show some representative values for major materials groups. The strikingly high embodied energy of aluminium metal is a direct consequence of the thermodynamic stability of chemically combined aluminium. Huge amounts of energy are required to release aluminium metal from its ores, and aluminium manufacture is often located near to hydroelectric power plants. The high embodied energy provides a strong incentive for recycling of aluminium as fabrication energies are much smaller than the primary energy of metal winning. We note in passing that thermodynamic instability of the metal with respect to the ore also provides the inexorable driving force for aluminium corrosion. We show the energy cycle for aluminium in **Figure 6**.

We may equally make estimates of the embodied carbon (carbon dioxide) of materials, embodied water or other environmental or resource costs. In civil engineering, Portland cement provides an important illustration of embodied energy. There is a heavy direct energy requirement to heat the large amounts of raw materials to kiln temperatures around 1450°C. Estimated embodied energy is about 5.1 MJ/kg cement. Since cement is manufactured from clays and limestone raw materials, and since carbon dioxide is liberated in large quantities in the firing of the limestone component, carbon dioxide is a major by-product of cement manufacture. Additional embodied carbon comes indirectly from the use of fossil fuels in kiln heating. Estimated embodied carbon is about 0.80 kg CO_2/kg cement produced. Because of the prodigious quantities of cement manufactured throughout the world (about 2.55 billion tonnes in 2006), about 3% of total anthropogenic

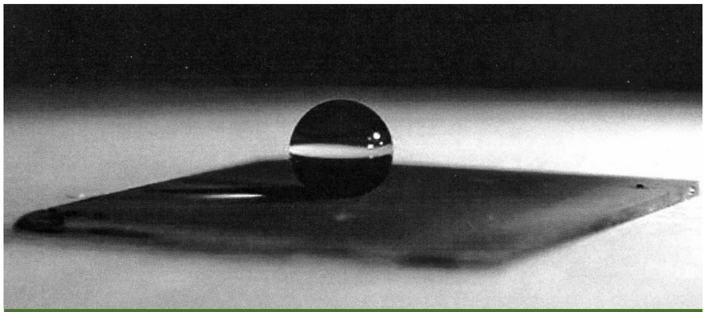

Figure 4 A water droplet (4.7 mm diameter) on a superhydrophobic surface formed from PTFE-coated carbon nanotubes (reprinted from Z. Yoshimitsu *et al*., Langauir ©2002 American Chemical Society)

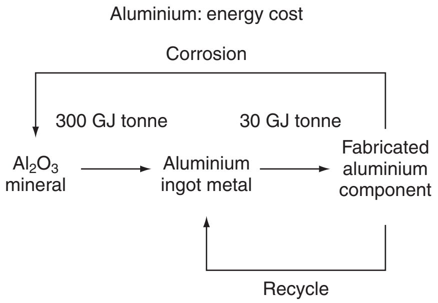

Figure 6 Aluminium: embodied energy associated with metalwinning and fabrication

CO_2 is traceable to cement manufacture. It is true that, in partial compensation, concrete also reabsorbs carbon dioxide from the air but only slowly and mainly superficially.

Health and safety in materials engineering

Many materials present some hazard both to workers at the time of construction and thereafter to the public at large. Health and safety considerations therefore receive attention throughout the manufacturing and construction process, and arise also at all subsequent stages throughout service life. We can find several examples in the recent history of materials in the construction industry to illustrate the risks and problems that exist.

The first is the painful story of the industrial use of asbestos. Asbestos is an inorganic mineral fibre which was widely used for many decades in building construction and shipbuilding as a thermal insulant and filler. Asbestos occurs in several forms, all composed of brittle fibres which, during handling, easily break into minute fibrils of such a particle size that they are readily inhaled and deposited deep in the lungs (**Figure 7**). After many years of extensive use, it was established by occupational health studies that these fine asbestos particles are extremely carcinogenic. By the time that this was known, absestos was already widely present in the built environment. The importation, supply and use of various mineralogical forms of asbestos were progressively banned in the UK in the 1980s and 1990s, but the recovery and disposal of asbestos from existing buildings remains a significant health risk (*Control of Asbestos Regulations*, 2006). The hazard arising from asbestos illustrates the dangers associated with fine powders and dusts.

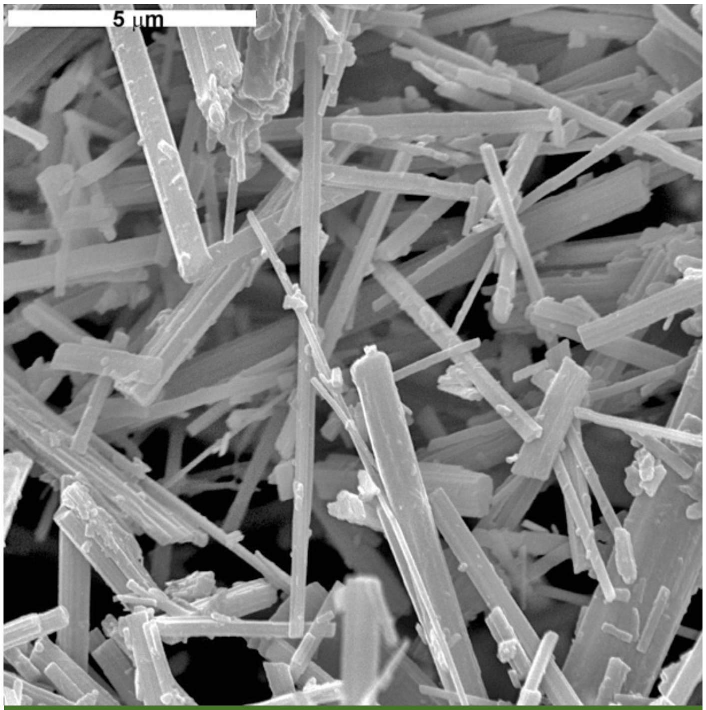

Figure 7 Amosite asbestos fibres at high magnification (courtesy of USGS)

A second example is the role of trace levels of chromium in cement in causing allergic contact dermatitis. This is a serious disabling condition which affects construction workers such as bricklayers and tilers who handle wet cements. The disease has been traced to the presence of water-soluble Cr(VI) in minute amounts in Portland cements. In the UK, a COSSH directive (*Control of Substances Hazardous to Health Regulations*, 2004) limited the Cr(VI) content of manufactured cements to 2 parts per million from 2005. A similar directive has been issued by the European Union (EU, 2003). The reduction in Cr(VI) is generally achieved by adding a small quantity of a reducing agent such as ferrous sulfate to the manufactured cement, so that in the wet mix the chromium is reduced to the less hazardous oxidation state Cr(III). The ferrous sulfate is effective for only a limited time, thus setting a corresponding shelf life for cement, typically a few months from the date of packing or dispatch. This is one example of many to illustrate the importance of controlling the exposure of construction workers to chemical substances. Many others arise in the use of paints and coatings, adhesives, solvents and concrete admixtures.

Types of materials: composition and microstructure

Engineering as a whole makes use of a vast diversity of materials. Civil engineering and building construction worldwide consume large quantities of natural materials such as stones, aggregates and wood. But there are also important manufactured materials such as cements, fired clay bricks and tiles, glass, plasters; an expanding use of polymer materials particularly for pipework, in geomembranes, as well as for coatings, fibres and adhesives; and of course all the major types of ferrous and non-ferrous metals: the steels, copper, zinc, aluminium, lead and others.

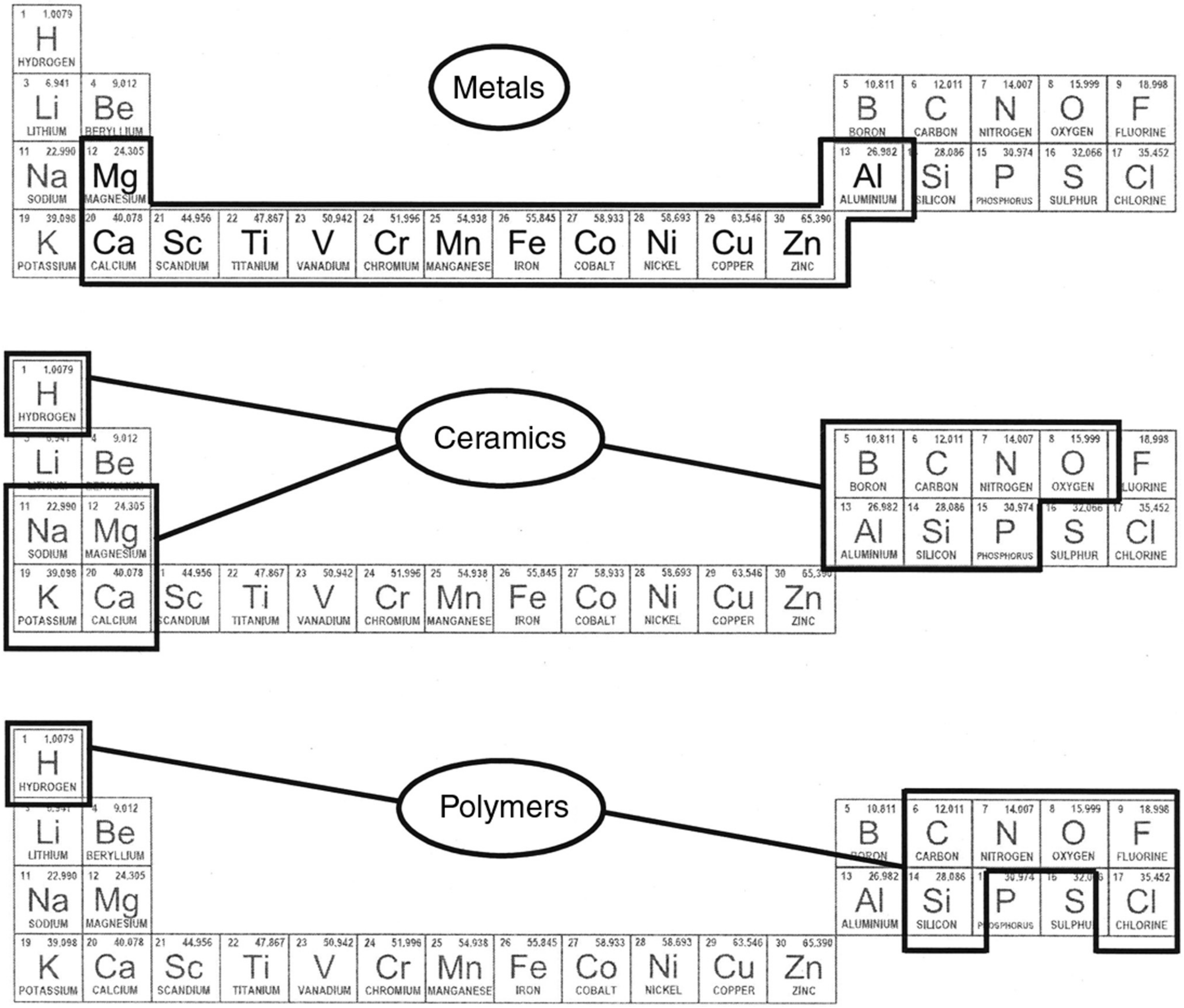

Figure 8 How the elements of the Periodic Table combine to form metals, ceramics and polymers

In materials science, it is conventional (and useful) to classify materials into three broad groups: *metals*, *ceramics* and *polymers*. These groups are distinguished fundamentally by their composition and their chemical constitution. The metals owe their useful engineering properties to the distinctive chemical bonding which exists in certain elements of the Periodic Table (**Figure 8**), such as iron, aluminium and copper. This bonding provides strong cohesion within bulk metallic materials, leading typically (but not invariably) to high strength, high density as well as high thermal and electrical conductivity. In contrast, ceramics (and glasses) owe their cohesion to ionic and covalent bonds. They are formed from combinations of non-metallic and metallic elements: for example, sodium, silicon and oxygen in simple glasses; or calcium, silicon and oxygen in Portland cement. The cohesion in ceramics is also strong, and this yields stiff, strong materials, generally less dense than metals but with high melting temperatures. The more complex internal bonding makes plastic adjustment to high stress more difficult than in metals, so that ceramics and glasses are characteristically brittle. Finally polymer materials are universally constructed from carbon, generally in combination with other light non-metallic elements such as hydrogen and oxygen, in distinctive chain and network structures. The interatomic cohesion combines strong, stiff bonds within the chains and weak interaction forces between them. This produces materials with low stiffness, low melting temperature and low density, but high plasticity.

Natural materials include both polymers such as wood and ceramics such as rocks, building stones and innumerable minerals. There is renewed interest in these materials from the standpoint of sustainability. It is striking, however, that the metallic elements are not found as simple metals in nature, and biology has evolved without developing or using metals as structural materials. Load-bearing materials in biology are invariably either polymers such as wood, or polymer-ceramic composites such as bone and shell.

The chemistry of materials

The fact that the main groups of materials map so clearly on to the Periodic Table of the elements shows us that ultimately and fundamentally the properties of materials depend on the chemical bonds between the atoms. It is these bonds that hold matter together. In the chemical bonds lies the origin of *cohesion* in solid materials, on which stiffness, strength and structural integrity depend. But this also reminds us that materials are chemical substances, able to participate in chemical reactions with other substances.

We take advantage of this reactivity in numerous fabrication processes. In civil engineering, the most striking example is the reactivity of cement towards water, a thermodynamically driven chemical hydration reaction on which the whole of cement and concrete technology turns. Other chemical reactions occur in the setting of adhesives, the hardening of polymer resins in the fabrication of composites, in the drying of paints, in plaster setting. However there are also chemical reactions between materials and their environment, which frequently cause degradation in materials and in their performance. A common case is the corrosion of metals in wet conditions caused by chemical reactions between the metal, water and oxygen. All engineering metals (gold is the only exception) are thermodynamically unstable with respect to conversion to their oxides and hydroxides; the only questions are how long does this reaction take? – and how ingenious can we be in impeding it? Understanding the chemistry of metal surfaces in contact with water and air allows us to understand why changing the electrical potential of a metal structure (say a buried pipe) is effective in preventing corrosion. The different groups of materials have different vulnerabilities. Polymer materials are generally unaffected by contact with water, but are degraded by the joint action of oxygen and sunlight. Being mainly composed of carbon and hydrogen, polymer materials are generally also combustible: they burn freely in air with low ignition temperatures. Ceramic materials are also vulnerable to chemical processes of degradation. Concrete, leached by water, loses some of its more soluble components and is weakened. It is attacked by acids and susceptible to damaging chemical alteration by dissolved sulfates, for example from groundwaters. Fired-clay products such as brick and tile are subject to long-term progressive rehydration by water which leads to slow expansion, with structural consequences in masonry. Building stones (**Figure 9**) are damaged by many chemical processes. Limestones are weakened by sulphur gases in the atmosphere, carried by driving rain, through chemical reactions in which calcite and dolomite minerals are converted to gypsum. The theme of chemical reactivity in construction materials is developed further in the chapter on durability.

Figure 9 Stone decay by chemical alteration (photograph by Andrea Hamilton, University of Edinburgh)

Composition of materials

The properties and behaviour of materials are fundamentally determined by what they are made of and by their internal architecture. We have defined the chemical elements which constitute the main groups of materials. In most materials, when we examine materials at the highest resolution, we find that the atoms of the various elements present usually take up regular geometric arrangements. Such ordered *crystalline* structures are favourable for maximising the energy of interaction between neighbouring atoms. That is to say, the chemical bonding between adjacent atoms is at its strongest. These bonds, collectively, provide the *cohesion* which is essential for engineering materials. **Figure 10** shows at nanoscale resolution (see **Box 2**) the surface structure of the mineral magnetite (which is found in black rusts on iron and is also the material used in magnetic recording media). The image was acquired with a *scanning tunelling microscope* which is able to reveal the positions of individual atoms. Here we see clearly the hexagonal crystalline packing of the oxygen atoms. (We also see that a few positions are unoccupied: such *point defects* or *vacancies* are almost always present. These and other kinds of defect can have a strong influence on the engineering properties of materials.)

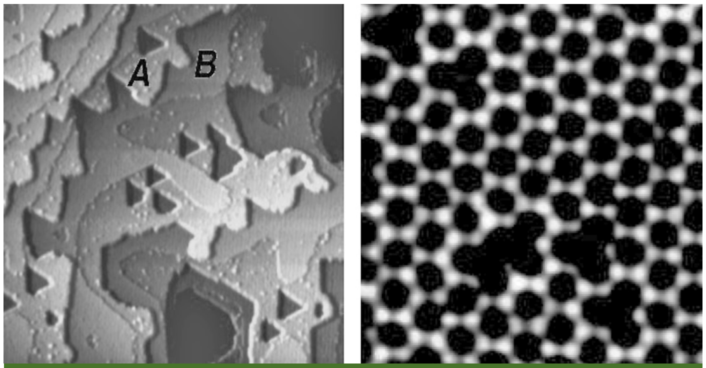

Figure 10 Scanning tunnelling microscope STM image of the surface of iron oxide. The left image shows two kinds of elementary steps in an image field of 150 × 150 nm. The right image shows atomic resolution of the hexagonal surface lattice with vacancies, image field 6 × 6 nm (courtesy of P. W. Murray, University of Liverpool)

In **Figure 11** we show the surface of a copper alloy, and again a well-ordered crystalline arrangement is immediately visible.

When we examine materials at a lower magnification, we usually find that the bulk material is made up from a myriad of crystalline regions randomly oriented in relation to each other (**Figures 13** and **14**). At the engineering scale, the materials are *polycrystalline*. The individual crystallites or grains are typically micron to mm in size. In metals and ceramics, the grains are commonly formed as the material cools from the molten state during manufacture. During cooling, the crystalline solid forms spontaneously at many places throughout the material, and these crystallites grow during solidification until they encounter other grains, eventually forming a continuous solid material. The final size of the grains is controlled by the ease of nucleation and the velocity of the subsequent crystal growth. These processes, in their turn, are strongly affected by the cooling rate and the nature of the material. In metals, in particular, the microstructure may be more or less modified during processing. The grain structure is capable of modification by heat treatment of the solid. The grain boundaries play an important role in determining many engineering properties. In minerals, crystallinity is also the general rule. **Figure 15** shows the surface of the mineral portlandite formed in the hydration of Portland cement. The geometrical regularity of the surface reveals the underlying crystallinity.

Box 2 The nanoscale: the smallness of atoms and molecules

In **Figure 12** is a 2p coin, made from hard-wearing bronze alloy, a mixture of copper and tin. One coin contains about one hundred billion trillion (10^{23}) atoms. If we were to magnify the coin until it covered most of western Europe and reached from Edinburgh to Rome, then each atom (about 0.2 nm [2×10^{-10} m] diameter) would itself be about the size of a 2p coin. A typical polymer chain molecule would be as big as a car, and a single particle of clay would cover a football stadium.

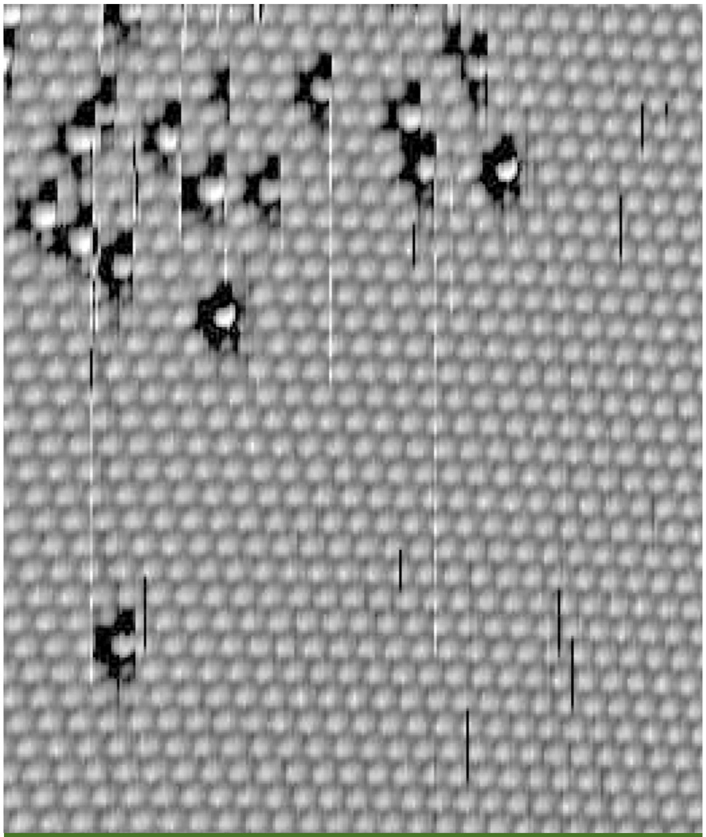

Figure 11 Scanning tunnelling microscope STM views of the surface of metallic copper (reprinted from C. Nagl *et al*., Surface Science ©1994 Elsevier)

While regular crystalline arrangements of atoms are almost invariably found in metals and minerals, frequently

Figure 12 A bronze coin enlarged to cover western Europe

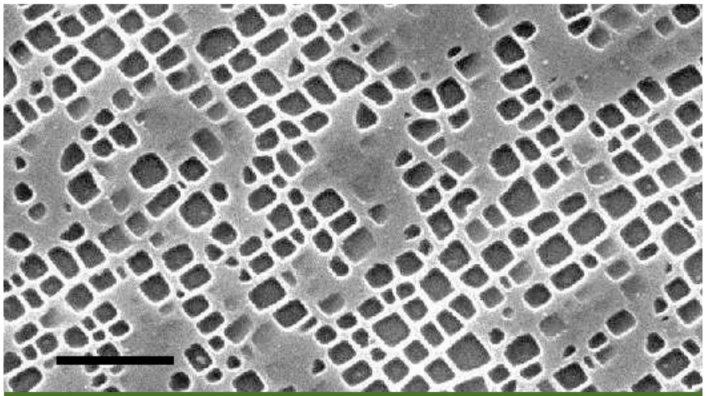

Figure 13 Nickel-based superalloy: Microstructure after casting and heat-treating showing intermetallic particles. Scale bar 2 μm. (SEM image by Jane Blackford, University of Edinburgh)

also in manufactured ceramics and sometimes in polymers, there are important exceptions. In some materials, it proves difficult for the constituent atoms and molecules to form ordered crystalline arrangements in the solid state. Instead, the distribution of atoms and molecules lacks the long-range order found in crystals. Such materials are described as *amorphous*. Inorganic glasses ('glass') are such amorphous materials. The main constituent of inorganic glasses is silica, which in the molten state forms strong three-dimensional networks of Si–O chemical bonds. This leads to a melt so viscous that the nucleation of crystalline domains is severely impeded. In polymers, it often takes a long time for long chain molecules to disentangle themselves sufficiently to form ordered arrangements. As a result, the solid state of many carbon-based polymers is amorphous and glassy. In other cases, the bulk polymer materials are only partially crystalline: such *semi-crystalline polymers* include both widely used commodity plastics such as polyethylene PE and polypropylene PE as well as advanced engineering plastics such as the aramid fibres. In semi-crystalline polymers, however, the crystalline domains may constitute the majority of the bulk material. Typically, the crystallites combine to form complex *spherulites* which can be seen easily in the light microscope, see **Figure 16**.

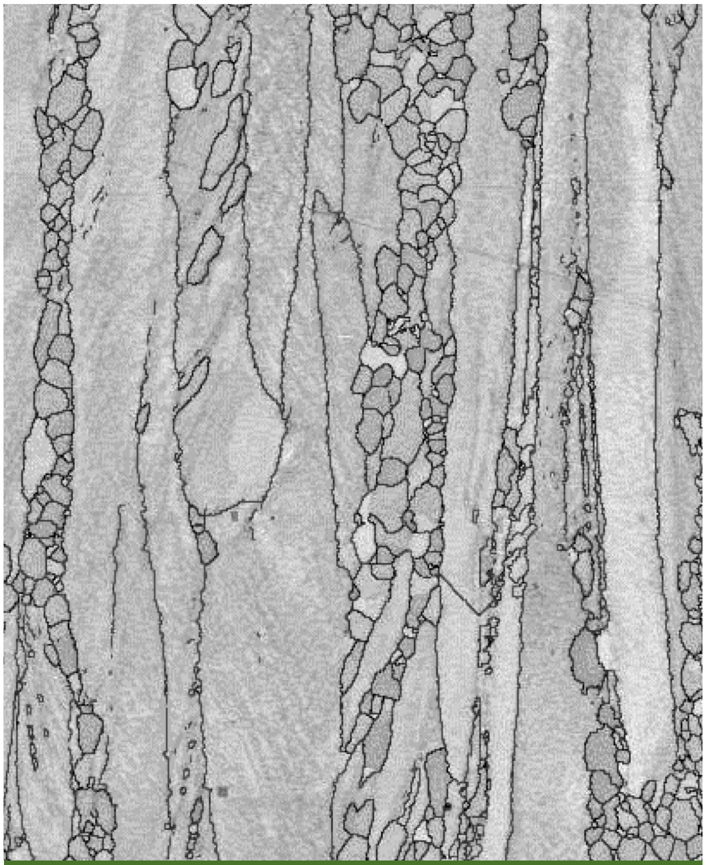

Figure 14 Iron aluminide: Polycrystalline grain structure. The electron back-scattered diffraction image shows the orientation of the grains after casting, deformation and heat treatment. (Image by Jane Blackford, University of Edinburgh)

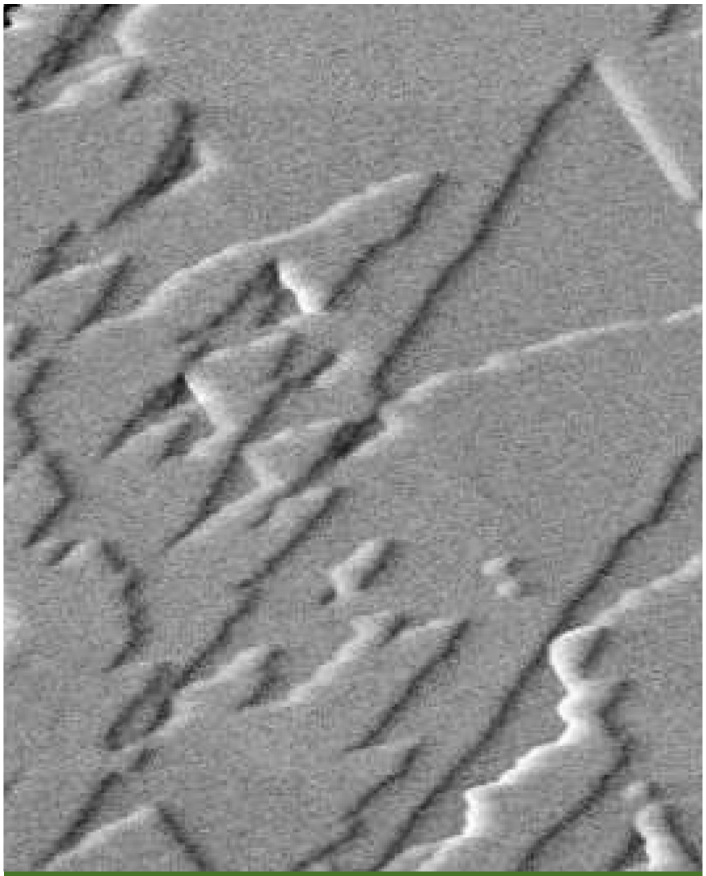

Figure 15 Portlandite 001 cleavage surface in air imaged by atomic force microscopy. Image field 5 × 5 μm. (C. Hall, D. Eger and Bosbach, Universities of Edinburgh and Muenster)

A third example of a material with *frustrated* crystallinity is a material of unique value to civil engineering: the main constituent of the hydration of Portland cement is also

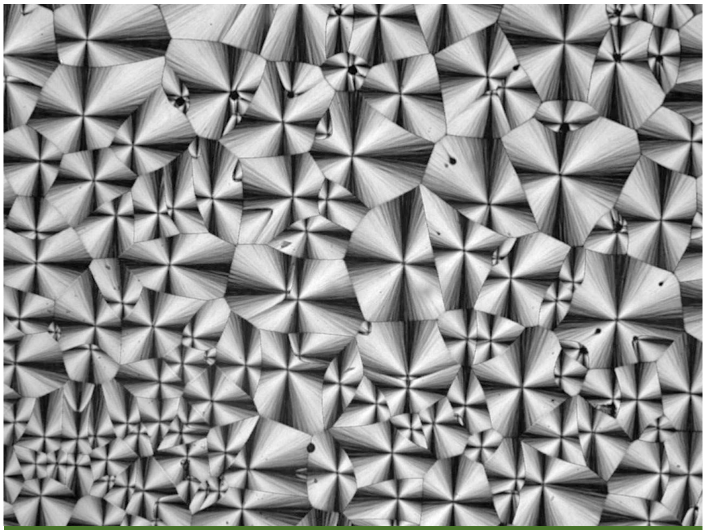

Figure 16 Spherulitic microstructure in poly(vinylidene fluoride) PVDF. Image field 725 × 630 μm. (Image by Seyhan Ince-Gundoz and Peggy Cebe, Tufts University)

largely amorphous or at least poorly crystalline. This material, denoted C–S–H, is the complex product of the chemical reaction of Portland cement with water. It is C–S–H which provides the main binding agent in normal concretes, responsible for cohesion, stiffness and strength. When cements are hydrated at higher temperatures (as they are in certain autoclave processes used in manufacture or in cementing in high-temperature oil wells), the products of hydration are usually crystalline. But in normal concrete mixes, the lower temperatures mean that the chemical reactions are sluggish and that the thermal energy is insufficient to jog the complex constituents into long-range crystalline arrangements. Even in this material, the nanostructure shows an emergent regularity, although this does not extend beyond a few nanometers (**Figure 17**).

The idea that most materials have a regular, ordered structure at the nanoscale has greatly helped us to understand the physical basis of engineering materials properties. Without worrying too much about the fine details, we can represent materials by a *lattice model*: a regular three-dimensional array of atoms and molecules, held together by chemical bonds. The stronger the bonds, the greater the lattice energy. We expect strongly bonded materials to have high melting temperatures, and probably to be stiff and strong. Each atom or molecule of the lattice seeks a site of minimum energy, and to break the cohesion of the solid requires work or energy. It is found rather generally that the interaction energy U of neighbouring atoms can be represented by a curve such as that plotted in **Figure 18**, the *Universal Binding Energy Relation* (UBER). The UBER curve shows that close to r_0 the potential energy $U(r)$ is well represented by a parabola, and the potential energy of the neighbouring atoms resembles that of a simple spring. We can think of the lattice dynamically as a vast three-dimensional network of springs. As a first approximation, we may consider that the springs vibrate symmetrically about their mean position, and this leads to the concept of the *harmonic* lattice. In fact, as can be seen, for large departures from the mean position, the UBER curve is markedly asymmetric. By examining the mechanical, thermal, dynamic and thermodynamic behaviour of an extended lattice, we can learn a great deal about the engineering properties of materials.

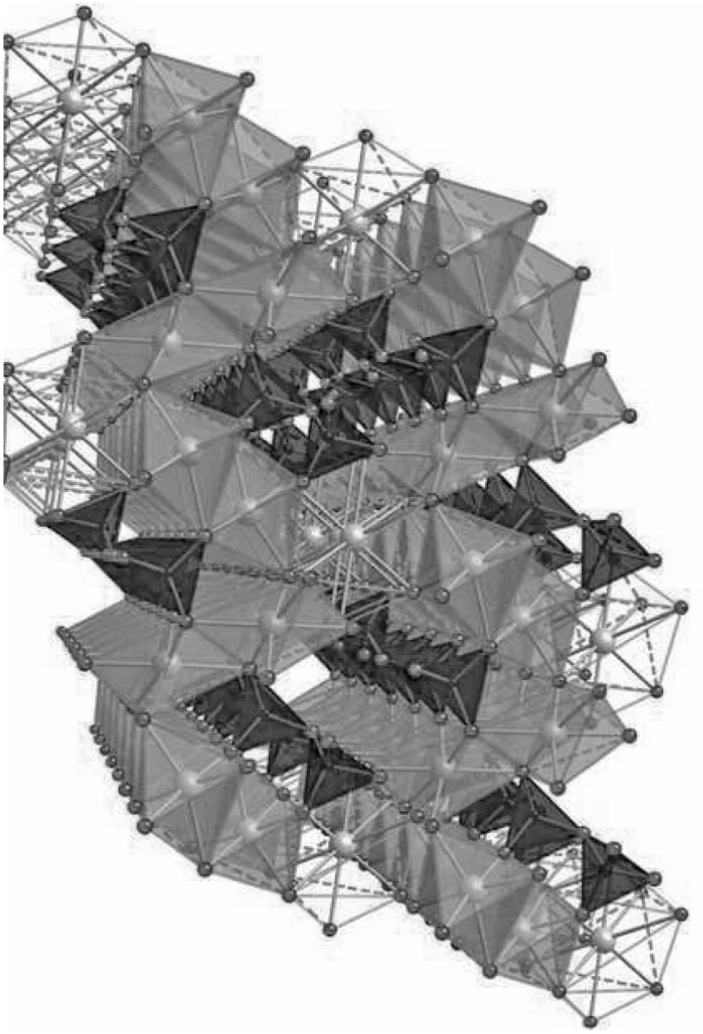

Figure 17 Atomic structure of a C–S–H nanoparticle in hydrated Portland cement (reprinted from I. G. Richardson, Cement and Concrete Research ©2008 Elsevier)

The crystalline organisation present at the nanostructural level in most materials is almost invariably imperfect. That is to say, our simple lattice model is an idealisation. Materials science recognises the existence of several kinds or imperfections of *defects*. There may be isolated unoccupied lattice sites (*vacancies*); or isolated lattice sites occupied by the atoms of the wrong element (*substitutional defects*). Defects

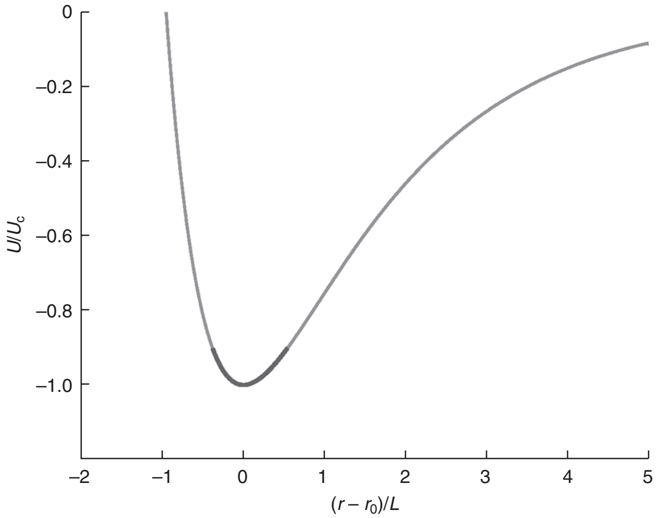

Figure 18 UBER: Universal Binding Energy Relation

such as these involving only single sites are called point defects. Although point defects have only local effects, they are important in introducing strain into the lattice and distorting it in their neighbourhood. Vacancies play a role in allowing long-range diffusion to occur by thermally activated site-hopping movements, which occur more readily as the temperature rises.

Of greater importance for material behaviour is the existence of larger-scale defects or *dislocations*, which extend in two or three dimensions. A common dislocation is the *edge* dislocation in which an entire sheet of atoms (a single lattice plane) terminates abruptly within the lattice. The neighbouring lattice planes then bend around the dislocation in order to maintain lattice continuity. This produces a strain in the lattice over the entire length of the edge. Under an applied stress, the edge is able to move sideways through the lattice, thus providing a powerful mechanism for irreversible plastic deformation and yield. A geometrically different dislocation is the *screw* dislocation in which the lattice is twisted about an axis. Screw dislocations play an active role in promoting crystallisation of solid materials in crystal growth processes from solution. Dislocations (and their movement) may be observed at high magnifications using the transmission electron microscope (**Figure 19**).

Although most materials are essentially crystalline in constitution, most conceal their crystallinity rather effectively. The distinctive facetted shape of a large single crystal is easily recognised but most materials develop as polycrystalline aggregates, and the distinctive crystallinity can be seen only by microscopic examination. **Figure 20** shows the microstructure of an almost pure copper material, showing that the solid material is composed of innumerable crystallites or grains, all identical in composition (single phase) but randomly oriented and separated by more or less straight grain boundaries. The grain diameter in this structure is about 125 microns. The crystallites develop from individual nucleation events as the material cools from the molten state. As the grains grow they eventually collide and adhere to form a cohesive polycrystalline metal.

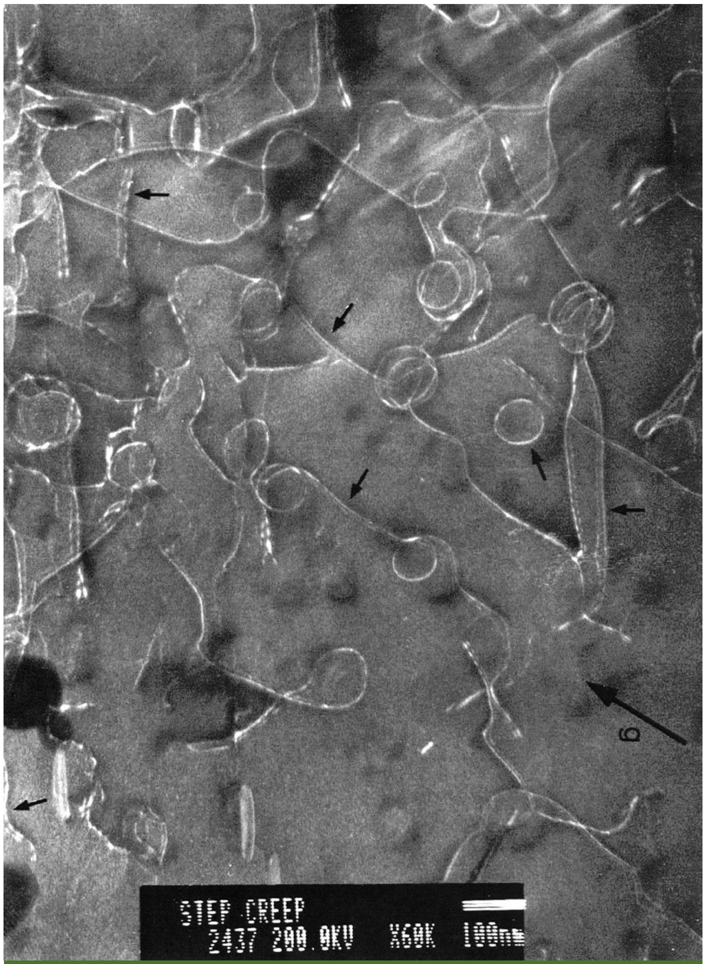

Figure 19 Dislocation in a nickel-based superalloy observed by transmission electron microscopy following creep testing. The dislocations loop around γ' intermetallic particles. Image field $1.25 \times 1.15\,\mu$m (courtesy of Y. H. Zhang, Cambridge University and TWI, UK)

In many materials, the grains have different compositions: polyphasic microstructures are commonly found in all types of materials: metal alloys, in ceramics (and rocks) and in polymer blends. For example, **Figure 21** shows the microstructure of cement clinker, the fired product of the cement kiln before grinding. Here we can see four different components, each distinguishable in this *back-scattered electron* image by the grey level. This image is highly informative since important properties of the cement (such as early-age rheology and strength) are controlled by the relative proportions of these components, which may be estimated by quantitative image analysis methods.

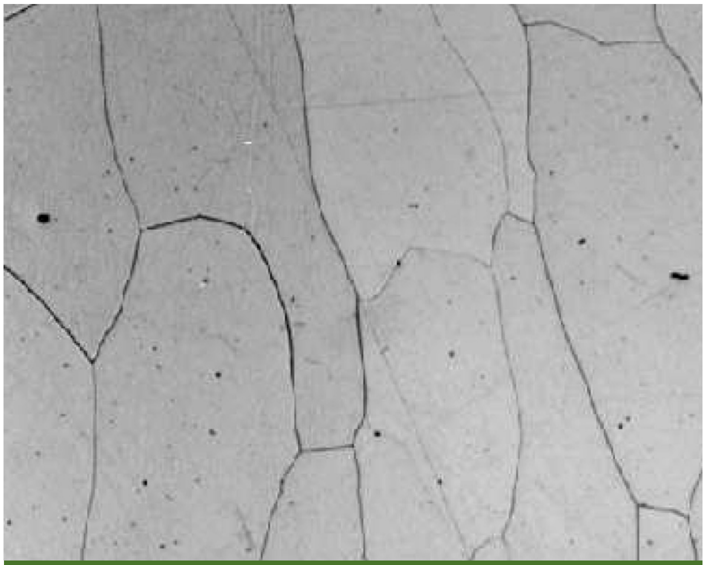

Figure 20 Microstructure of a Cu 50, Zn 50 (wt per cent) brass, chill cast. Image field 1.5 × 1.5 mm (courtesy of Professor Robert Chocrane, University of Leeds)

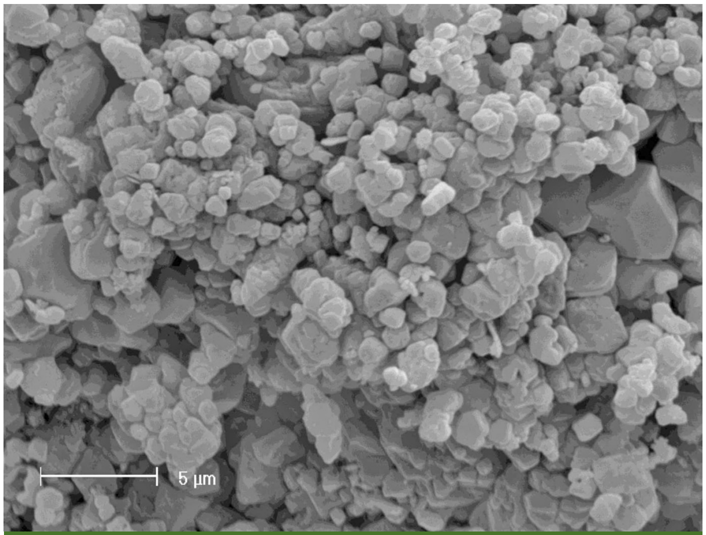

Figure 22 Porosity in Lépine limestone, showing the packing of fine calcite grains (SEM image by Jane Blackford, University of Edinburgh)

Porous and cellular materials

A common and distinctive microstructure found in many materials is generated by *porosity*. Porosity is the rule rather than the exception in the inorganic construction materials such as fired clay brick, concrete and stone, examples of which are shown in **Figure 22**. In these materials, porosity arises mainly from the granular nature of the materials, formed by compaction of mineral powders in manufacturing or geological processes. The *volume fraction porosity f* commonly lies in the range 0.1–0.3. The pores usually form connected networks, so that water

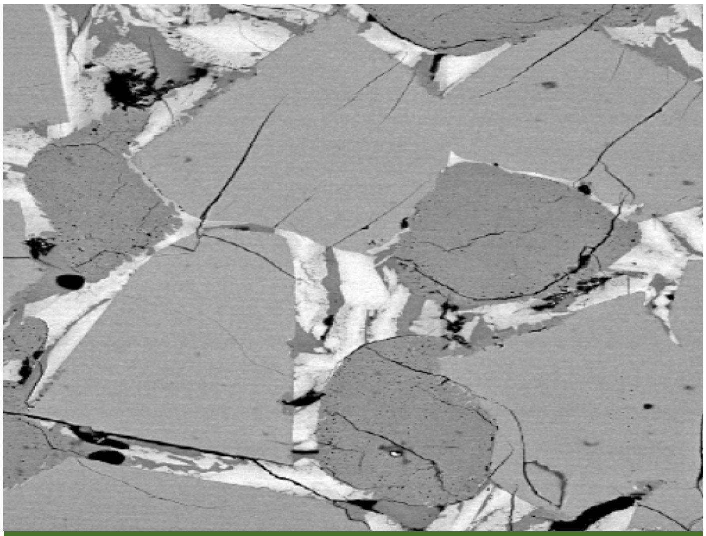

Figure 21 Microstructure of cement clinker, showing multi-mineral composition. Image field is 200 μm wide (back-scattered electron image by Paul Stutzman, NIST)

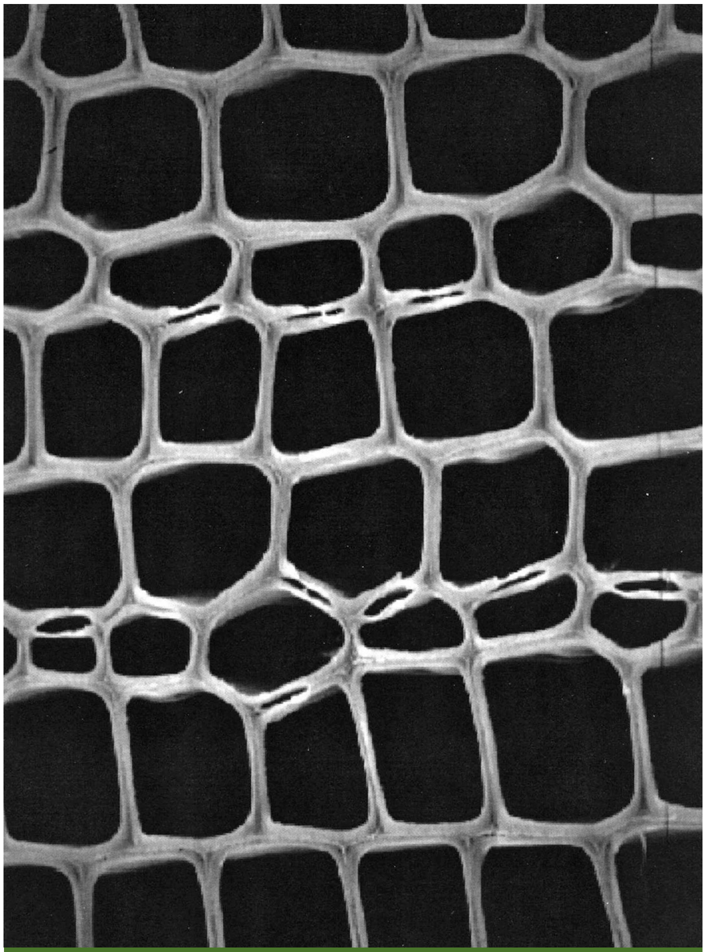

Figure 23 SEM micrograph of transverse cross-section of spruce wood. Image width 240 μm (reprinted from Trtik *et al*., ©2007 Elsevier)

(and other liquids) are absorbed by capillary forces. In such materials the transport of water plays a key role in long-term deterioration and damage. In sedimentary building stones such as limestones and sandstones the water transport characteristics are generally different parallel and perpendicular to the geological bedding plane: the materials are *anisotropic*.

Wood is another porous material, with an intricate cellular structure, also with anisotropic properties (**Figure 23**). Cellular forms of many manufactured materials, including polymers and metals, as well as cement-based materials, can be made to provide low-density and good thermal barrier properties. In manufactured materials, the porosity can be controlled and strongly influences the properties of the material (Gibson and Ashby, 1997).

The porosity defines simply the fraction of voids within a material but tells us nothing about the size of these voids. Some properties (for example the density) depend only on the porosity, but the transport properties such as the permeability are sensitive to the dimensions of the pore space and the pore network. To describe this, we develop the idea of a *pore size distribution*. Characteristic pore sizes differ enormously from material to material; and in nearly all porous materials there are pores of many sizes. Pores may be of molecular sizes, for example able to accommodate a few water molecules; or they may be large enough to see with the naked eye. Normal concrete contains pores of all such sizes.

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CONTENTS

Chapter 2

Engineering properties of materials

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Well defined engineering properties allow us to bring some order to the great diversity of materials behaviour. In turn, we can make sense of these properties by examining the composition and microscopic internal architecture of materials.

Introduction

An important element of the framework for describing the behaviour of materials is the notion of *engineering properties*. As a basis for the rational selection of materials for particular engineering purposes, we try to define key attributes, ideally in a way that can be quantified and measured by well defined procedures and which are to a large extent common to different groups of materials. In this way, engineers can obtain objective, comparative data to guide the choice of materials and with which to carry out design calculations. It is useful to group these engineering properties into clusters, bringing together, for example, the thermal properties which describe the behaviour of materials under changes of temperature or when heat flows through them. In this chapter we discuss thermal properties as such a cluster. We then describe mechanical behaviour and finally the chemical properties which play an important role in the durability and deterioration of materials in the working environment. First of all we consider the density as an engineering property.

We shall see that the key engineering properties also provide a point of entry to the scientific understanding of material behaviour: we can provide answers to a question such as 'Why is the density of steel greater than the density of aluminium?' Or 'Why do stiff materials show low thermal expansion?' We find many answers to such questions through understanding the physical and chemical processes which underlie the engineering properties. This is often by examining the microscopic structure of materials – the microstructure. The relation between material properties on the engineering scale and the microscale can provide a deeper insight into material behaviour. Equally important, it can also suggest how properties may be improved by microstructural modification.

Density

Density is a fundamental property of all engineering materials. It enters into numerous engineering formulae and is a factor in the selection of materials for minimum weight design. It is an unexpectedly interesting engineering property. It is simply defined for a material as the ratio mass/volume, and usually denoted by the symbol ρ. In solid materials, it is determined primarily by the masses of the constituent atoms. As the atomic number increases (as we go from top to bottom and from left to right in the Periodic Table of elements), the atom mass increases much faster than the atom volume, so that the effective atomic density rises rapidly. Materials such as polymers, which are composed of light elements such as carbon (C), oxygen (O) and hydrogen (H) (elements with a low atomic number, found at the top of the Periodic Table) have intrinsically low densities. Metallic materials contain predominantly elements of higher atomic number and consequently have much higher densities.

In addition, the density is influenced by the packing of the atoms in the solid lattice. For a series of materials with the same crystal structure, such as the face-centred cubic (FCC) metals, the solid density and the atomic number (or atomic mass) track each other quite closely. Crystalline and amorphous forms of the same substance have significantly different densities because the crystalline material has more efficient lattice packing. Thus vitreous silica which is amorphous has a density of about 2200 kg/m^3 while the density of α-quartz, the crystalline form of silica stable at normal temperatures, is 2650 kg/m^3. Under very high pressures, there is an even more close-packed crystalline form of silica, *stishovite*, with a density of 4290 kg/m^3. In the case of the semi-crystalline thermoplastics, the density varies with the degree of crystallinity. Thus, low-density polyethylene (LDPE) with about 40% crystallinity has typical density 910–930 kg/m^3, while high-density polyethylene HDPE with about 80% crystallinity has a higher density around 960–970 kg/m^3.

Range of accessible densities

Figure 1 shows the densities of a number of engineering materials. Note the logarithmic scale on the vertical axis. The values spread over about three orders of magnitude.

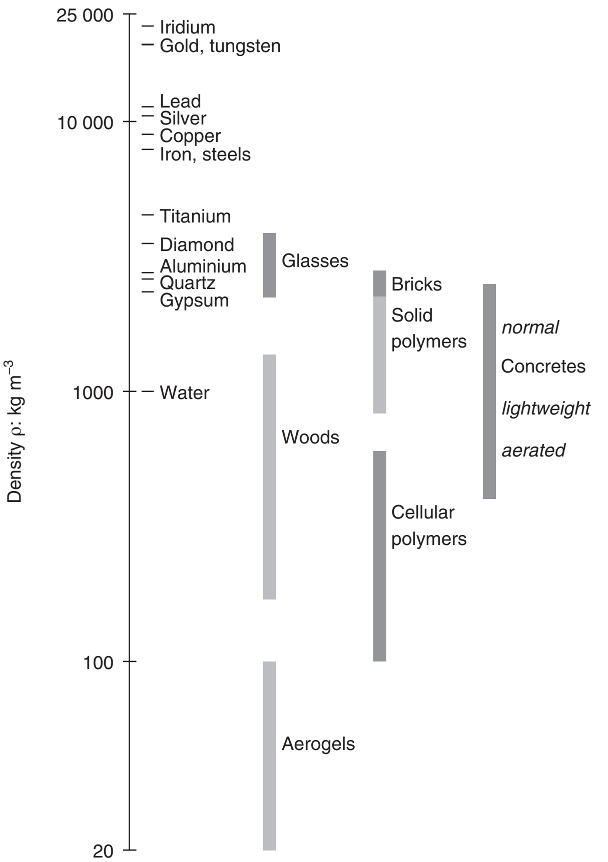

Figure 1 Density of some engineering materials

What is the highest density we can attain? The upper limit is set by the available heavy elements. The highest density in a solid engineering material is around 20 000 kg/m^3. The materials with these high densities are tungsten, gold, platinum and uranium. These metals are nearly three times as dense as iron and steel. On the other hand, the major ceramic materials, brick, glass and concrete, are about three times less dense than steel.

What is the lowest solid density we can achieve? In practice, in solid engineering materials we cannot go much below about 800–900 kg/m^3, as in some hydrocarbon polymers which contain only C and H. This is much the same as the density of water (around 1000 kg/m^3). Polymers which contain other elements such as F, O and Cl have significantly higher densities. For example, as we have seen, the density of polyethylene $(CH_2)_n$ is about 910–970 kg/m^3 but replacing half the H by O to produce polyoxymethylene (POM or *acetal*) raises the density to just 1100 kg/m^3. Polytetrafluoroethylene (PTFE) in which all the H is replaced by F has a density of 2280 kg/m^3.

Porous materials

The only general route to low-density materials is by incorporating porosity, and much lower densities are attainable in porous and cellular materials than in solid materials. Since the density of air is so low (1.2 kg/m^3), low bulk densities can be achieved in porous or cellular materials. Several processes are available to do this: for example, foaming of metals, polymers and even cement-based materials.

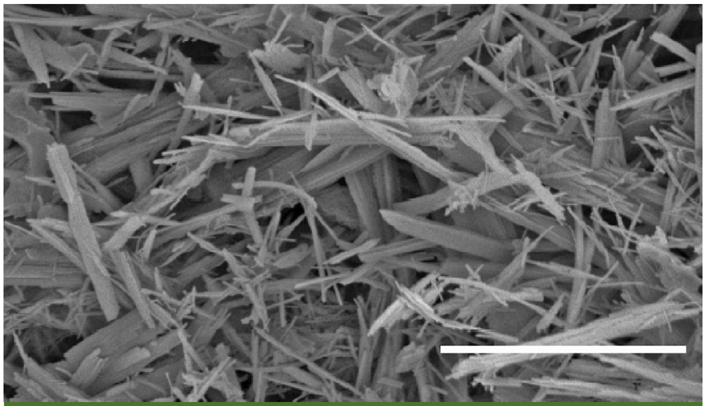

Figure 2 Xonotlite (scale bar 3 μm). (Electron microscope image courtesy of Dr Andrea Hamilton, University of Edinburgh)

Lightweight concretes can also be made by including porous low-density aggregates. In addition, porous materials can be formed from fibres or from particles by compaction or partial sintering. By these methods we can make materials whose bulk density is much lower than the solid density. The bulk density of a porous material $\rho_b = \rho_s(1 - f)$ where f is the volume fraction porosity and ρ_s is the solid density. The highly porous cement-based material shown in **Figure 2** is composed of the hydrated mineral xonotlite which has a solid density of 2700 kg/m^3 and which grows as long fibres in the hydrothermal manufacturing process. The randomly oriented fibrous particles form a remarkably open haystack microstructure. This material, used for thermal insulation in wall construction, has a volume fraction porosity of 0.89 and a bulk density of about 250 kg/m^3.

In fact most non-metallic inorganic construction materials are porous. Fired clay brick provides another important example. Commercial bricks of the standard format size vary considerably in weight, from 1.8–3.8 kg. Although the chemical composition varies slightly according to the raw materials used, the solid density ρ_s is generally close to 2600 kg/m^3. The variation in dry bulk density is largely determined by the porosity. **Figure 3** shows the variation of porosity and bulk dry density for some 60 British bricks of various types.

Woods are natural polymeric materials with complex cellular structures and, as for bricks, the density in the dry state is largely determined by the porosity. The solid density of wood substance is about 1540 kg/m^3, close to the densities of cellulose and lignin, the main solid components of wood. Because of wide variations in porosity, the dry bulk density

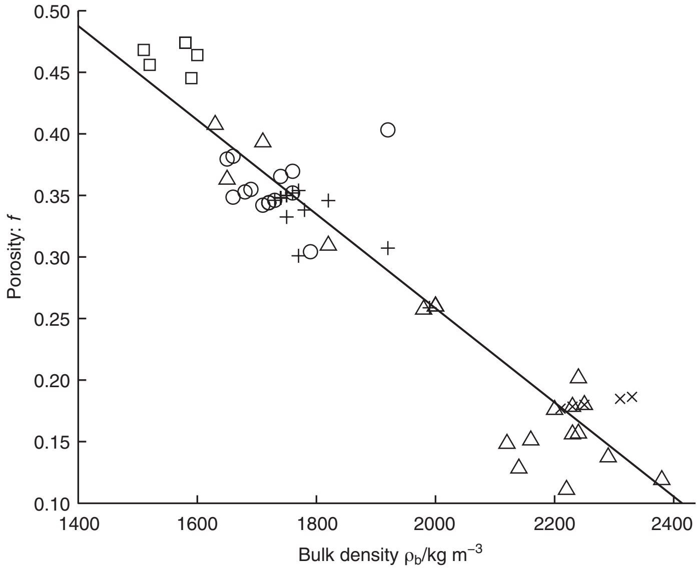

Figure 3 Porosity vs bulk density for fired clay brick (Hall *et al.*, 1992). Symbols denote bricks of different make or type. The straight line is the least squares fit to the whole dataset, $f = 1.022 - 3.821 \times 10^{-4} \rho_b/(\text{kg m}^{-3})$

ranges widely from species to species, as **Figure 1** shows. In the case of wood, the pores also contain water, so that the so-called *green* or unseasoned density of the wood is rather greater; in use, the water content depends on the environmental humidity, so that the bulk density varies somewhat.

The manufactured materials with the lowest bulk density are the silica aerogels: fragile materials composed of glassy silica membranes. These have bulk densities in the range 20–100 kg/m^3. We see, therefore, that the overall range of density available in engineering materials is about 20–20 000 kg/m^3.

Measurement of density

The density of homogeneous solid materials is generally straightforward to determine. It requires only measurements of mass (or weight) and of volume of a specimen of material. Of these, the volume is the less easy to measure accurately. For a regularly shaped specimen, the directly measured dimensions may suffice. For an irregularly shaped specimen the volume is usually (and best) obtained from the so-called Archimedes' weight (see, for example, prEN 12390-7: 2008). This is the weight W_w of the specimen suspended in water. The specimen volume $V = (W_a - W_w)/\rho_w$ where W_a is the weight in air and ρ_w is the density of water. For porous materials, both bulk and solid density may be required. As we have seen, these are related through the porosity. The measurement of porosity is fully described in Hall and Hoff (2002). For heterogeneous and composite materials such as concrete, it is important to measure the density on specimens which are sufficiently large to be representative of the mean composition.

Significance of density

The density is a significant property because it appears widely in engineering calculations. It is found frequently in formulae that relate one material property to another, often where the underlying physics involves inertia. For example, the Newton–Laplace equation $v = (E/\rho)^{1/2}$ links the velocity of sound v, the Young's modulus E and the density ρ. In other cases, the density serves to relate quantities defined in terms of mass to other properties defined in terms of volume or length. Thus, the density ρ enters into the definition of the composite engineering property known as the thermal diffusivity $a = \lambda/\rho c$, where λ is the thermal conductivity and c is the specific heat capacity. The quantity ρc is the heat capacity per unit volume.

Density may, of course, be an important criterion for materials selection in its own right if weight is a primary concern in design.

Thermal properties

In civil engineering, the thermal behaviour of materials is important in many situations and for many reasons. It controls the heat loss from buildings, and determines the response of structures to fires, to mention two important areas of design and performance. More generally, there is often a close coupling between thermal and mechanical effects, since in constrained components changes of temperature may produce changes of stress (*thermal stress*).

The engineering properties in the thermal cluster allow us to answer questions such as:

- How does an engineering component change size when heated?
- How well does it transmit heat?
- How will its temperature change when it is heated?
- At what temperatures can it be used?

There are four main engineering properties in the thermal cluster. These are:

- thermal expansivity
- thermal conductivity
- specific heat capacity
- melting temperature.

Thermal expansivity

Its definition is simple and precise: we write

$$\alpha = \frac{1}{l}\frac{\mathrm{d}l}{\mathrm{d}T} \tag{1}$$

where l is a length and T the temperature. The equation immediately makes clear what is the dimension and hence what are the units of thermal expansivity: dl/l is a strain (and itself dimensionless) and T is a temperature, so α has dimension [temperature]$^{-1}$ and the SI unit is K^{-1}.

Typical values of expansivity are:

- for metals, $10\text{–}20 \times 10^{-6}\,\mathrm{K}^{-1}$
- for ceramics and glasses, $1\text{–}10 \times 10^{-6}\,\mathrm{K}^{-1}$
- for polymers, $30\text{–}300 \times 10^{-6}\,\mathrm{K}^{-1}$.

A factor of about 300 spans the thermal expansivities found in engineering materials.

In many cases, we find that the value of α is more or less independent of temperature. This is generally true of engineering metals and ceramics but less so for polymers. If α is independent of T we can write Equation (1) in the more practical form $\alpha = (l - l_0)/[l(T - T_0)]$. The thermal expansivity as defined in Equation (1) describes the change in linear dimension, such as the change in length of a bar (or equally its width). It should be clearly distinguished from the volume thermal expansivity β, but for isotropic materials $\beta = 3\alpha$ exactly. It follows directly that the change in density ρ with temperature is given by $\mathrm{d}\rho/\mathrm{d}T = -\beta\rho$.

Figure 4 shows density data for pure iron over a large temperature range. The density falls as the temperature increases because the material expands. The thermal expansivity can be calculated from the slope of the line. It is evident that from ambient temperatures up to about 700°C the expansion of iron is an approximately (but not exactly) linear function of temperature, so that a single value of α adequately describes the expansion behaviour. However, we see a striking discontinuity in the density at 912°C, where the density abruptly increases by about 1%. Beyond the discontinuity, the expansion once again continues, but with a slightly greater slope.

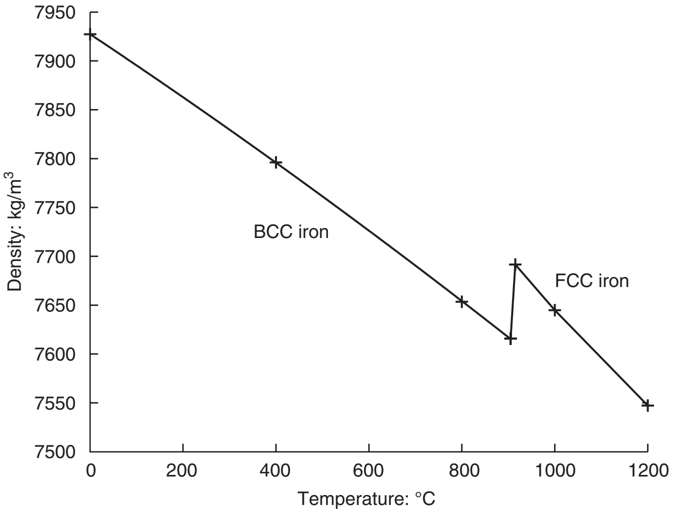

Figure 4 Density of iron vs temperature

The discontinuity is caused by a structural reorganisation of the iron crystal lattice – the arrangement of iron atoms within the material. At the lower temperatures, the iron atoms are arranged on a cubic grid, with atoms located not only at the grid points but also at the centres of each elementary cube. This is the so-called body-centred cubic or BCC structure. Above the transition temperature, a different structure becomes thermodynamically more stable (has lower energy). This high temperature structure is also a cubic lattice but now iron atoms are located at the centres of the faces of each elementary cube rather than at the cube centres.

This face-centred cubic or FCC structure has a higher packing factor (0.74) than the BCC structure (packing factor 0.68), hence the increase in density as we pass upwards through the transition temperature. Because pure iron can pass from one structure to another by a relatively slight and easy repositioning of atoms, the transition is rapid and reversible. We shall see later, however, that, in the case of steels, in which iron is combined with small amounts of other elements, notably carbon, this transition is strongly modified. It occurs at lower temperatures (adding 0.3 wt% carbon lowers the transition temperature to 700°C) and it may become sluggish, allowing the high temperature structure to be trapped at lower temperatures. On this phenomenon depends much of the practice of steel production. Another example is the α–β phase transition in quartz, which occurs at 573°C. This is also a rapid structural phase change, giving rise to a 2% volume increase on heating. Quartz is a primary component of granite, quartzite and other siliceous aggregates used in concrete, and this phase can produce explosive spalling in concretes under fire conditions. Generally, structural transitions of this kind are common in materials. Their possible occurrence should always be considered.

Volume expansion occurs with increasing temperature not only for solids but also, of course, for both liquids and gases. As in **Figure 5** we see again a graph with volume (or density changes) which are smooth, roughly linear functions of temperature but interrupted by a discontinuity, here the ice–water solid–liquid melting transition. The well-known increase in density on melting is evident, as also is the highly unusual behaviour of water from 0–4°C where water contracts as the temperature increases, thus exhibiting a negative thermal expansivity. β for water increases markedly with increasing temperature: it is $0.88 \times 10^{-4}\,\mathrm{K}^{-1}$ at 10°C but $2.6 \times 10^{-4}\,\mathrm{K}^{-1}$ at 25°C. For gases, volume thermal expansion can be calculated from the gas laws, so that $\beta = (1/V)\,\mathrm{d}V/\mathrm{d}T$ is approximately $1/T$ for all gases since $PV = n\mathbf{R}T$. At normal temperatures, therefore, $\beta = 3 \times 10^{-3}\,\mathrm{K}^{-1}$ for all gases.

The thermal expansion of materials with increasing temperature is essentially universal behaviour. It arises because as temperature rises the material stores increasing

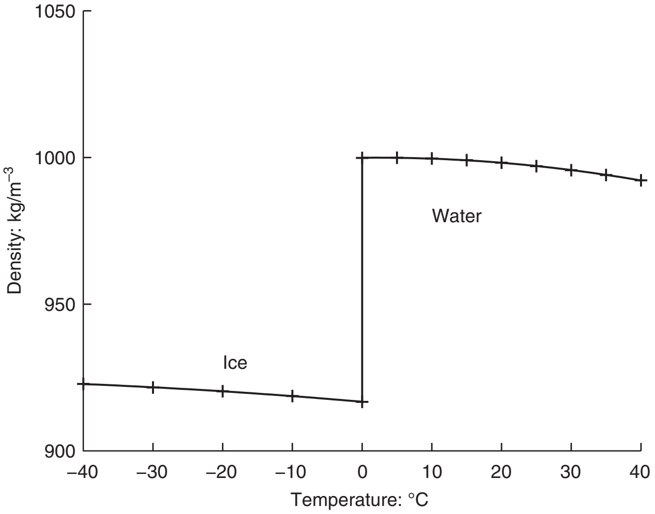

Figure 5 Density of the ice–water system

amounts of thermal kinetic energy, mostly as vibrations within the lattice. At the atomic level, atoms vibrate about their mean lattice positions with ever greater amplitude. In the case of metals, electrons also carry increasing thermal kinetic energy. The vibrational motions of the lattice (or *phonons*) are kept in check by the cohesive forces acting between the atoms and molecules, but as the phonon energy increases, the mean separation of the atoms and molecules increases and we see thermal expansion. The expansion depends on the fact that the interatomic energy curve (the UBER curve of **Chapter 1**) is decidedly asymmetric (the lattice is *anharmonic*), so that as the vibrational amplitude increases, the mean separation of the atoms becomes larger. In fact, the simple harmonic lattice has zero thermal expansivity. This shows that the thermal expansion depends on rather subtle features of the interatomic bonding. However, we can broadly say that the deeper and steeper the interatomic potential energy curve is, the smaller is the thermal expansivity.

We note finally that although positive thermal expansion by simple bond extension is an almost universal feature of the solid state lattice, examples are known of materials with negative thermal expansivities. These arise from rather unusual changes in lattice bonding geometry. They typically involve concertina motions of rigid polyatomic units. An example of such a material is the ceramic zirconium tungstate ZrW_2O_8. A more consequential example in building engineering is the crystal of calcite, which has thermal expansivity of $26 \times 10^{-6}\,K^{-1}$ parallel to the *c*-axis of the lattice; but $-3.7 \times 10^{-6}\,K^{-1}$ normal to this direction (Pavese *et al.*, 1996). Calcite is a primary constituent of marble, and the strongly directional differential thermal strain caused by temperature changes is considered to be a prime cause of deterioration in marble slabs, for example used for cladding. In other materials, progressive changes in crystal structure can largely or entirely cancel out the underlying positive lattice expansion, at least over a range of temperatures. Such is the explanation for the remarkable properties of the iron–nickel alloys of the *invar* family. Invars (**Box 1**) have very low thermal expansivities and the numerical value can be tailored by tuning the Fe:Ni ratio of the alloy.

Box 1 Invars: iron-nickel alloys

Iron-nickel alloys with about 36 wt% Ni are known as invars. They have unusual properties; in particular they have very low thermal expansivity (for invar 36, $1.18 \times 10^{-6}\,K^{-1}$ in the temperature range 25–100°C) and several important specialised applications as a result. Alexander and Street (1988) state: 'A tape of aluminium laid from Marble Arch to Dover would increase in length by 2 m if the temperature rose by 1°C; a similar tape of invar would expand only 7 cm.' Strictly speaking, the term invar is restricted to the 36% Ni composition with the lowest thermal expansivity. By slightly increasing the Ni content in the range 36–50%, alloys may be produced to match the thermal expansivity of many common glass compositions.

Discovery. In 1896, by Swiss physicist Charles-Edouard Guillaume (1861–1938). Invar = 'invariant' in length. Guillaume won the Nobel Prize in 1920, the only Nobel Prize ever awarded for a metallurgical discovery.

Structure. Invars have the FCC Fe lattice with random substitution of Ni. The origin of the low thermal expansivity is now known to lie in the highly unusual magnetic behaviour of these alloys. Around the invar composition, the Fe–Ni alloys can exist in two magnetic states (ferromagnetic and antiferromagnetic) which have very similar energies. The antiferromagnetic state has the lower volume and it is favoured by higher temperatures. Therefore, as the alloy is raised in temperature the normal thermal expansion due to the anharmonicity of the interatomic bonding is opposed by the shift towards the antiferromagnetic state. The details of this magneto-volume effect are only now becoming fully clear, since it requires massive computing power to simulate the magnetic interactions in the unit cell (van Schilfgaarde *et al*., 1999).

Applications. Invars are widely used in the construction of precision instruments, notably in making rods, staffs and tapes for accurate geodetic survey work. A 3 m invar tape changes length by only about 0.04 mm for a 10°C temperature change. Invar alloys are also used in the construction of double-skin hulls for liquified natural gas tankers. The low thermal expansivity greatly reduces thermal stresses.

Thermal conductivity

The thermal conductivity is the property which tells us how well a material transmits heat. Materials with low thermal conductivity can be used as insulators to impede the flow of heat, for example to reduce heat loss from buildings. The defining equation is:

$$J_q = -\lambda \frac{dT}{dx}. \qquad (2)$$

Here J_q is the heat flux density (heat flow per unit area), and dT/dx the temperature gradient. It follows that λ has the dimension [energy/(time × length × temperature)], and therefore the SI unit of thermal conductivity is W m^{-1} K^{-1}. **Figure 6** shows the thermal conductivity of a number of engineering materials. The conductivity range available to the engineer is about five orders of magnitude, a factor of 100 000 between the lowest and the highest values. Metals typically have high and polymers low thermal conductivity. However, we note that single crystal diamond has the highest thermal conductivity of all. The lowest values are found in porous materials where the heat flow is controlled largely by the low thermal conductivity of air in the pores.

We can understand these values at least qualitatively from the lattice model. The transmission of heat from a region of low temperature to one of higher temperature occurs by the diffusion of lattice vibrations or phonons. In all materials, the thermal vibration of atoms about their mean lattice positions in the hotter regions is communicated through the lattice by the coupling of interatomic bonds. The phonons travel most rapidly in crystals with stiff, strong chemical bonds, and most efficiently when the crystal grain size is large and defects are few. This explains the exceptionally high thermal conductivity of large diamond crystals. Grain boundaries scatter phonons, reducing the thermal conductivity considerably. So also do point defects such as lattice substituents, so that alloys generally have lower thermal conductivities than the pure

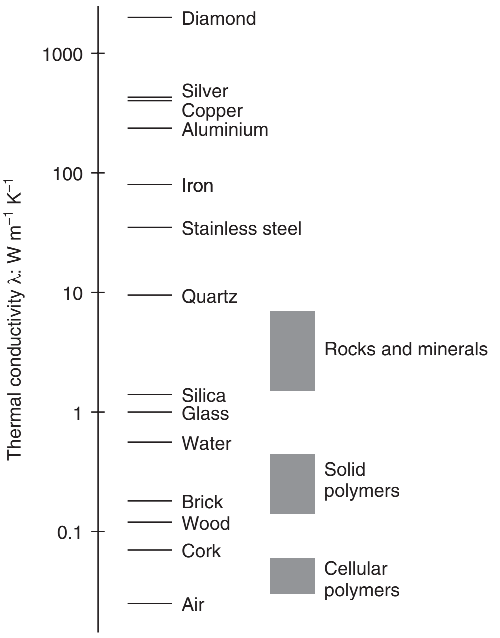

Figure 6 Thermal conductivity of a number of engineering materials

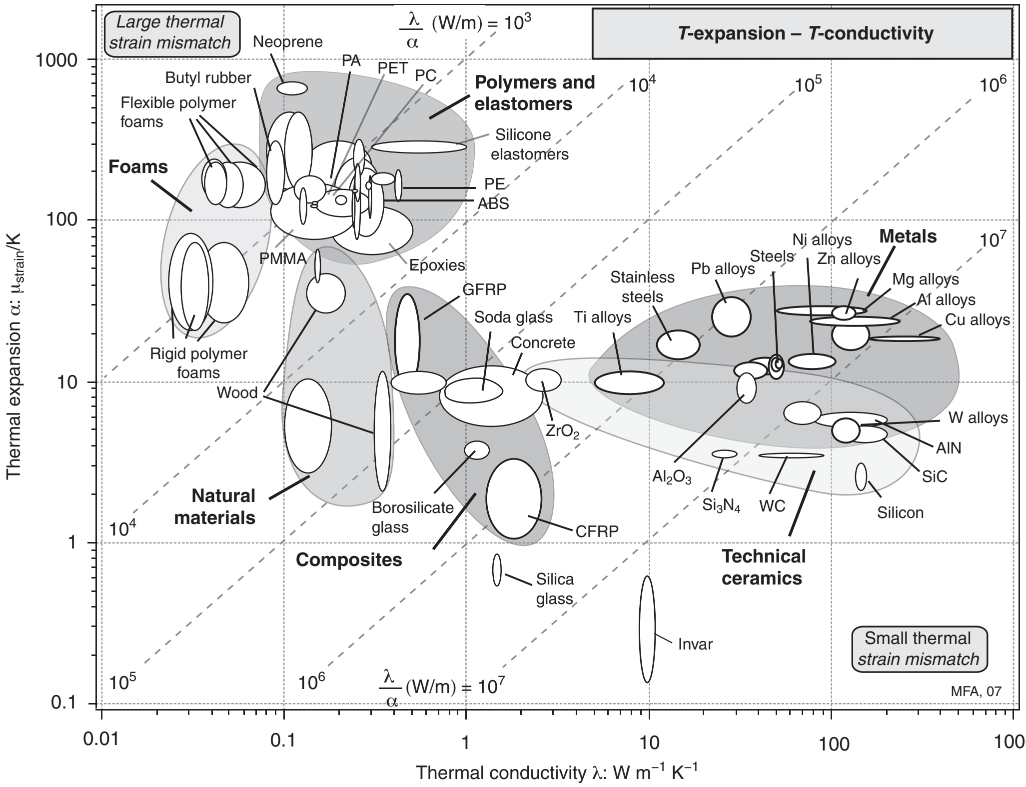

Figure 7 An Ashby chart showing the correlation between thermal expansivity and thermal conductivity (reprinted from Michael F. Ashby, Materials Selection in Mechanical Design, 3rd edition ©2005 Elsevier)

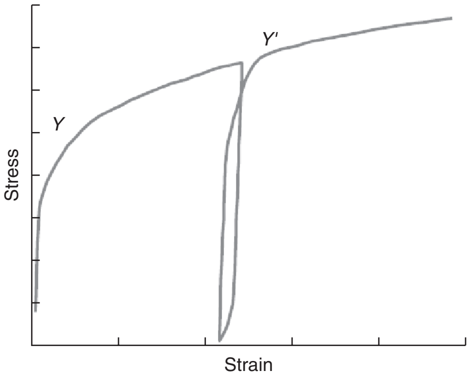

Figure 8 Yield and irrecoverable plastic deformation. *Y, Y′* are successive yield points in a strain-hardening material

metals on which they are based. For example the high chromium stainless steels have thermal conductivities about 10 times smaller than that of pure iron.

While phonon diffusion transports heat in all materials, there is a second mechanism of heat conduction in metals in which thermal energy is carried by free electrons. This explains why metals have relatively high thermal conductivities. It also underlies the empirical Wiedemann–Franz law which states that the ratio of the electrical conductivity and the thermal conductivity of a metal at any temperature is roughly constant. Lorenz later showed that this ratio in fact increases roughly linearly with temperature.

We have noted that materials with strongly bonded lattices tend to have both low thermal expansivity and high thermal conductivity. This provides an example of a correlation between engineering properties which might appear at first sight to have little to do with each other. **Figure 8** shows that over the great variety of engineering materials there is a broadly inverse correlation between these two properties. Of course, it is far from an exact relation. The thermal expansivity is dominated by the interatomic bonding and is not sensitive to defects (or indeed the presence of free electrons), while the thermal conductivity is a more complex *structure-sensitive* property. Nonetheless the broad inverse correlation is found. The Ashby chart shows also the range of thermal expansivities found in engineering materials. For civil engineering, it is a remarkable fact that the thermal expansivities of steel and concrete are more or less the same. If it were not so, then thermal stresses in reinforced concrete would be much greater than they are.

Specific heat capacity

It takes twice as much heat to raise the temperature of a kg of aluminium by 10°C as to raise the temperature of a kg of iron by the same amount. The material property that describes that relation is the specific heat capacity $c = (1/m)\,\mathrm{d}Q/\mathrm{d}T$, where Q is the heat energy, T the temperature, and m the mass of material. It follows that c has dimension [length$^2 \times$ time$^{-2} \times$ temperature^{-1}] and has SI units J kg^{-1} K^{-1}. The designation *specific* indicates that this is the heat capacity *per unit mass*.

The specific heat capacity has a small numerical range. Representative values are: polyethylene HDPE 2.3, magnesium 1.02, aluminium 0.88, glass 0.78, iron 0.39, lead 0.13 kJ kg^{-1} K^{-1}. It has long been known in solid state physics that the specific heat capacities of solid materials vary significantly with temperature (tending to increase towards a limit as temperature rises) and also to decrease with atomic number or atomic weight. Indeed the classical law of Du-long and Petit claims that the specific heat of any substance $c = 3\mathbf{R}/M_\mathrm{r}$ where **R** is the gas constant and M_r the mean molar mass. This is in fact a simple consequence of the fact that as temperature rises the increasing thermal energy of the lattice is spread equally between the various vibration modes, and depends mainly on how many of them there are, rather than the atomic or compositional details. Thus, at room temperature and above, for the metals the law holds rather closely. The law does less well for non-metals, although the trends are correct, low-density materials in general having higher specific heat capacities than high-density materials.

Although the specific heat capacity is of fundamental importance in solid state physics, it is much less significant from an engineering standpoint because of its small numerical range. We have already seen in the previous chapter that it appears in the definition of thermal diffusivity, the material property which controls the response of materials to non-steady heat flow.

Melting temperature

The melting temperature of a substance is a good indication of the strength of its lattice cohesion. A high melting temperature shows clearly that the lattice bonding is able to resist intense thermal vibrations without collapsing to form a liquid. Iron melts at about 1540°C, tungsten much higher at 3420°C. The hard ceramic alumina (aluminium oxide or corundum) melts at 2054°C and diamond at about 3550°C. Thermoplastic polymers, on the other hand, have weak inter-chain cohesion and melt readily; polyethylene at around 100°C. The macromolecular chains themselves retain their integrity in the melt but can move around like snakes in a box. Some high-performance engineering polymers, such as polyetheretherketone (PEEK), have melting temperatures of 300–400°C. For thermoplastics which are often processed by extrusion or moulding, these high melting temperatures may present manufacturing challenges.

Some materials may decompose chemically on heating before they melt. Gypsum, for example, loses water and converts to the lower hydrate mineral bassanite at about 55°C, and by further loss of water to the mineral anhydrite at higher temperature. The melting temperature is of course often of direct engineering significance in relation to the design of components for use at high service temperature. More fundamentally, it is often useful to consider material properties in relation to the *homologous temperature*, the ratio T/T_m where T is the working temperature. In calculating the homologous temperature, the absolute (kelvin) temperature is used. As a rule of thumb, phenomena such as creep which depend on thermally activated transport processes become much more pronounced at homologous temperatures greater than about 0.7. For polyethylene (PE), normal ambient temperatures correspond to an homologous temperature of about 0.75, while for steel the homologous temperature under ambient conditions is only about 0.2.

The existence of a sharp melting temperature T_m at which a material transforms from a solid to a liquid is characteristic of a simple single-phase crystalline material: the melting temperature marks a thermodynamically well defined first order phase transition. We note in passing that experience shows that solids can never overshoot the melting temperature, but on cooling the melt can often persist to temperatures well below T_m. In fact melts always *supercool* to some degree before the crystalline solid nucleates.

In multiphase materials, the individual phases each have their own characteristic melting temperatures. Frequently, the phase or phases with the higher melting temperatures are at least partly soluble in the melt formed by the lower melting phases. This leads to the existence of a range of melting temperature for the material, and a more or less complicated phase diagram. A sharply defined melting temperature occurs only at the eutectic composition(s), for example in a lead-tin solder containing 37 wt% lead. For other compositions the material is in a *pasty* or *mushy* state throughout its melting range. Non-eutectic solders were widely used for making ‘wiped’ joints in traditional lead pipework.

In glassy (amorphous) materials, there is no sharp melting/freezing transition. Such materials characteristically exhibit a progressive softening on heating (most easily expressed as a decrease in viscosity). Similarly, thermoset polymers and elastomers which are cross-linked into a single giant network structure do not have a liquid state and do not therefore show a melting transition with a recognisable melting temperature.

Other thermal properties

The melting point sets an absolute upper limit to the temperature at which solid materials can be used. But, in practice, the maximum working temperature is always appreciably lower. In many materials, there are significant changes in mechanical properties at high homologous temperatures: frequently materials become less stiff, and we have already noted the tendency towards increased creep rates. Durability may also be impaired at high temperatures, with increased rates of oxidation, embrittlement and corrosion. To provide guidance on materials selection, it is common for materials suppliers to speak of an *upper service temperature* or a *maximum working temperature*. This is generally a conservative indication based on practical experience, although in some cases well defined test procedures may be used.

Mechanical properties

Mechanical behaviour in materials is considerably more diverse and difficult to describe than thermal behaviour. While simple linear laws are largely adequate as a basis for defining thermal properties, linear laws provide only a starting point for describing the mechanics of materials. Furthermore, the mechanical behaviours of metals, ceramics and polymers are qualitatively different and a fully integrated description is elusive. There is, however, some common ground in small strain elastic phenomena.

The diversity and complexity of mechanical behaviour in engineering materials can be seen immediately if we try to attach precise scientific meanings to everyday descriptors such as stiff, rigid, strong, brittle, tough, hard. To capture these attributes in well defined and distinct engineering properties is an important aim. Likewise we shall see that materials science can to some extent account for the characteristics that they describe through an underlying lattice model of the solid state.

Reversible (elastic) deformation

We want to define engineering properties which describe how materials respond to stress and deformation. For example, we might ask how a rod of uniform cross-section behaves when a constant tensile load is applied. (Equally well, we might examine deformation caused by the axial compression of a solid cylinder or the bending of a rectangular plate.) For all solid materials, for sufficiently small strains (up to a more or less well defined *proportional limit*), we find that the longitudinal strain ε is proportional to the longitudinal stress σ, so that we have the simple linear law $\sigma = E\varepsilon$. Such behaviour is similar to that of a Hooke’s law spring and materials which behave in this way are said to be *hookean* (**Box 2**). The stress depends only on the strain and vice versa. The response is generally essentially instantaneous and fully reversible. On removal of the stress the strain returns to zero. This fully reversible behaviour is said to be *elastic*, and the simple *constitutive equation* is that of linear elasticity. E is an elastic modulus, the Young’s modulus, a property of the material. E is a measure of the stiffness of the material. E can be defined and measured for all materials. It has a large numerical range in engineering materials. For many materials, E changes little over practical temperature ranges, although this should not be

Box 2 Hooke, Young and Poisson

Robert Hooke 1635–1703. Hooke's main scientific contributions were in writing the first book on microscopy and in original work on springs and clocks. His linear law of the spring ('*Ut tensio, sic vis*') is the basis of the simplest constitutive law of linear elasticity. Solid materials which behave in a linear elastic manner are often described as hookean.

Thomas Young 1773–1829. Young made important contributions to solid mechanics, fluid mechanics and optics. His valuable legacy in elasticity was in identifying the modulus as a material property rather than a system property. His important work in capillarity phenomena was clouded by acrimonious relations with Laplace, with whom he is nonetheless perpetually associated through the Young–Laplace equation for the pressure in a bubble.

Siméon Denis Poisson 1781–1840. Poisson was a nineteenth century French scientist of extraordinary range who contributed to the mathematical development of solid mechanics.

assumed in materials with low melting temperatures, notably polymers. Polymers are much stiffer below their glass transition temperatures than above.

The general linear elastic response of materials at sufficiently small strains can be traced to the nature of cohesive forces within engineering materials. The chemical bonds acting between atoms and molecules are deformable and for small departures from the equilibrium configurations can be closely approximated by linear deformation laws. The bulk behaviour is simply the collective consequence of a myriad of such interactions. In the lattice model, we see (**Box 3**) that the small strain elastic modulus is directly related to the properties of the lattice bonds. In the harmonic lattice, the modulus has the same value for tensile and compressive deformations, as is generally found for small strains in engineering materials. Materials with strong chemical bonds are stiff (large E) and for the same underlying reason have low thermal expansivity, high melting temperatures (and tend to have high thermal conductivity).

In simple uniaxial deformation, the transverse strain obeys the linear law $\varepsilon_y = -\nu\varepsilon_x$. The constant ν is an important material property known as Poisson's ratio. This property has a small numerical range but plays an important role in the mechanics of materials. We can easily show that if a material under uniaxial stress maintains constant volume as it deforms, ν would have the value 0.5. For many engineering materials, ν is found to be around 0.2–0.3. If such a material is subject to a uniaxial tensile stress, the lateral contraction is less than required to compensate for the increase in length, and therefore as the tensile stress is applied the material *increases* slightly in volume: the volumetric strain or *dilatation* is $(1-2\nu)\varepsilon_x$. There is no fundamental reason why Poisson's ratio cannot take values <0, and indeed a few such *auxetic* materials are known.

Box 3 A simple lattice model of elastic deformation

The UBER model (shown in Chapter 1, Figure 18) describes the lattice energy as a function of lattice spacing. In the absence of applied forces, the lattice spacing has the equilibrium value r_0 at energy U_0. The quantity d^2U/dr^2 in the vicinity of r_0 represents the force/deformation ratio, which we take as a rough measure of the modulus. For small deformations, the force evidently depends approximately linearly on the deformation (whence Hooke's law) and is about the same in tension and in compression.

Of course the UBER curve describes the interaction energy for changes in separation along the line joining two neighbouring atoms or molecules. The stiffness in other directions relative to the lattice vectors is somewhat different, and the complete elastic response of a single crystal must be defined by a stress–strain tensor. For a cubic crystal, there are just three elastic constants, but for crystals of low symmetry, the elastic stiffness tensor has many components (up to a maximum of 21). The full stiffness tensor can be obtained by acoustic or X-ray diffraction methods. Since most bulk materials are polycrystalline and effectively isotropic, the engineering properties are captured in a mean elastic modulus. For the important example of α-iron, the body-centred cubic form of the metal stable at normal temperatures, the elastic constants $c_{11} = 230$ GPa and $c_{12} = 135$ GPa. These combine to give the engineering Young's modulus $E = 217$ GPa (Adams *et al*., 2006).

Our discussion of Poisson's ratio shows that in uniaxial deformation, such as is used to determine Young's modulus, the material changes in both shape and volume. This suggests that E is a composite quantity. We therefore introduce two further elastic moduli: the shear modulus G defined by $\tau = G\gamma$ which describes the response to a shear stress τ and which involves changes only of shape; and the bulk (compressive) modulus K defined by $\Delta P = -K\Delta V/V$ which produces only changes in volume (but not of shape) in response to an applied hydrostatic pressure P.

The relations between E, G, K and ν are important. Of the four quantities (material properties) only two are independent: knowing any two, the other two can be calculated. We have $E = 2G(1+\nu)$, $E = 3K(1-2\nu)$ and $E = 9KG/(3K+G)$. These relations are exact for linear elastic materials. For soft (compliant) materials, $G \ll K$ and therefore $E \approx 3G$ (and $\nu \approx 1/2$); for stiff materials, the relations between E, G and K depend on Poisson's ratio. For many structural materials, ν is about $1/3$, so that $E \approx 8G/3 \approx K$. In advanced work on elasticity, it is common to introduce a fifth quantity, the Lamé constant (or Lamé modulus) $\lambda = 2\nu G(1-2\nu)$. The use of the Lamé constant simplifies many of the equations which arise in elasticity theory, but treated as a material property λ does not have a simple defining equation nor a simple physical interpretation. The relation between the five elastic constants shows – once again – that material properties may not be independent of one another.

We note that the linear equations such as $\sigma = E\varepsilon$ may equally well be written with the strain as the dependent variable: $\varepsilon = D\sigma$ where D is called the elastic compliance. Here $D = E^{-1}$. Similarly we may write the constitutive equations $\gamma = J\tau$ and $\Delta V/V = -K\Delta P/P$ to define the shear and bulk (compressive) compliances (also called simply the *bulk compressibility*). The two conventions are equivalent. Mechanical behaviour in metals and ceramics is frequently described in modulus language; the behaviour of polymer materials more often in terms of compliances.

The mechanical properties of the hookean solid form the material basis of the mathematical theory of linear elasticity (Timoshenko and Goodier, 1970).

Strength

A sample of material subject to pure hydrostatic compression (compressive stress equal in all directions) can only respond by contracting in volume and increasing in density. The atoms or molecules of which it is composed are squeezed ever closer together but the concept of *strength* has no meaning since the material cannot 'fail' or rupture. Geoplanetary science provides many excellent illustrations of how materials behave at extreme pressures. The Earth's core is composed of solid iron at a temperature of about 6000 K and a pressure of about 350 GPa. The estimated density is about 13 000 kg/m^3 (Stixrude *et al.* 1997), slightly less than twice the density of unstressed iron at the Earth's surface. The opposing effects of pressure and temperature have reduced the interatomic spacing by about 30%.

In a perfect crystal lattice, it would be expected that even under tensile and shear loading the chemical bonds would be capable of extension by at least 10% and still maintain the cohesion of the solid. Both experiment and theory confirm that strains of the order of 0.1 can occur in small defect-free crystals (such as carbon nanotubes and graphene sheets) and even in small regions of bulk materials. At higher strains comprehensive rupture of chemical bonds occurs, so that we expect an *ideal* or maximum strength of the order of $0.1E$ for a solid material of any kind (for example about 20 GPa for iron, Clatterbuck *et al.*, 2003). In fact such elastic strains and ideal strengths are never observed in bulk materials because at much lower strains other processes intervene. In brittle materials, materials fail in shear or tension by developing cracks, which may propagate catastrophically to cause *brittle failure*. In ductile or plastic materials, the material yields irreversibly by one or more of several mechanisms, notably by slip processes which take place by the movement of dislocations. In both cases, the cohesion of almost all the solid material is maintained after failure. In brittle fracture, cohesion is lost only across the fracture surface; while in plastic deformation there is no loss of cohesion, merely an irreversible (and often radical) reshaping.

Practical measures of strength include the *yield stress*, the *ultimate tensile strength*, the *uniaxial compressive strength* and *the failure stress*. These quantities are not always well defined operationally. In any case, test results are often subject to considerable scatter. Strength is therefore an engineering property that should be handled with great care.

Non-hookean mechanical behaviour

The hookean solid provides only a starting point for describing the huge variety of behaviour found in the mechanics of engineering materials. There are many extensions to this description, although most retain the hookean response for small strains. Of the many extensions, we list:

- Nonlinear but fully elastic stress–strain behaviour (beyond the proportional limit) (nonlinear elastic).
- Retarded elastic or anelastic behaviour, where the elastic stress–strain response is not instantaneous.
- Plastic behaviour, where deformation is permanent and not recovered beyond a more or less well defined yield stress (plastic or elastoplastic).
- Time-dependent linear stress–strain behaviour (linear viscoelasticity).
- Behaviour in granular materials such as soils and porous materials such as rocks where stress–strain relations depend on liquid pressure.

We discuss briefly what underlies some of these phenomena and some additional engineering properties which arise in developing constitutive models of such behaviour.

Irreversible (plastic) deformation

Elastic (fully reversible) deformation is a general property of solid materials found universally at sufficiently small strains. In *brittle* materials, behaviour is elastic up to a point at which catastrophic failure occurs. However, in many materials, elastic behaviour extends only as far as a more or less well defined *yield* point, beyond which the material maintains cohesion but deforms *plastically*, **Figure 9**. Plastic deformation is not recovered or reversed on removal of the applied stress. From a microstructural point of view, the crossover from elastic to plastic behaviour occurs when simple lattice stretching (without bond breaking) is overtaken by large-scale displacement of lattice planes. This occurs commonly through the movement of dislocations on *slip planes*. Such dislocation slip is generally easy in pure metals, especially in those with FCC lattice structures, where numerous slip planes exist. The movement of dislocations is significantly impeded by grain boundaries in polycrystalline materials. By making the grain size smaller, the onset of plastic deformation is delayed and the yield stress, σ_y, is raised. According to the Hall–Petch relationship $\sigma_y = a + bd^{-1/2}$ where d is the grain size and a and b are material constants. Dislocation

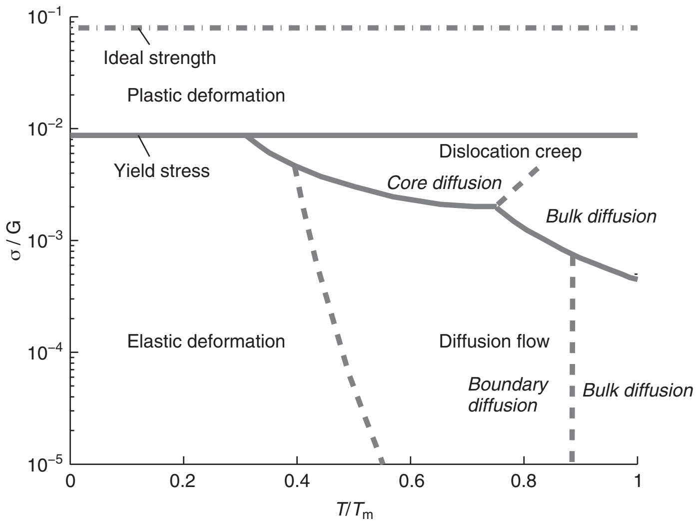

Figure 9 Deformation mechanisms at different normalised stresses and homologous temperatures (adapted from Ashby and Jones, 2005)

movements are impeded by various distortions introduced by alloying, so that alloys are stronger than pure metals. In particular, solute atoms in alloys stabilise dislocations. The microstructure is also greatly altered by *annealing*, where soaking at a high homologous temperature brings about diffusive recrystallisation of the grain structure; and *work hardening* in which plastic deformation causes dislocations to become entangled and less mobile. Thus, in metals, there are a number of powerful methods of modifying the mechanical properties by control of microstructure and dislocation density. Plastic deformation occurs in polymer materials largely through thermally activated slip between adjacent polymer chains. Here, plastic deformation may be prevented by chemical bonding between chains, as in cross-linked rubbers. The cross-linking tags the chains so that on relaxation of stress the chains can recover their original positions even though very large strains may have been imposed.

Soft materials: viscoelastic behaviour

An important deviation from hookean behaviour is found in many polymers (plastics and rubbers), which can also include bitumens. A sample of material subject to a constant applied stress shows a strain which increases with time. The stress–strain ratio $\sigma/\eta = E$ is no longer a material constant but a function of time, that is $E(t)$. Such materials exhibit pronounced *creep*. Likewise, if we deform a material to a constant strain, we observe that the stress falls with time: the sample exhibits *stress relaxation*. These are two important manifestations of *viscoelastic* behaviour, in which the material exhibits a combination of elastic and viscous flow characteristics. Models of viscoelastic constitutive behaviour can be developed by considering series and parallel combinations of hookean and newtonian elements.

The creep and stress relaxation which are characteristic of thermoplastic polymers arise because the separate polymer chains are not linked through primary chemical bonds and, given time and adequate thermal energy, can slide past each other in response to an applied stress. The relative motion of polymer chains is impeded by entanglement but occurs given sufficient time. Most mobility resides in the amorphous regions of a semi-crystalline polymer and the crystallites also serve to tie separate chains together. Because mobility depends on thermal energy, creep rates (measured, for example, by some suitably defined creep strain rate $\dot{\gamma}$) are strongly temperature dependent. On cooling, a temperature T_g is reached at which molecular mobility is frozen out. This marks the glass transition below which large-scale chain mobility is absent. The material becomes brittle and glassy.

Creep

Creep is a familiar characteristic of polymer materials which are above their glass transition at normal service temperatures. It arises because, superimposed on the instantaneous elastic response, slower irreversible or retarded deformation processes come into play. This is one example of a more general feature of the mechanics of materials: that if we explore a sufficiently wide range of conditions, most materials exhibit complex responses going well beyond simple elastic hookean behaviour. We have seen that high homologous temperatures promote creep, and that dynamic response depends on timescales of internal microstructural reorganisation. We have also seen that high stress promotes plastic yield in ductile materials. Constant stress creep rates are generally not proportional to the stress but increase more rapidly, often following a *power law* such that $\dot{\gamma} = a\sigma^n$ where a is a constant and the power n is in the range 3–8. Power law creep is found widely in metals, ceramics and polymer materials.

It is therefore challenging to describe and integrate such diverse behaviour, and this makes the definition of mechanical properties difficult. However, a synoptic view is provided by the deformation-mechanism maps constructed by Frost and Ashby (1982). **Figure 10** shows such a map in schematic form. Here the stress normalised by the shear modulus is used as a master variable together with the homologous temperature T/T_m. At the left side at low homologous temperature, we see the familiar low stress elastic behaviour which gives way at a well defined yield stress to purely plastic deformation. At higher homologous temperatures, creep becomes significant at stresses well below the yield stress and which fall with temperature. Power law creep involving dislocation motion gives way to diffusion mechanisms at higher temperatures.

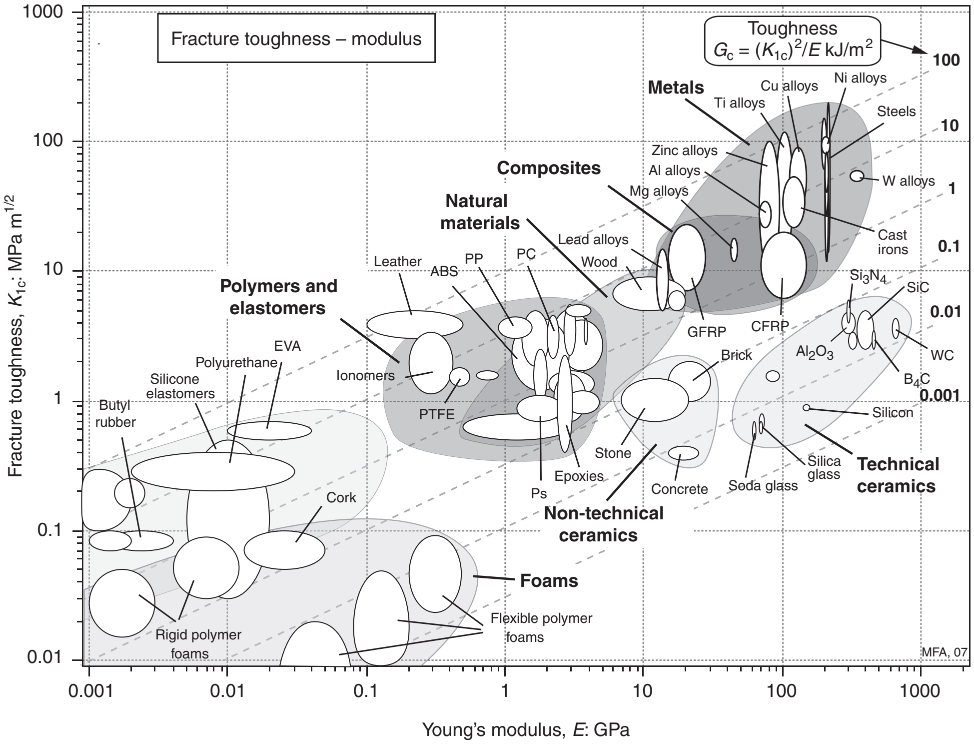

Figure 10 Ashby chart: fracture toughness vs strength (reprinted from Michael F. Ashby, Materials Selection in Mechanical Design, 3rd edition ©2005 Elsevier)

Fracture toughness and impact

Brittle materials characteristically fail by catastrophic fracture without any prefailure or premonitory yield. This behaviour is generally found in materials in which mechanisms of plastic deformation and yield such as slip or creep do not operate. In such cases, it might be hoped that materials would show high failure strengths approaching those of the ideal solid, but it is generally found that failure strengths in brittle materials are disappointingly low. The subject of fracture mechanics (and especially linear elastic fracture mechanics in which hookean constitutive behaviour is assumed) has developed to guide safe design in brittle materials and to underpin improvements in their mechanical properties.

The early experiments of Griffith (**Box 4**) on glass showed that freshly drawn glass fibres showed remarkably high tensile strength, but that this rapidly declined with age over days and weeks. Griffith suggested that the glass was progressively weakened by microscopic flaws or microcracks. In the vicinity of the tip of an elongated microcrack, the far-field or mean stress acting on the material is greatly amplified, typically by several orders of magnitude, to the point where the material ruptures at something which may approach the ideal strength. For sharp cracks, the stress concentration factor scales as $l^{1/2}$, the square root of the crack length l.

Griffith was concerned with the question of why cracks of this kind may sometimes be stable and sometimes propagate rapidly and catastrophically (**Box 4**). He proposed a fundamental stability criterion which, in a generalised form, is the basis of modern fracture mechanics. According to Griffith's theory, a crack becomes unstable when the elastic strain energy released by an incremental increase in crack length equals or exceeds the increase in fracture surface energy. Broadly speaking, the incremental strain energy released increases as $l^{1/2}$, while the surface energy required to extend the crack is independent of crack length. Therefore, as cracks become longer, a critical length is reached at which the Griffith energy balance criterion is satisfied and at which the crack becomes unstable. In glass, an ideal brittle material, the energy

Box 4 Alan Arnold Griffith 1893–1963

Griffith made seminal contributions to understanding the difficult topic of brittle fracture. He established that brittle materials such as glass, ceramics and many metals are weakened greatly by the presence of microscopic flaws and defects (*Griffith flaws*), which act as stress concentrators. He proposed the *energy balance criterion* for unstable crack growth which led to the development of linear elastic fracture mechanics. He made fundamental advances in the design of gas turbines and jet engines during a career at the Royal Aeronautical Establishment, Farnborough and with Rolls Royce.

needed to extend a crack is indeed mainly crack surface energy. In other materials (glassy polymers, metals) other processes such as crazing and local plastic deformation may absorb energy in the vicinity of the growing crack tip and these increase the critical crack length. The resistance of a material to unstable crack growth is embodied in the *fracture toughness* property.

Fracture toughness is a complex material property which is measured in several standard test procedures, most of which determine the influence of a deliberately created sharp-tipped notch on the strength of a material specimen in tensile loading or bending. Although the stress distribution around the crack may be unknown in detail, it is a general result that the stress amplification or *intensity factor* K_{I} scales with $l^{1/2}$, so that, for any material failing by a brittle process, the measured failure stress σ_{f} varies with $l^{-1/2}$. Thus we can define a quantity $K_{\mathrm{Ic}} = \sigma_{\mathrm{f}} l^{1/2}$ as a measure of the fracture toughness. The dimension of K_{Ic} is [stress × $\mathrm{L}^{1/2}$] and common units $\mathrm{MPa\,m}^{1/2}$.

Typical values of the fracture toughness are: for metals (and some fibre-reinforced composites), 10–100 $\mathrm{MPa\,m}^{1/2}$; for polymers and ceramics 1–10 $\mathrm{MPa\,m}^{1/2}$. The Ashby chart showing the relation between fracture toughness and strength is shown in **Figure 10**. The toughest materials are metals capable of relieving stress concentrations in the crack tip by extensive plastic yield. Polymer materials above their glass transition temperatures can also be usefully tough, although their intrinsic strength is more modest. Below their glass transition temperatures, where polymers are stiffer, the toughness is enhanced by polymer chain extension across crack surfaces (*crazing*). Similar dissipation mechanisms act on a larger scale in fibre-reinforced concrete, where fibre pull-out stabilises cracks and promotes a valuable pseudo-plastic yield. Ceramics and glass as a group have low toughness, and good general toughening mechanisms have proved elusive. In zirconia, an unusual mechanism associated with a polymorphic phase transformation in the process zone significantly increases the fracture toughness.

Resistance to impact damage is an important practical requirement in many materials. There are several well-established test methods for obtaining an empirical *impact strength*, notably the Charpy and Izod tests. These tests both use a pendulum to deliver an impact to a notched test specimen, and can be used on all major types of materials.

Hardness

Hardness is used in the mechanics of materials to mean resistance to indentation by a point load. It is therefore a useful property to describe resistance to particular kinds of damage in service. Hardness is measured by several long-established test methods in which the size of the indentation formed by a shaped indenter under a prescribed load is measured. Hardness is an example of a property which is only defined operationally in relation to closely specified test procedures. It is not a fundamental material property. The Vickers, Rockwell and Brinell hardness scales are the most widely used for metals; the Vickers scale is used for ceramics; and the Shore *durometer* scale is used for polymers and particularly rubbers.

The mechanics of the indentation process is highly complex, so that hardness values obtained in these tests cannot be expressed generally in terms of more fundamental mechanical properties. However, it is well established that for certain metals the hardness is proportional to the yield stress (see, for example, Tabor, 1951).

Fatigue

In service, many engineering structures are subject to variable or repeated stresses. Causes may lie in traffic or wind loading, for example; or in repeated cycles of pressurisation in tanks and pipes. For many materials, it is found that the failure stress decreases as the number of stress cycles increases. This phenomenon is called *fatigue*. It can be attributed to the slow accumulation of material damage (for example, microcrack or dislocation formation, or subcritical crack extension) (**Figure 11**). Fatigue processes occur in all families of materials: metals, polymers and ceramics. Standard tests for characterising fatigue apply cyclic stress to the sample until failure occurs. The well-known Wöhler S/N curve shows how the number of cycles to failure varies with stress amplitude σ_0. For some materials, there is a more or less clearly defined limiting stress (*fatigue limit*) below which cyclic loading does not produce further reductions in strength. In a cyclic strain test, the peak stress may increase or decrease with time for metals; but polymers invariably exhibit cyclic softening. According to Basquin's

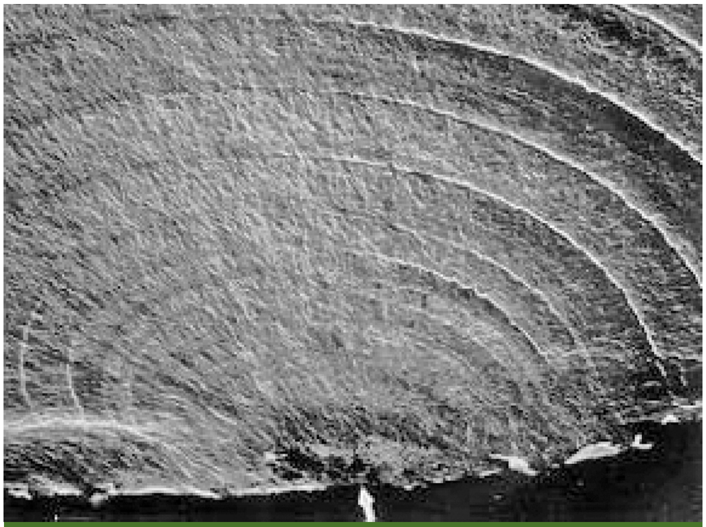

Figure 11 Beach marks indicating slow crack growth in cyclic fatigue (courtesy of Metallurgical Technologies Inc., P.A.)

empirical power law, the fatigue failure lifetime $t_f \sim \sigma_0^{\beta}$ where β is a constant, which however varies widely from material to material. This law is now understood to be a statistical consequence of how microscopic damage accumulates within the material.

In some materials, even prolonged *static* loading is sufficient to produce a decrease in failure stress. Such is the case in glass, where *static* fatigue has long been recognised. Here the origin of the effect lies in chemical interactions between material and environment. Similar effects are found also in many polymers.

We also note that in viscoelastic or plastic materials in which a rapid cyclic deformation produces a non-elastic response, the dissipation of energy generates heat within the material. This thermal effect may contribute greatly to fatigue damage. Apart from reducing stiffness, the rise in temperature may cause polymer molecular chains to break. Polymers are particularly prone to thermal fatigue because of their generally low thermal conductivities.

Friction

The classical laws of friction (Amontons' laws) state that the force needed to achieve steady sliding between a block and an extended surface is proportional to the normal force acting on the block; and is independent both of the area of contact and of the sliding speed. The coefficient of friction is the constant of proportionality μ. μ depends on both materials: the slider and the block. In fact the laws hold quite well for hard, stiff materials (ceramics and many metals), but less well for soft, deformable materials such as viscoelastic polymers (thermoplastics and rubbers). The friction process has a microscopic underpinning: the inevitable roughness of surface and countersurface mean that the true contact area is less (often much less) than the apparent contact area. The surfaces touch only at the tips of the asperities, where the local stress is high. A simple elastic deformation model shows that the true contact area increases as the normal stress. Energy is dissipated in breaking the cohesive bonds formed during these transient contacts. Amontons' laws fail for polymers because the deformation dynamics involves viscoelastic processes, which introduces a complex dependency of μ on sliding speed and temperature. μ does not have a large numerical range: it lies between about 5 (rubber on steel) and about 0.05 (PTFE on steel).

Properties of liquids

Most engineering is done with solid materials – but not all. It is therefore worth giving some attention to the engineering properties of liquids. We have already mentioned the volume thermal expansivity of liquid water. Our discussion of viscoelasticity suggests that newtonian liquids can be considered as a limiting case of the viscoelastic material with zero shear modulus. The shear behaviour is described through a simple viscous flow law (due to Newton), in which the shear strain rate is proportional to the shear strain rate (or velocity gradient). Liquids which show this linear behaviour are known as *newtonian*. The viscosity of liquids generally decreases noticeably as the temperature rises. The viscosity controls Darcian flow through permeable materials such as concrete and soils; and underlies Poiseuille's equation for pipe flow under laminar flow conditions at low Reynold's numbers. We note that the Reynold's number is not a material property since it depends on the velocity of flow, so it is an intensive system property. However, the Reynold's number contains the property factor η/ρ, sometimes called the *kinematic viscosity*. This is another example of the involvement of density in a composite engineering property.

In some other respects, liquids behave like solids: the same definition of compressibility applies to both, so that a bulk compressibility can be defined and measured for liquids as for solid materials. For water at 25°C this is $0.452\,\mathrm{GPa}^{-1}$, about three times larger than that of ice at 0°C. Of particular importance in the mechanics of liquids is the surface tension property which controls capillarity phenomena, including wetting and spreading behaviour, and the shapes of droplets, bubbles and menisci. Some property data for water are collected in **Table 1**. These data show that some engineering properties vary appreciably with temperature, especially as here at high homologous temperature. Liquids are of course isotropic in their properties. The definitions of thermal properties that apply to solid materials carry over to liquids for the most part unchanged. Values of volume thermal expansivity β are generally around $1\text{–}10 \times 10^{-4}\,\mathrm{K}^{-1}$.

Mass transfer properties

The ability of a material to resist penetration by substances such as water, oil and oxygen (to take just three common examples) is often an important consideration in engineering design. This is an obvious requirement in choosing materials for tanks, pipes and membranes. However, mass transfer (especially transport of water) is intimately involved in many processes of material degradation. Understanding water absorption (and water flux) therefore contributes greatly to our knowledge of durability. This is conspicuously true in civil and construction engineering since so many of the primary materials (brick, stone and concrete, soil, and wood) are porous and permeable.

In describing mass transfer processes, we make use (once again) of some simple linear laws, here relating flux to pressure or concentration gradients. Similar models apply whether the fluid is a liquid or a gas. However, it is useful to distinguish between situations where the fluid passes more or less as a bulk substance through distinct pores or channels within the material; and where it moves as a molecular substance after dissolving in the material through which it is passing. In the latter case, the material need not be porous.

Permeability

In materials with open or connected porosity, water and other liquids can be absorbed by the action of capillary forces and can flow under the action of hydrostatic pressure gradients. The importance of such processes in civil engineering can hardly be overstated since all the main inorganic construction materials – brick, stone and concrete – are porous. Water in these materials mediates many processes of alteration and deterioration, and so mass transfer properties are closely linked to durability.

The water absorption and flow processes are essentially much the same as those which occur in soils. How easily water may flow through the material is captured in the permeability property. Following the original work of Henry Darcy, mathematical descriptions of liquid flow in porous media are based on Darcy's law. This states that the volumetric flow rate Q of liquid through a specimen of porous material is proportional to the hydrostatic pressure difference p across the specimen, inversely proportional to the length L of the specimen, and proportional to the cross-sectional area A. Darcy's law is expressed simply as $Q = kA\Delta p/L$. We call the constant of proportionality defined by Darcy's law the Darcy permeability of the material. The quantity Q/A has dimension [LT^{-1}] and is the flow rate, flux or Darcy velocity u, so that Darcy's law is more commonly written as $u = k\Delta p/L$. It is clear that Darcy's law is a simple linear transport law and has exactly the same form as Ohm's law, Fick's law and the heat conduction equation. Each of these laws defines a transport property, a conductivity, that relates to the flow process that each describes, and we might envisage a unifying theoretical framework which could have application to heat, diffusion and electrical current and to water flow. In fact valuable insights into water flow processes may be gained by comparing these with heat flow problems.

If AB in **Figure 12** is fully saturated with liquid, then imposing a difference in hydrostatic pressure $\Delta p = p_A - p_B$ between A and B leads to a steady Darcian flow $u = k\delta p/L$. u is the (scalar) flow rate (dimensions [LT^{-1}]). Darcy's law may be expressed locally as $\mathbf{u} = k\nabla p$, with $\mathbf{u}$ the vector flow velocity. We do not assume that the Darcy permeability k is necessarily constant in time and indeed for water in concrete there is evidence that this is not always the case.

The permeability k as defined in this equation has dimensions [M^{-1}L^{3}T]. However, it is common to express hydrostatic pressures in terms of the pressure potential $P/\rho g$, where ρ is the liquid density. P has dimension L and is entirely equivalent to the hydrostatic head. We then express Darcy's law as

$$u = -K_s\nabla P \tag{3}$$

where $K_s = k\rho g$. The quantity K_s is the conventional saturated permeability of the material, with dimensions [LT^{-1}].

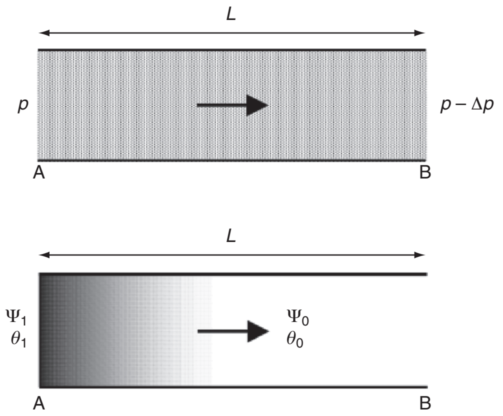

Figure 12 Darcy flow and capillary absorption (adapted from Hall and Hoff, 2002)

Intrinsic permeability

The permeability K_s depends both on the material and on the fluid. For permeation flows which are geometrically similar (such as newtonian liquids in laminar flow in inert non-swelling media), the permeability k (and K_s also) varies inversely as the fluid viscosity η. We can therefore define an intrinsic permeability k' such that $\mathbf{u} = -(k'/\eta)\nabla p$, in which k' is a material property independent of the fluid used to measure it. k' has the dimensions [L^2]. Both definitions of permeability are used, although K_s more widely than k'. The conversion between the two depends not only on the viscosity but also on the density of the fluid at the temperature of measurement since $k' = K_s\eta/\rho g$. For water at 25°C, k'/K_s is 9.1×10^{-8} m s.

Some indicative values of permeability for construction materials are given in **Table 1**. In this small group of common materials, the highest and lowest permeability differ by a factor of 10^6. This reflects the underlying dependence of the intrinsic permeability k on l_c where l_c is some characteristic length scale of the pore system. Of course these materials are not geometrically similar, but nonetheless the sensitivity of the permeability to the fine-grainedness of the porosity is apparent. There are some examples of the permeability of cement-based materials in **Table 2**. The wide range of these values is notable, extending over at least four orders of magnitude. This of course is hugely greater than the range of porosity and reflects the connectivity and the length scale of the percolating porosity. In dense well-compacted cement-based materials, the pore space is extremely fine-grained. This is the case with

Temperature: °C	Density: kg m^{-3}	Viscosity: mPa s	Surface tension: mN m^{-1}	Vapour pressure: kPa
0	999.84	1.792	75.64	0.611
5	999.91	1.518	74.94	0.873
10	999.65	1.306	74.23	1.23
15	999.10	1.137	73.49	1.71
20	998.20	1.002	72.74	2.34
25	997.04	0.8901	71.98	3.17
30	995.64	0.7974	71.19	4.25

Table 1 Physical and chemical properties of water from 0 to 30°C (adapted from Hall and Hoff, 2002)

hardened cement paste and high-performance concretes made at low water/cement w/c ratios. The wide range of permeabilities and how they depend on free water content, w/c ratios, curing and drying conditions is clear from recent research.

Sorptivity

The conventional permeability just discussed is defined for situations in which the material through which the water flows is fully saturated at all times. In this case, there is no change in the water content of the material during flow, and the total flow inwards across some boundary surface is equal to the total flow outwards through a boundary surface elsewhere. There are situations where structures are fully saturated in use (some marine and underground structures, for example) and for these a knowledge of permeability is necessary. However, it is much more common for construction materials to have moisture contents less than full saturation and for these to vary with time, according to season or circumstance. In such cases, water is absorbed into a porous material by the action of capillary forces, and it is gradients of these capillary forces which cause moisture to move and redistribute within a structure. For these situations (which closely resemble those in the vadose zone in soils), the permeability is not an appropriate engineering property. We need instead a property which describes the tendency of a material to absorb and transmit water (or other liquid) by capillarity. This property is the *sorptivity*.

Material	Permeability K_s: m s^{-1}	Sorptivity S: mm min$^{-1/2}$
Lépine limestone	2.5×10^{-9}	1.00
St Maximin fine limestone	2.5×10^{-6}	0.39
Clay brick, $f = 0.40$	3.2×10^{-8}	–
Clay brick, $f = 0.46$	–	2.35
Clay brick, $f = 0.30$	3.8×10^{-9}	–
Clay brick, $f = 0.07$	–	0.09
1:3 cement-sand mortar	4.6×10^{-11}	–
1:3:10 cement-lime-sand mortar	–	1.94
High strength concrete, moist-cured	7.7×10^{-13}	–
High strength concrete, air-cured	1.9×10^{-11}	–
1:2:4 cement-sand-aggregate concrete w/c 0.55	–	0.19

Table 2 Typical values of water permeability and sorptivity of construction materials (adapted from Hall and Hoff, 2002)

The sorptivity can most easily be defined by means of a simple experiment. An initially dry block of material is placed in a shallow tray of water, so that water is absorbed by capillarity through its base. The amount of water absorbed Δw is measured from time to time by weighing the block. If the area of the base of the block is A, then the cumulative absorbed volume per unit area is $i = \Delta w/\rho_w A$. Plotting i against $t^{1/2}$, the square root of the elapsed time t generally gives a good straight line. The slope of this line is the sorptivity S, with dimension $[\mathrm{LT}^{-1/2}]$, and commonly used units mm/min$^{1/2}$. In **Table 2** we give sorptivity values for a number of construction materials.

Diffusion

It is not necessary for a material to contain even microscopic pores for substances to pass through it. Molecules may pass through a material by diffusion, working their way through the solid material by thermally activated jumps making use of atomic-scale lattice vacancies or passing through cavities of atomic dimensions (Crank, 1975). Such processes are usually rather slow except at high temperatures, but even so can be important especially in thin materials. Thus both oxygen and water can diffuse through paint coatings to feed corrosion reactions even if the coating contains no detectable porosity. Oxygen can enter polymer materials and cause embrittlement by slow chemical oxidation reactions. In most cases, the mass transfer is adequately described by Fick's diffusion law $u_i = D\nabla p$ where ∇p is the gradient of the partial pressure p of the diffusing substance.

A rather different example of molecular diffusion occurs in salt solutions where ions such as chloride and sodium move by random motion through a solvent such as water. This process occurs in wet porous materials such as concrete, brick, stone and soils. The ions migrate under the action of concentration gradients by diffusion through the water in the pores, even if the water itself is stationary. Diffusion through liquids is considerably more rapid than diffusion in the solid state, so that processes of this kind can transport salts deep into the interior of, for example, concrete structures. In these cases, we can define a fickian *diffusivity* property D by $u_i = D\nabla c_i$ where ∇c_i is the gradient of the ion concentration c_i. In some cases, diffusion through the water is retarded by adsorption of the ions on the pore walls, so that the diffusivity is a property of the salt/water/matrix system, rather than of the salt solution itself.

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Chapter 3

Durability

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This chapter explains the importance of durable construction and provides guidance on current best practice in this area. There is an overview of the current and proposed international standards on durability and service life planning, as well as the new Standardised Method of Life Cycle Costing for Construction which is intended for British use. These are becoming essential reading as clients seek to obtain better value from the built assets they are commissioning.

There is also an overview of the need to construct, maintain and repair buildings as well as guidance on the ways in which different agents cause degradation of buildings, materials and components in the built environment. The chapter ends by looking at what happens when built assets and their components reach the end of their functional life, including the choices to reuse, recycle or put to landfill. This chapter will assist readers of other chapters understand the ageing process that affects all materials.

CONTENTS

Why durability is important

Although it is often taken for granted, the durability of the built environment is of the greatest importance. The cost and difficulty of constructing the infrastructure of roads, railways and buildings means that most societies cannot afford to recreate them every few years. Many roads still in use in Europe are on the alignment of roads first set out by the Romans. Towns and cities are positioned where they are because of the transport routes which were essential to trade and defence, and there has been a huge investment by successive administrations over the centuries in constructing towns and cities and their associated infrastructure.

If none of this was durable and so had to be replaced with the same frequency as clothes, shoes and other consumer goods, the world would be a very different place. In countries where there are few durable shelters, such as parts of Africa, Asia and South America, the advancement of civilisation in Western terms has been held back as the people need to spend more time and energy on the basics of life, including regularly rebuilding their villages.

Because most of our built environment is durable, the expenditure is effectively spread over very many years and with appropriate maintenance, life-spans of structures in excess of a hundred years are commonplace. It is therefore important to consider the durability of all forms of construction as part of the design process. As this chapter will go on to demonstrate, durability of the materials and components needs to be considered from the earliest stages of design if the intended service life of the construction is to be delivered in practice.

What is meant by 'durable'

The term 'durable' in the construction industry tends to be used to indicate that a particular material or product has a longer predicted life than its competitors. This can be misleading as a 'durable' sealant might last only 10 years before needing replacement, while the stone it is applied to could last for many centuries, but might not be classed as particularly durable.

All materials are affected by the process of ageing, with the appearance and physical properties changing over time, dependent upon the conditions they are exposed to. Some of the visible changes may not affect the functional performance and vice versa. In general, this chapter looks at functional performance as the measure of durability.

Materials that are capable of being durable can fail in a short time span if used inappropriately or if placed in locations where they are exposed to conditions that are deleterious to them. The purpose of this chapter is to provide an insight into the methods and processes by which each material or component can be made to last for as long as possible and also to avoid situations where they may adversely affect any neighbouring materials.

Because durability is important, it should be a primary consideration of any designer embarking upon a new project. The first question directed at the client should always be – how long has this got to last? Extreme durability is not normally required, but selection of the appropriate materials can only be achieved if the projected life of the construction is known. It is unnecessary and wasteful for very durable materials to be used in a built asset that has a short predicted life, unless the intention is to disassemble the asset and reuse the components again.

It is therefore important that the intended service life of any construction is agreed as part of the initial briefing process.

The design process should consider the selection of materials and components in the context of the total anticipated life of the built asset and to tie in with any predicted major maintenance activities. Materials that are used to form the foundations and structure need to last for the life of the building, but windows, doors and similar components can be replaced easily and therefore do not need to be as durable. In this way, the best value for the owner and end-user can be delivered across the life of the structure.

Standards relating to durability

British Standard Code of Practice 3 Chapter IX

This standard was issued in 1950 by the British Standards Institution (BSI) as a part of the postwar awareness that Britain had to be rebuilt after the war to a better standard than had been prevalent during the interwar years, when there was a concern that poor-quality buildings were being erected using materials that were neither robust nor durable. It set out the 'designed life of a building or its component parts or installations' so that designers knew what the anticipated life of each building type should be. This was stated as periods of less than 10 years, 10 years, 40 years and 100 years.

It also identified a range of causes of deterioration of the materials and the need to maintain buildings to achieve the best life from them. Although this was potentially a huge step forward in setting standards, the reality was that this advice was widely ignored by the construction industry and the need to achieve durable construction was not seen as a priority. The lack of interest may have been partly because the standard did not explain how to achieve the design life. It was therefore seen as aspirational rather than a practical tool to enable design of appropriately durable buildings.

From 1950 until the 1980s, the general opinion was that buildings were meant to last 60 years. This was because government funding for buildings such as council housing, hospitals and so on were based on loans being repaid over that period. Little attention was paid to the ultimate life that would be achieved and even less attention to the need to maintain and replace parts of the buildings during their life.

BS 7543: 2003 Guide to the Durability of Buildings and Building Elements Products and Components

This standard was first published in 1992 and was revised in 2003. It has not been well known or extensively used and has recently been simplified as certain aspects are now covered by parts of ISO 15686. It deals with civil engineering projects involving buildings but is only partially relevant to other civil engineering projects such as roads bridges and dams.

The core of the standard is about the need to state a required service life for a building, so that appropriate components can be selected, taking into account the conditions of use and the probable maintenance levels.

The standard provides a useful introduction to the need to consider durability and provides details in Annex A of the factors that cause deterioration of materials, so limiting their durability (see **Table 1**). A key concept is that it is a combination of factors that causes the most rapid deterioration, so an effort to control or limit contact with one agent can exert an influence on the performance of other agents of decay.

Although not widely recognised, this standard did at least provide in 1992 an informed opinion on the way to achieve durable constructions and it was a useful stepping stone to the more thorough understanding of achieving durability which is contained in ISO 15686, first published in 2000.

ISO 15686: Building and Constructed Assets – Service Life Planning

This standard is still being completed, with Parts 1 to 8 (with the exception of 4) already published and Parts 4, 9 and 10 in the process of drafting. It represents the most recent thinking on planning to achieve a particular service life for a building or constructed asset and as such needs to be considered as an integral part of selecting materials or components for a project.

The parts deal with different aspects as follows:

- Part 1: General Principles
- Part 2: Service Life Prediction Procedures
- Part 3: Performance Audits and Reviews
- Part 4: Data Structures for Service Life Planning (in draft only)
- Part 5: Life-cycle Costing
- Part 6: Procedures for Considering Environmental Impacts
- Part 7: Performance Evaluation for Feedback of Service Life Data from Practice
- Part 8: Reference Service Life and Service Life Estimation.

The following parts are in preparation:

- Part 9: Guidance on Assessment of Service Life Data
- Part 10: Levels of Functional Requirements and Levels of Serviceability – Principles, Measurement and Use.

Part 1: General Principles

This part of the standard sets out the principles that are embodied in service-life planning, including definition of

Nature	**Origin**			
	External to the building		**Internal to the building**	
	Atmosphere	**Ground**	**Occupancy**	**Design consequences**
Mechanical agents				
Gravitation	Snow loads, rain water loads	Ground pressure, water pressure	Live loads	Dead loads
Forces and imposed or restrained deformations	Ice formation pressure, thermal and moisture expansion	Subsidence, slip, heave	Handling forces, indentation	Shrinkage, creep, force and imposed deformations
Kinetic energy	Wind, hail, external impacts, sand-storm	Earthquakes	Internal impacts, wear	Water hammer
Vibration and noises	Wind, thunder, aeroplanes explosions, traffic, machinery noises	Traffic and machinery vibrations	Noise and vibration from music, dancers, domestic appliances	Services noises and vibrations
Electro-magnetic agents				
Radiation	Solar radiation, radioactive radiation	Radioactive radiation	Lamps, radioactive radiation	Radiating surface
Electricity	Lightning	Stray currents	–	Static electricity, electrical supply
Magnetism	–	–	Magnetic fields	Magnetic fields
Thermal agents	Heat, frost, thermal shock	Ground heat, frost	User emitted heat, cigarette	Heating, fire
Chemical agents				
Water and solvents	Air humidity, condensation, precipitation	Surface water, groundwater	Water sprays, condensation, detergents, alcohol	Water supply, waste water, seepage
Oxidizing agents	Oxygen, ozone, oxides of nitrogen	Positive electrochemical potentials	Disinfectant, bleach	Positive electrochemical potentials
Reducing agents	–	Sulfides	Agents of combustion, ammonia	Agents of combustion, negative electrochemical potentials
Acids	Carbonic acid, bird droppings, sulfuric acid	Carbonic acid, humic acids	Vinegar, citric acid, carbonic acid	Sulfuric acid, carbonic acid
Bases	–	Lime	Sodium hydroxide, potassium hydroxide, ammonium hydroxide	Sodium hydroxide, cement
Salts	Salty fog	Nitrates, phosphates, chlorides, sulfates	Sodium chloride	Calcium chloride, sulfates, plaster
Chemically neutral	Neutral dust	Limestone, silica	Fat, oil, ink, neutral dust	Fat, oil, neutral dust
Expansive materials	–	Steel, slag, pyritic shale	–	–
Biological agents				
Vegetable and microbial	Bacteria, seeds, moulds, fungi, algae	Bacteria, moulds, fungi, roots	Bacteria, fungi, moulds	Need to exclude moisture ventilation
Animal	Insects, birds	Rodents, termites, worms	Domestic animals	–

Note: This table is based on Table 4 of ISO 6241

Table 1 Agents relevant to building performance (reproduced with kind permission from BSI)

the different types of 'service life' relating to a component or a built asset.

It is important to understand the exact terms, as otherwise it is difficult to comprehend the differences between them. Use of the wrong term will lead to misunderstandings and so potentially to mistakes in the specification of materials and components. The most useful terms are 'reference service life', which is the default life for a building or component when exposed to standardised conditions and the 'estimated service life' which is what that building or component should achieve when exposed to the actual conditions of use and maintenance patterns on that particular site. These are then related to the 'design life', which is what the designer is seeking to achieve. The estimated service life should equal or exceed the 'design life' (but not exceed it by too much, otherwise there has been overdesign of the built asset).

There is then the concept of parts of the building or built asset that are considered to be permanent (e.g. structure,

Agents	Factor	Relevant conditions (examples)	
Agent related to the inherent quality characteristics	A	Quality of components	Manufacture, storage, transport, materials, protective coatings (factory-applied)
	B	Design level	Incorporation, sheltering by rest of structure
	C	Work execution level	Site management, level of workmanship, climatic conditions during execution of the work
Environment	D	Indoor environment	Aggressiveness of environment, ventilation, condensation
	E	Outdoor environment	Elevation of the building, microenvironment conditions, traffic emissions, weathering factors
Operation conditions	F	In-use conditions	Mechanical impact, category of users, wear and tear
	G	Maintenance level	Quality and frequency of maintenance, accessibility for maintenance

This method uses the following equation:

$$\text{ESLC} = \text{RSLC} \times \text{factor A} \times \text{factor B} \times \text{factor C} \times \text{factor D} \times \text{factor E} \times \text{factor F} \times \text{factor G}$$

where

ESLC is the estimated service life; RSLC is the reference service life of the components;

factors are as given in Table E.1.

Table 2 Agents relevant to building performance (from ISO 15686-1 Annex E Table E1) (reproduced with kind permission from BSI)

foundations) or replaceable (e.g. windows, roof finish). Parts that are replaceable should be designed to be replaced without causing damage or disruption to the permanent parts. This allows economic replacement of the relevant parts to be carried out at the appropriate intervals as the extent of the work required is minimised.

The standard goes on to offer guidance on how to carry out service-life planning during the design stage and how to forecast the estimated service life using data derived either from experience in use or from the application of the factor method. The factor method takes the various causes of deterioration in a component and combines them by applying a factor that is greater or less than one to arrive at an estimated service life. The factors are listed from A to G (see **Table 2**) and require the designer to consider whether the component is likely to be affected or exposed to a lesser or greater degree than normal. Annex F of the standard gives a worked example. This means that the quality of the component, quality of installation and exposure during use (among other things) are all taken into account. In this way, the financial and environmental costs over time of the specification, installation, use, maintenance and disposal of each major component are examined and an estimated service-life figure is provided.

The difference between LCA (life-cycle assessment) and LCC (life-cycle costing) is explained within this part of the standard. The former deals with the environmental impact of a product from ‘cradle to grave’, while the latter looks at the cost over a specific period of time. There is the potential for some tension between the two approaches, as the most economic choice may not be the one with least environmental impact.

The service-life planning process does not provide a method for calculating the risk of functional, technical or economic obsolescence of a component or built asset, but the standard advises that these risks should be considered, as a replacement may be carried out of a component that is still serviceable, but which has become obsolete. This is a particular issue with emerging sustainable technologies as further research and development in this field creates products that are more efficient than their predecessors.

The Annexes to Part 1 provide guidance on ways in which you should carry out the estimation of future costs, estimated design lives and so on.

Overall, Part 1 provides a good introduction to the concepts, techniques and processes of service life planning. It should be noted that the factor method will be taken out of Part 1 when it is next revised as the factor method is now included in Part 8.

Part 2: Service Life Prediction Procedures

This part builds upon the structure provided in Part 1 and provides guidance on how to assess components to obtain a predicted service life using different approaches and techniques including accelerated testing.

As the various agents of degradation can operate at different rates, depending on the extent of exposure, this means that the critical factor that determines the end of the service life could be different in practice from that shown in the accelerated test result. An understanding of the dose response function, where the extent of degradation for a known dose of the agent is measured, can help in predicting the rate of degradation in a particular location and exposure condition.

Where manufacturers are providing a reference service life, it is important to be able to access information on the exposure levels used in any testing associated with determining the reference service life, or the location used to provide actual data if it is related to experience in use. Without this data, it is hard to predict the estimated service life in another location with different levels of exposure.

Part 3: Performance Audits and Reviews

Because the durability of the building or built asset is critical in assembling a business case for the construction to proceed, it is important that the data used can be checked and verified. Auditing is an important management tool for ensuring that stated performance requirements are capable of being met. This means that the data used in the design process must be clearly recorded in a standardised format and be made available to the auditor or reviewer to check when required. Where the audit is being carried out as part of a bid review process, such as in private finance initiative contracts, the quality and accuracy of the life-cycle costing can win or lose the bid.

The use of audits and reviews also improves the quality of the data being used, by increasing knowledge of the actual performance achieved in use, thus providing a feedback loop that can be used to further improve performance.

Part 4: Data Structures for Service Life Planning

This has been partly drafted and is currently a technical document rather than a standard. It deals with the use of building information models and the application of industry foundation classes as defined by the International Association for Interoperability (IAI) in identifying various components and the related durability data. Once published as a standard, it will provide the means to assemble and check durability data within the building information model and so make it easier to achieve the required durability within the overall design process.

Part 5: Life-cycle Costing

This part of the standard is of crucial importance as it enables selection of the best value materials and components for the projected built asset. Best value can only be delivered where the cost of the built asset is measured against the benefits it can deliver over its whole life.

This part explains the difference between life-cycle cost, which is 'the cost of an asset or its parts throughout its life-cycle while fulfilling its performance requirements' and whole-life costing, which is described as 'the methodology for the systematic economic consideration of whole life costs and benefits over a period of analysis as defined in the agreed scope'. It also provides guidance on other aspects that should be considered, such as the social, environmental and sustainable aspects within the long-term measurements of the performance of the built asset. Intangibles such as business reputation, functional efficiency and workforce comfort can also be included in the appraisal of whole-life value.

There is guidance on the use of real, nominal and discounted costs, and the formulae to calculate these and net present value and net present cost are provided.

Because the analysis of the built asset is looking into the future, it is important to decide where the data contain any elements of uncertainty and risk and to factor this into the calculations, using Monte Carlo analysis and confidence modelling to check the sensitivity of various possible scenarios based on key assumptions.

This process is most useful when decisions are being taken on the materials to be selected for major components and finishes. Best value may be delivered by the cheapest or the most expensive component (or one in between), but this is not apparent until the calculations have been done.

Standardised Method of Life-cycle Costing for Construction Procurement

This forms a supplement to Part 5 of ISO 15686 and is published by BCIS and BSI for use in the United Kingdom as guidance on the use of Part 5. It has also been written to be able to stand alone as more accessible guidance on what is seen as best practice by experienced practitioners of life-cycle costing. As such it provides a good place to start if you wish to understand how life-cycle costing works and to discover the potential benefits of using the process as part of building and built asset procurement.

The publication of this supplement makes it much easier to compare alternative solutions using robust data, as previously it had been left open to the life-cycle assessor as to where various headings of cost were placed and what was included or excluded in each heading. This meant it was not easy to compare two or more life-cycle plans in a tendering situation as the same headings could contain different assumptions. Adherence to the standardised method will simplify such comparison exercises and so speed up the process while increasing confidence in the results.

Part 6: Procedures for Considering Environmental Impacts

This part provides a link to the ISO 14000 series which deals with environmental management and in particular with ISO 14040: 2006 on environmental management life-cycle assessment procedures.

It provides guidance on how a life-cycle assessment prepared under ISO 14040 can be integrated into a service life plan prepared under ISO 15686. This provides the facility within a financial model to introduce aspects of sustainable construction and look at the benefits that can be delivered over the lifetime use of the building or built asset. At present there are little data available on sustainable technologies that consider their full life-cycle impact, but hopefully this situation will improve as environmental impacts become part of service-life planning.

Part 7: Performance Evaluation for Feedback of Service Life Data from Practice

This part provides the methods that allow the actual performance in use to be measured, along with the impact of various levels of maintenance and usage patterns. Performance assessment in use compares the factors used to provide the estimated service life with what has actually taken place. This provides benefits to the building owner, who can amend the predicted service life of components, taking the actual condition at the time of assessment into account. This may mean that maintenance cycles can be extended, or planned replacements moved back, but it could also mean the reverse.

This information can then be used to amend the data that will be used in future projects through applying this feedback. There is clearly a need for a centralised data source so that experience of performance in use is shared and kept up to date.

Part 8: Reference Service Life and Service Life Estimation

This part deals in more detail with applying the factor method to arrive at estimated service lives from reference service lives, and this information will be removed from Part 1 when it is next reviewed and revised.

The calculations required look quite complex at first, but are actually relatively easy as the majority of factors have a reference level which represents normal usage. It should also be noted that not all components need this assessment to be carried out, and it is for the project team and the building/asset owner to decide which should be checked. The need for service life estimation is most important for those components upon which the timing of repair or replacement may be critical to the use of the building. A typical example could be an assessment of the durability of a floor in a hospital operating theatre, where replacement could mean the closure of an operating suite for a period, with the associated disruption to the running of the hospital.

Agents of degradation

All materials used in the construction of built assets, no matter how durable, have a finite serviceable life, after which they cease to perform the function expected of them by the designer or constructor. The degradation of the material is caused by exposure to various agents that cause wear and damage, but different agents affect different materials in different ways. Some may cause a slow decay in performance, while others can result in an abrupt failure. Because of this, it is important to be aware of which materials are affected by what agents and how modification of the behaviour can be achieved by the use of protective coatings, regular cleaning and effective maintenance. Where a particular agent of degradation is prevalent, a designer should consider the use of materials that are most resistant to that agent.

Water

Water is probably the most damaging agent for the majority of materials as, apart from its own properties, it can carry with it other agents of degradation such as salts, acids, alkalis and other pollutants.

Moving water alone can erode quite hard materials over time, so the management of water flows over surfaces is important if damage is to be limited. Traditional building design provided projections with grooves beneath them to throw water away from the face of the building. Rainwater is normally collected at roof level and piped to a drainage system to reduce the amount of wetting and erosion experienced by the building's exterior. Modern architecture appears to have forgotten the importance of these techniques, but it is hoped that measurement of the consequences of poor water management will enable an assessment of where better water control will bring benefits.

Reducing the period during which a material is wet extends its life, as a wet surface can support algal growth, which in turn leads to mosses colonising the surface and then larger plants. The action of the roots of larger plants within crevices and cracks opens these up, so assisting the water to reach further into the structure. By keeping surfaces as dry as possible, this process of degradation is minimised. It is noticeable that buildings can remain in a serviceable condition for a long period while the roof is sound and the walls are kept dry, but then quickly turn into a ruin as soon as water begins to reach the walls and interior through a defective roof.

Wet surfaces are also prone to damage if the water within the material freezes. This causes spalling and a breakdown in the surface, allowing more water to enter the pores and so cause further damage upon freezing. This freeze–thaw cycle can be simulated in accelerated testing and the results of such tests are used to classify certain materials for frost resistance, such as bricks.

The impact of water on durability varies with different materials. Some species of timber are naturally resistant to decay caused by moisture, and the performance of less durable timber can be improved by treatment with antifungal compounds. However, without regular maintenance of a protective coating, most timber will decay if exposed to a combination of water and air. Timber when fully immersed in water can last for a long time under the right conditions as evidenced by wrecks such as the *Vasa* and *Mary Rose*.

Although a particular timber might be classed as durable, it is important to also consider the type of fixings that are used and the proximity of other materials to the timber. Oak is durable when exposed externally, but tannins within the timber will leach out and can cause unsightly staining where water runs off. The tannic acid also attacks any ferrous fixings to create black stains; this reduces the tensile strength of the oak, so brass or another non-ferrous metal should be used to avoid premature failure in such locations.

Most of the metals commonly used in construction are adversely affected by a combination of water and air, which causes corrosion to take place. Stainless steel is normally resistant to corrosion, but can be attacked where water is present but air excluded, such as at laps in cladding. A notable case was at St Mary's Hospital on the Isle of Wight, where the stainless steel cladding had to be replaced due to corrosion damage after only six years. Two articles concerning the need for replacement and the costs incurred are available on the internet.

Structures in saltwater face particular difficulties as the presence of both water and salt in copious quantities provides a very aggressive environment for metals. An article dealing with these issues can be found on the internet.

Aluminium can be durable when exposed, as the surface oxide layer protects the metal beneath, but, if this is removed or electrolytic action takes place, then the life of exposed aluminium is significantly reduced.

Corrosion of metals can be accelerated by salts or acids carried by the water, as these accelerate the corrosion process by acting as an electrolyte or dissolving the metal surface. Chlorides can attack stainless steel and reinforcement in concrete is dependent upon an alkaline environment to prevent corrosion taking place. Corrosion has two major consequences. First, the corroded section loses strength compared to the initially installed section, giving rise to risk of failure and collapse. Second, the corrosion process adds volume to the section, and if this is restrained by being cast within concrete or built into brickwork, the expansion cracks the surrounding material, allowing water easy access to the metal and thus further accelerating the decay.

It follows that, to achieve the maximum longevity from a building or built asset, care and consideration in the capturing and effective disposal of rainwater is critically important.

Water is also present within the ground, where it can have salts, acids and alkalis dissolved in it, depending on the ground conditions. The foundations of any structure are difficult or impossible to replace, so to achieve a durable building or built asset, it is important to know the ground conditions and design any parts of the building in contact with the ground to be as resistant as possible to any deleterious substances that are present.

The third situation is where the structure contains water, such as a dam or swimming pool, or is built within water, such as port installations or bridge piers. Here the water exerts forces upon the structure that can cause movement which can give rise to cracking. It is clearly important that any allowance for movement does not permit cracking to take place, as entry of water into the structure will accelerate decay and reduce the achieved life.

Temperature changes

The majority of structures are subject to temperature variations, both short term as the sun heats exposed surfaces and longer term as the air temperature changes through the days, months and years. In general, materials expand as they are heated but by differing amounts, dependent upon the coefficient of expansion of the material and its colour. Differential movement causes stresses which can result in cracking where the stress exceeds the material's inherent strength. There are also stresses where different materials are fixed together, but have different coefficients of expansion, such as where a metal coping is fixed to a masonry wall. Temperature changes can also cause stresses when differential heating occurs, such as within insulated cladding systems where the exterior faces heat rapidly on exposure to the sun, but the substrate is kept at a steady temperature by the insulation within the cladding.

It is important in designing structures and assemblies that these movements due to temperature changes are considered and a capacity for movement is incorporated into the design solution. The thermal stresses can be as large as or larger than the structural loading and their repetitive cycling nature can cause significant damage to occur. If full allowance for movement is not provided, durability will be affected as materials and finishes are damaged, allowing water and other deleterious agents to enter the building or built asset.

Applied loads

All constructions carry load, including self-weight. To be durable, the structure has to be designed to resist all the applied loads without excessive deflections. Where structures are overloaded, damage can result, which will reduce the service life of the structure.

Some loads on the structure change due to the different characteristics of materials. Where a concrete structure has a brick infill, the concrete shrinks over time, while the brick expands. This can build up significant forces, which can result in failure of the bricks as the compressive strength is exceeded. Designers should provide soft joints to allow this interaction to take place without causing damage.

There are also instances where loads are applied that have not been considered in the design. This can occur where the building user is not aware of the limits of the existing structure, such as erecting scaffolding on a roof without checking the loads that are being applied. Most structures are designed to be adequately robust due to the requirements of the structural codes, but a designer should consider potential situations that will arise in the life of a building such as during recladding, where parts of the building that may be part of the structure need to be removed, to enable this to be done without recourse to major propping.

There is also a need to consider the demolition or deconstruction of the building as it is important that the partially demolished structure remains safe and stable during the demolition procedure.

Abrasion

One agent of degradation is abrasion, where protective coatings and surface finishes are abraded by blown sand or grit. This can be controlled by use of appropriate finishes to protect the underlying components and also by planning to repair or replace abraded areas before the damage becomes irreversible. Where the coating is performing a critical function, such as intumescent paint on steelwork, it becomes even more important to prevent loss of performance by protecting these areas.

Impact

Impacts can cause damage to buildings and built assets. Generally, the impact will not of itself reduce the life of the structure, but the consequences are often rain ingress or wind action in the damaged area that cause further more serious damage. To achieve good durability, it is important that the designer considers the risk of impacts causing damage and chooses suitably robust materials in locations where impacts could occur.

Typical impacts are those caused by vehicle movements, vandalism, unrestrained doors and windows, and flying objects in high winds. Designers who wish to achieve optimum durability should carry out a risk analysis of the likelihood of such impacts occurring and then specify appropriate materials to reduce the potential for damage to take place.

Ultraviolet light

Sunshine contains a significant proportion of ultraviolet light, which causes a breakdown in certain plastics. In areas where there is strong sunlight and at high altitudes where the atmosphere is thinner and UV levels are higher, the effect of ultraviolet light needs to be considered in the specification of exposed areas.

A number of pigments used in paints, coatings and plastics are also subject to damage by exposure to ultraviolet light, causing a chalky appearance. While the underlying material may be sound, the poor appearance can lead to the premature recoating or replacement of these components for aesthetic reasons.

Chemical agents

These comprise a wide range of substances, some of which are highly deleterious to certain building materials. Annex C of ISO 15686: Part 1 lists a number of these, including water (see above). In selecting a material to perform a certain duty, it is important to consider what substances will come into contact with it. These may be in the atmosphere, carried by water, contained within the building or within other components in close proximity.

When designing for durability, it is easiest to use a small range of materials, so that all the potential interactions between different combinations can be considered and compatibility between them can be assured. As the range of materials being used increases, so does the number of potential interfaces and also the risk of an unforeseen chemical reaction. Whilst some interactions are well documented and therefore can be avoided with care, there is an increased risk, when many different materials are present in a building, of one substance acting as a catalyst in the reaction between two other materials and so causing accelerated degradation.

Importance of specification

Achieving the required degree of durability in a construction starts with gaining a clear understanding of the client's requirements for the projected life of the building. In some cases low cost and speedy construction are required and there is no particular requirement for the completed building to be durable. In this case the client should be advised that durability has not been considered and that the anticipated life of the building is therefore unknown.

If a particular building life is required, it is also important to know what level of maintenance will be acceptable. This will then allow the specification of materials and components to be carried out to meet these requirements. As an example, it is hard to replace the floor finish in a commercial kitchen without removing all the equipment. This requires the closure of the kitchen for an extended period, which may not be acceptable to the client if its income is dependent upon it. It is therefore important that the estimated life of the kitchen floor is the same as the life of the major equipment within the kitchen, so that it is closed for a refit at agreed intervals only.

Specification for durability is best carried out within the context of ISO 15686, as this enables the process to be managed effectively. The end result can also be audited to assure the client or the funders that the building is capable of performing as planned.

Service-life planning

Service-life planning is the process described in ISO 15686. It provides detailed guidance and, when used with the Standardised Method of Measurement as produced for UK projects, it enables different potential solutions to be compared in terms of their value to the client.

Although this process is relatively new, it is anticipated that increasing numbers of clients will demand it when the benefits are known to them.

To carry out effective service-life planning, a number of factors need to be considered.

Incompatible materials

As stated under chemical agents, there are a number of materials used in construction that are mutually incompatible. Copper and aluminium are two such materials as the runoff of water from copper causes rapid corrosion of the aluminium. Bitumen and polysulfide sealants also have known incompatibilities.

There are too many known incompatibilities to mention individually in this chapter, but any specifier would be well advised to be careful in selecting materials which have little published guidance on their properties. Most of the traditional materials have a reasonable track record, and reliable information on their strengths and weaknesses is accessible from many sources. Newer materials or those that have unusual coatings or are combined with other materials in an innovative product should be treated with caution, particularly if the only data available come from one manufacturer with no independent verification.

Manufacturer's data

Where construction materials are being selected for a particular purpose, up-to-date manufacturer's information should be obtained, as formulations can and do change from time to time. There is normally a range of potentially suitable materials on offer for each use, often for widely varying prices and with subtly differing properties. Before making a selection, it is worth comparing the life-cycle cost of the options available, as it may be that a more expensive product, perhaps with a factory-applied finish, could represent better long-term value than a cheaper material that has to be painted on site.

As stated above, treat all information from the product and component manufacturers with care as certain properties may have been measured using standard tests that do not show the full picture. It is helpful to consider each material choice in the context of the use to which it is being put and the hazards that it will face in that location. Make sure that you are comparing like with like, as components sold for the same function can be made from different substances that perform differently in moisture absorption, movement under temperature change and so on.

Importance of correct construction

While the materials may have been chosen with care and consideration, this is only the start of achieving the desired durability from the building or built asset. If the construction process is not carried out with due care and to the appropriate standards, then the durability of the final building will be adversely affected.

Workmanship

With the increasing fragmentation of the construction industry, the operatives carrying out the work on site can be several contractual steps away from the main contractor. This means that drawings, specifications and working instructions can be 'lost in translation' with the result that the construction process is not as closely controlled as the designer had intended. If durable construction is to be achieved, then it must comply with good practice and this needs to be assured by putting in place appropriate contractual arrangements to ensure that the requirements of the designer are understood and properly carried out.

The contractual documents for a project need to contain clear guidance on the standards of workmanship that are required and attention needs to be given to how these will be achieved by the contactor and the subcontractors. If the operation can be carried out easily using normal skills and supervision, it is more likely to be carried out correctly. Insistence on complex details with tight tolerances and limited working space can result in poor compliance and subsequent problems when the lack of accuracy becomes apparent.

Assuring quality on site used to be the domain of a Clerk of Works, but fewer sites now have the benefit of an independent inspection by a properly experienced inspector. Such checks that are carried out are often perfunctory and focus more on percentage completion that on the quality of work or compliance with the drawings. To achieve the design life, each material or component has to be installed to a reasonably good standard – poor workmanship can result in replacement of the installed material or component before the building is handed over, which means that it has not achieved any service life at all!

Where possible, the team that will be carrying out the actual work on site should be consulted about the practicalities of each operation, as often they can offer simpler methods that will achieve a more reliable result.

Storage and handling

A number of components and materials are easily damaged during storage and handling on site, so that their service life is reduced. Where particular storage conditions are required by the manufacturer, it is important that these are complied with. Handling methods on site should be discussed with the main contractor and steps taken to minimise the number of times each component or material is handled. Where possible, 'just in time' delivery to the site should be preferred, as it reduces the risk of storage and handling damage to the minimum.

If damaged materials are installed in spite of appropriate precautions, then the achievement of the estimated service life is compromised and this should be picked up and factored into the need for an amended maintenance plan or by a reduction in the estimated service life.

Accidental damage and protection

As for storage and handling, further damage can occur on site after installation if adequate protection of the component is not provided. Any damage sustained can significantly affect the achievement of the estimated service life. Accordingly, steps should be taken to ensure that adequate protection has been priced for within the contract and that it is clear whose responsibility it is to install and maintain the protection. The timing of the removal of protection is also critically important, as there needs to be sufficient time to inspect the installation after the protection has been removed and for repairs or replacements to be carried out if damage has occurred.

While many product manufacturers can provide repair materials to hide damage, these products are generally not as durable as the original materials and it is prudent to record where repairs are carried out as these areas will probably require a different maintenance regime from the undamaged areas if the overall durability is not to be affected. Once again, the estimated life-cycle should be amended as appropriate to reflect the actual situation, rather than the predictions made during the design phase.

Importance of maintenance

Once the building or built asset is complete and handed over, most construction professionals will walk away without a backward glance. However, the building is just at the start of its life and is now in a position to deliver value to the owner and users, rather than being a drain on their resources.

If the building design process has been carried out with durability as a required attribute, then there should be a review of the quality of the building as delivered by the construction process. Post-occupancy evaluations have been carried out for some time, but are still not commonplace. The type of checks on delivered quality that should be carried out are included in Part 3 of ISO 15686 and these should enable the estimated service lives to be adjusted to reflect the quality of construction that was achieved.

The next stage following handover should be to put in hand the maintenance plan so that appropriate actions are taken at timely intervals. Without maintenance, the rate of deterioration of the building or built asset will increase. While short delays in carrying out the defined tasks can be accepted and may have little effect, continued avoidance of necessary maintenance will result in unplanned failures and premature replacement of components.

At the design stage, it should be considered whether fixed inspection and maintenance intervals are appropriate or if condition-based maintenance is to be used, where the frequency of maintenance is predicated on the condition of the item being maintained. There are strong arguments for and against both approaches, but it is better to decide during the design phase which items will come under each regime. There is otherwise a risk that neither approach will be applied effectively and certain components will therefore fall between the two systems.

Protective coatings

The category of protective coatings includes paints, seals, anodising and so on. They share the property that they are specified to protect the underlying material and can be considered in most cases as sacrificial in that they can (in most cases) be replaced by overcovering at intervals in the life of the component or material.

As with general maintenance, undue delay in recoating will increase the cost and difficulty of the operation and may result in irreversible damage to the substrate. When specifying coatings, it is important to remember that they will normally require to be replaced within the lifespan of the component and that such replacement needs to be carried out without adversely affecting the performance of the component. Problems with the operation of joinery are quite common after repainting, because there was an inadequate allowance for the increasing thickness of coating.

Water ingress

Poor maintenance can result in the entry of water to within the structure or materials of the building. As noted above, water causes deterioration in many materials and so exclusion of water is important if the estimated service life is to be reached. There is a need to check the condition of roofs and other water-controlling measures so that the risk of water penetration is kept to the minimum. Failure to carry out simple tasks, such as regularly clearing leaves from gutters, can result in serious damage to building materials and interiors.

Where water ingress would be catastrophic, for example in a computer server room, then it may be appropriate to consider the installation of secondary means of defence so that failure of the primary layer does not affect the performance of the critical operation. This then allows the planning and subsequent replacement of the primary layer without the function of the protected space being compromised. A decision on whether it is prudent to take this approach can be assessed using life-cycle costing of the combined systems and advice from the users on the probable cost of a failure without secondary protection.

Lubrication

Quite simple maintenance tasks can have far-reaching consequences on the performance of certain components. Hinges which are exposed externally, or are rarely used, are often left without any maintenance, so that they fail prematurely. Providing them with occasional lubrication means that they can reach their estimated service life. A good maintenance plan will have considered these more minor components and included the requisite maintenance in the schedule of planned operations.

Correct cleaning

Manufacturers often specify that components or finishes are cleaned at regular intervals. Again, this is often ignored, so that by the time cleaning is attempted, the dirt has stained the surface and pollutants may have degraded the finish. While the correct cleaning frequency will depend on the location of the building or built asset, this should be considered at the design stage and provision made for access to be achieved without requiring major temporary works.

When cleaning does take place, it is important that the correct cleaning agents are used and the runoff does not damage other surfaces below the area being cleaned. Some cleaning chemicals are highly corrosive and can cause severe damage if applied to the wrong types of surface.

Provision for the appropriate maintenance operations should be included in the life-cycle costs prepared during the design phase of the project. This information needs to be transmitted to (and understood by) the owner and users of the building or built asset and the responsibility for carrying out the required maintenance clearly assigned to the appropriate staff.

If the life-cycle costing has been carried out correctly, the owner or tenant of the building should be aware of the maintenance requirements before they take over the building and the appropriate budgets should be available throughout the life of the building or built asset. As noted above, failure to maintain and clean the building will in time result in higher bills and reduced functionality, thus making short-term savings a poor option.

Importance of repair

Early repair

The need to carry out repairs to installed components and materials needs to be monitored, as a series of individual minor repairs may be an indication that there is a common defect or that usage is more severe than had been anticipated. It is possible that prompt intervention can reduce or prevent further defects. If, for example, doors are being damaged by being thrust against the end of the hinge travel, installation of a limiting stay could prevent this happening.

All installed components are prone to a series of early failures, generally known as ‘burn-in failures’, where a manufacturing flaw in a proportion of the installation causes a few defects to occur. As above, these need to be investigated and monitored. If they are burn-in failures then the incidence will drop with time.

Correct materials

When repairs are carried out to installed components and materials there is a tendency to do the minimum required to make the repair. This can mean using a nail instead of a hinge pin, or tape to cover a split in vinyl flooring. These poor repairs can result in further failures, but these would be due to the repair failing rather than the component.

A good maintenance plan will include information on the manufacturer and model of all components, so that repairs can be carried out by replacing like with like. Exceptions should be made when it is clear that the installed component is not robust enough for the duty cycle that it is experiencing. Here it is important to involve the original designer and manufacturer, as they need to know of the problem if future shortfalls are to be avoided. Their advice should be sought on the replacement component, because a stronger component will exert greater load transfer to its fixings and surrounding materials, so one failure mode may be replaced with another.

Workmanship

As for the construction phase, good workmanship in carrying out repairs is equally important. Much maintenance work is carried out by multi-skilled individuals, but it is important that they receive training in the correct methods of dismantling, repairing and installing the various components. There can be a tendency to replace rivets with screws, or use the tape or sealant that is to hand, rather than the correct material. This can lead to continuing minor failures which are an irritation and reduce confidence in the durability of the affected components. It is often better to spend more on a thorough repair or replace the affected unit.

Understanding of failure mode

When a component fails, it is important to consider the reasons for that failure. The correct and early diagnosis of a failure can allow changes to be made to maintenance or usage regimes so that further failures can be avoided. Most industries are much further advanced than construction in failure mode effect analysis (FMEA). This process looks at the failures that occur and analyses what caused the failure. This allows changes to be made in the design and fabrication so that the incidence of that particular failure is reduced or takes place after more duty cycles.

This approach to failure needs to be introduced to construction failures so that the industry understands the causes of premature failures and is able to respond by producing an improved product. ISO 15686 Parts 3 and 7 go some way towards this but the key issue is that the industry as a whole needs to be much more open about failures when they occur and to share understanding on the causes and remedies.

End-of-life issues

Ease of replacement

As various components and materials reach the end of their service life, the replacements planned for during the design stage need to be carried out. If the design process has been thorough, the replacement should be straightforward, as the fixings will be accessible and the perimeter seals or overlaps will have been designed to permit the easy removal and replacement of the time-expired item.

It is also possible that some components may have failed prematurely, or a defect has led to a component not designed for replacement to need repair or replacement. These problems should be recorded and fed back to the designers, manufacturers and constructors so that this information can be used to reduce the instances where this will recur.

As the industry has not been accustomed to designing components for replacement, the current cost of replacing components is far greater than it should be since adjacent finishes and components are often damaged during the replacement operation. Provision for easy access has also not generally been considered, requiring a significant investment in temporary works.

Designing for replacement means considering exactly what package of works will be required and making provision to minimise collateral damage.

Damage to surrounding materials

As noted above, all replacements involve an interface with adjacent components and materials. Use of cover trims and push in/pull out seals can enable the time-expired component to be removed and replaced with no damage to the surroundings. These provisions need to form part of the original design and the additional cost over, for example, plastering up to the interface has to be justified in life-cycle cost terms. These decisions need to be expressed in the maintenance manual so that when the time comes for the replacement to take place, the decision tree can be revisited and the perimeter detail either replaced as originally planned or changed to reflect greater knowledge of the costs and difficulties of the replacement when it occurs.

Access provision

As noted above, the need for component replacement, where the estimated service life is less than that of the complete building, has to be considered in developing the original design. This should include making provision for ease of access, such as installing screws in eyes to restrain access equipment, as this costs little at the time of construction and enables any replacement operation to take place more quickly and safely.

The size and weight of components to be replaced have to be considered, as larger items may need mechanical handling to allow safe removal and replacement.

The Construction (Design and Management) Act requires designers to consider the risks of cleaning and maintenance operations and to ensure that any risk inherent in these operations is minimised.

Disposal issues

Once the time-expired components have been removed, a decision will be required on what is done with them. Disposal to land fill is an expensive option and may not be available in future, so the two main options are reuse or recycling.

Reuse

Reuse is where the removed components are put back into use with minimal remanufacturing. Reuse is generally achieved where components can be repaired or refurbished to restore them to a usable condition. Because less energy is required, it is generally more economical to reuse components rather than recycle them.

Some materials can be cut down to smaller sizes to permit reuse with minimal cost, such as timber from demolition which can be planed down to smaller sizes. Other components can be reused after parts such as seals have been replaced with new. Reuse may also be achieved by changing the use to which the component is being put, such as incorporating glass bottles on their side to provide an insulated wall that allows light to enter the building while maintaining privacy. By identifying potential for reuse at the end of the service life, the value delivered by that component can be increased, as it can have a positive value when removed.

Recycling

Recycling of removed components and materials means that they either return to the raw material chain by being shredded or smelted down or alternatively they are used only after a significant manufacturing input. A typical example is the conversion of concrete from demolition into aggregate for new concrete.

The value of materials for recycling can be affected by the use of coatings and treatments applied for the original use. Plain glass is comparatively easy to recycle, but tinted glass with an applied surface film which is part of a double-glazed unit is much less easy to prepare for melting down.

As with reuse, early identification of the recycling potential can benefit the value of the component, particularly if the component is designed so that the recycling value can be enhanced. European and American legislation is requiring an increasing number of manufacturers to consider the end-of-use situation for their products. This can mean that a product is made of only one type of plastic or that the different plastics can easily be clicked apart, as this makes it easier to shred and recycle the individual parts.

Conclusion

Durability can only be achieved in construction by the combined efforts of the designer, constructor, maintainer and client. Where life-cycle costing is carried out to provide an assessment of the whole-life value that can be delivered by the building or built asset, the benefit that can be delivered by working to the plan is understood and it is easier for each of the parties to play its part. Without consideration of durability as part of service-life planning, it is highly probable that the end result will not perform as well as it might and that defects in construction and premature failure of components will be more likely.

Because achieving appropriate durability is important, it is critical that it is put high on the agenda from the beginning of the building procurement process and that the results are measured and reported back to all parties. In this way the virtuous circle of continuous improvement can be created and the construction process can start to move towards the best practice used in other manufacturing environments.

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Chapter 4

The economics of materials supply

John Short Steel Derivatives, Dubai

CONTENTS

Almost all finished structures can be achieved by more than one approach and most approaches will have the potential to feature different components and mixes of those components. Every location throws up its own idiosyncrasies with respect to surety of quality, timely availability and cost, and almost every project's macro- and micro-economic environments, from government imposed trade barriers to local problems such as suppliers going out of business, will demand changes to theorised procurement function. Decision tree analysis may reveal to the civil engineer an unsettling degree of complexity and risk. It is likely, however, that even the most well-researched analysis of this type will not reveal the full extent of the complexity and risk, nor necessarily provide any meaningful insights into how to manage them.

Introduction

The economics of materials supply is a vast subject and could provide enough content for an entire series of books. In order to tackle the subject in a relatively brief chapter such as this, therefore, the economics of materials supply is addressed by focusing broadly on one of the more important items, namely steel. Steel has been chosen for the following three reasons:

- Steel is one of the higher price items and important to get right; if there are problems with the supply during construction, it can lead to other, greater problems.
- There are other chapters on steel in this book to which the engineer can refer.
- By taking steel as a representative material, the reader is invited to examine the economics of materials supply across the breadth of inputs required on an engineering project, and thereby appreciate the challenges of each supply chain, consider how to minimise the risks at the outset, and think about how to approach or overcome the issues that will inevitably arise from time to time.

Through this focus the chapter aims to illustrate to the civil engineer the challenges to understanding the complexity and risk involved when choosing any materials for a project, and how to ensure enough redundancy and contingency in the chosen approach to accommodate both foreseen and unforeseen changes to the economics of materials supply.

The chapter begins with a brief look at areas for consideration when discussing how a particular structure is going to be built, before focusing on the economics of supply of the steel component parts of the project once the high-level decision (e.g. whether to use a steel frame or concrete reinforcement) has been made. At each stage the chapter will demonstrate the importance of identifying and mitigating the risks involved (e.g. price risk, product risk, delivery risk, inventory risk), in order to equip the civil engineer with the skills required to undertake similarly detailed analysis for any material inputs to a project.

Economics of materials: steel frame vs rebar

At the macro level, the key consideration is steel frame vs concrete reinforcement.

Steel-frame buildings are usually constructed from a combination of hot-rolled steel sections and/or pre-engineered steel sections.

There are three primary families of hot-rolled steel section, defined by their shape:

- European, including IPE/IPN, HEA/HEB, and UPE/UPN, within which there are further sub-categories of sizes. For example, IPE 100s come in three sizes – IPE, IPEA and IPEAA. The same goes for HEAs.
- Imperials, which have two dominant families: UK imperials, commonly known as UB/UC, and US imperials, commonly known as W-sections. They are similar, but not identical. Each section size also comes in a number of sub-sizes, commonly referred to by their kg/m or lb/ft figures (foot weights).
- Japanese, commonly referred to as JIS sections.

Pre-engineered sections tend to be built from hot-rolled steel sheet (decoiled flat steel) and/or hot-rolled steel plate (flat steel derived directly from slab; not previously coiled). It is quite possible to have a steel plate and a steel sheet in exactly the same dimensions and made from exactly the same grade of steel, and even produced by the same producing mill, which nevertheless behave differently in certain applications. It is important to consider, therefore, whether the project in question will be sensitive to this difference.

Reinforced concrete

Production of reinforcing bar (rebar) is more or less standard throughout the world, so the physical shape of the bar varies little according to region.

Much of the following section is devoted to availability, since the first thing to consider before settling on and accepting a design is the consistent availability and surety of the specified product, as well as suitable contingencies.

1 Availability of the product:
- dimensions:
  - shape
  - size (foot weights)
  - lengths
- tolerances.

2 Availability of grade, and certification of grade:
- availability of volume
- availability of pre-fabrication
- surety of delivery
- surety of price.

Availability of the product

1 Dimensions.
- Shape: how consistently can a specified size (e.g. UB 533 × 197) be secured? Is the guarantee presented by the chosen supplier reliable? How does the supplier propose to guarantee it? Does the supplier produce the materials themselves, and how many times a year. (All rolling programmes require a minimum number of tonnes to justify them. It is likely that your order alone will not be enough to justify such a program)
- Size: 533 × 197 comes in three different combinations. Are all the ones needed consistently available?
- Lengths: if required, are 'outsize' lengths readily available?

- Dimension tolerances.
  - The rolling tolerances of the various standards used are often employed differently by producing mills around the world. This is mostly due to commercial reasons, e.g. payments made on theoretical weight vs gross weight vs net weight vs actual weight. What tolerances are applied to the product in your market? Do your drawings reflect this? Can you live with sections with different tolerances, should only those become available in a few months time?

Availability of grade/quality

1 Grade.
- Commercial quality steels in standard sizes tend to be available everywhere, as long as the money is there to pay for them. The same cannot be said for high-tensile varieties, or those with particular properties, such as steel with high chromium content for corrosion resistance and ductility.

2 Origin.
- There are some very good steel mills in China, making state-of-the-art steels. However, there are also some very bad steel mills in China producing steels of less than adequate quality. The same is true of almost every country with a steel production industry. Origin is, therefore, no determinant of quality. Indeed, many of the larger steel companies that have emerged in the last decade have several production lines in several countries, ostensibly capable of making the same product. Despite this the product from each line is rarely of the same quality. Producer name is also, therefore, no guarantee of steel quality. Generally the most important factor that influences decisions regarding suppliers tends to be previous knowledge of the producing plant's capabilities and product. Close attention should also be paid to certification, traceability and protection.

3 Certification.
- Even if the product is traceable to the producing mill, you should consider whether the mill actually produced the steel, or whether it merely converted steel made somewhere else. It is important to be sure you know what you are buying.
- The consultant might demand rebar from CARES-approved mills only, or EN 10204: 3.1b certs on all procured steel. How will you manage if only 2.2 certs – or no certs – are available?
- Do you have access to retesting laboratories with suitable equipment which offers an acceptable turnaround time?
- Can you place a third-party inspector at the mill to check your steel as it goes through production and dispatch?

4 Protection.
- Will the supplier duly protect steel that is perhaps more perishable or prone to damage? In the Middle East, the chloride-laden atmosphere encourages the development of rust in steel with concentrations of chloride contamination that are unacceptably high for immersion into concrete. You should consider whether the supplier is protecting the rebar suitably.
- Epoxy-coated rebar is widely accepted as a suitable protective coating, but this protection is only effective if the rebar's epoxy coating is completely intact when the steel is immersed in concrete. Even if the epoxy-coating plant delivers a pin-hole/holiday-free coating, most transit/processing/site handling will induce gaps in the coating, which can lead to major problems down the line.

5 Packing.
- will the lighter section and rebar be suitably packed so that it remains straight, or indeed, in the processed dimensions to which it was ordered when it eventually arrives at the point of construction?

Availability of volume

Here it is necessary to consider a (chosen) supplier's capacity to deliver, in all circumstances. Can they deliver:

- the contractual volumes of each shape and size of section/rebar in the stipulated grade?
- the necessary volumes in the contracted time?

Most frequently the supplier will not have direct control over their supplies. Rather they will act as go-between – or will be dependent on at least one or more such go-betweens – in a complex supply chain usually originating before the producing mill.

Consider the traditional rebar supply chain:

1. scrap collector
2. scrap trader
3. EAF (electric-arc furnace) mill, converting scrap to billet
4. billet trader
5. billet re-roller, converting billet to rebar
6. local stockist, possibly with their own cut and bend processing operation
7. contractor
8. developer.

There are up to seven possible parties within this chain taking title to the goods, and taking price risk too. Internationally, on average, rebar trades 4.5 times in the supply chain, while sections (beams/joists/columns, etc.) trade 3.5 times.

Most local stockists' business model is to purchase stock based on predictions of what they think the market will need in three to six months time, at prices that make those future sales profitable. Rarely do they know exactly what the market will need, and rarely will they know the market price even for the following week, let alone as far ahead as three to six months. It is almost certain that they will run out of particular sizes, shapes and grades from time to time. With the lead time on most locally produced steels of two to six weeks, and of imported steels two to five months, delivery of the required volume at the required time is uncertain at best. It is important to have contingency suppliers and/or to be able to use alternative dimension steels as the circumstances dictate.

Availability of prefabrication

Many jobs will out-source prefabrication of steel prior to its arrival on site. Off-site prefabrication may be undertaken by the contractor, by some joint-venture (jv) or a third party. Consider, though, whether that supply is assured.

The product may be purchased by the contractor/developer and then free-issued to the fabricator, the service fee, therefore, essentially being one of tolling. It is arguably a safer set-up than having the fabricator purchase on your behalf, as you then become reliant on their procurement processes and cannot ensure that delivery will be taken in a manner that ensures the timely supply of prefabricated product to site.

Free-issuing is not without its pitfalls. Often the fabricator will have a strong relationship with a string of suppliers, ensuring that they can get a product when perhaps you cannot.

In considering these issues, the following questions should also be posed: will the fabricator be in business throughout the project? Do you have a contingency?

Can the prefabricated pieces be transported to the site easily, within budget and on time? Will large pieces fit under bridges? Is a police escort necessary? Will long pieces get around bends in the road from the fabrication yard to the site?

Surety of delivery and surety of price

As mentioned above, the steel supply chain is a complex one. Late delivery ex-mill is endemic, and rarely compensated.

The fact is, however, that there is very rarely an actual shortage of any steel product. What is much more common is that certain steels are not readily available at a price a buyer somewhere along the supply chain is prepared to pay. The surety of delivery is inextricably linked to surety of price.

Is it possible, therefore, to ensure delivery at the budgeted price? More often than not, the answer is no. This is because there are too many links in the supply chain between which price certainty cannot be maintained over extended periods. This phenomenon largely arises due to the lack of tools or contractual arrangements required to overcome the conflicts of interests and differences in business models between the buyers and sellers in the supply chain.

The project will assume a fixed price for 12–24 months. Yet:

- Scrap prices vary by the hour; a scrap trader is usually long or short of scrap and gambles according to their market view.
- The EAF mill tends to be long in scrap; it is obliged to be so if the plant is to continue working. The question is whether it is one week, one month, or one quarter long. This will depend on the mills owner's view of the market. It may seem prudent to keep certain volumes of finished product in ready stock, but almost all steel is made to order, and mill inventories of finished steels are generally kept as low as possible.
- The billet trader also tends to be long or short and may try to back-to-back a deal. However, even back-to-back deals will mean the trader is exposed for at least half a day, and sometimes as long as two or three days. Often a long or short position will be adopted according to assumptions made regarding market conditions. Billet prices vary by the hour.
- As with EAF mills, stockists also tend to be structurally long of steel in stock and on order. The question is whether they consistently hold 'enough' volume of all shapes, sizes and grades. Market prices available to the stockist for finished steels tend not to be volatile intraday, but they can vary from day to day or week to week. The decision as to when to buy depends not only on the tonnage the stockist expects to be able to move in three to six months time, but on the price it is expected to fetch. It is unlikely that you will be the only party being supplied.

- Consider now whether the fixed price that has been agreed with the supplier is going to be sufficiently attractive, in all market conditions that may occur during the supply contract, to ensure good supplier performance.
- Are the prices of the supplier's other customers higher or lower than your agreed price? When the going gets tough, suppliers are more likely to supply the higher paying customer first.
- Has the supplier already paid, or agreed to pay, prices that in turn ensure performance of their supplier? Your supplier may have a fixed price contract signed with their supplier, but if the market has moved too much against their supplier, it is possible that the contract will not be fulfilled at the agreed contract price. On the other hand, the supplier may be gambling that they will be able to purchase your product cost-effectively some time hence. If this gamble does not pay off there could be serious ramifications.

Each floating price market (for scrap, billet, rebar, iron ore, and section) is less well correlated than the supply-chain participants would ideally like, with 90-day volatilities well over 20% (in 2008 this was more than 50%), and there are basically no derivatives with which to mitigate steel price risk in the same way risk can be mitigated for currency fluctuation, or even price fluctuations for other metals (copper, aluminium, zinc, gold, etc.). The steel supply chain is rife with price risk; this along with price volatility and the resulting absolute steel prices creates the conditions for delivery risk.

A long-term fixed-price contract is probably only effective if price volatility *along the supply chain* is limited, ensuring that all commitments between the various parties are honoured in a timely fashion. The final contract in the supply chain – between the stockist and contractor – is probably the most strained there is, as this is where all the floating price risk must be converted into fixed prices. Contracted price escalation clauses only help to deal with the problem at times of low price volatility. Contract price-renegotiation is endemic in the steel industry, from lowly players such as the local scrap yard right up to the big-hitting suppliers like Arcelor Mittal. This is because the methods by which price risk is managed are not as well established in the steel supply chain as they are in other metal supply chains.

Availability of logistics

For jobs which are dependent on imported steels, a small but important consideration is the cost of transporting the materials. Due to fluctuations in price, it is not always easy to keep this within budget. By way of an example, in 2008 international freight rates quadrupled, and *force majeure* was claimed by many international importers as they and their suppliers were simply unable to find vessels at a price they were comfortable paying.

Trade barriers are another critical aspect of logistics, delivery and price surety. These can be imposed by national governments (e.g. the US) and supra-national bodies (e.g. the EU) on certain types of steel (defined usually by HS codes) from certain origins. There are numerous variations, but a few of the most common are:

- export taxes and/or anti-dumping duties
- import tariffs and/or quotas
- availability of export and/or import licences.

Should your supply chain involve international transport, and thereby import–export, it is important to be aware of the problems that trade barriers can pose to your logistics, sometimes making it impossible – or at least very expensive – to get what you ordered.

Availability of finance

Trade finance is a key element of steel supply chains, and its cost and availability have much to do with what types of steel are available when, and at what price. Traditional trade flows can be switched off overnight should financing dry up (e.g. due to political reasons).

Financing lines may become limited to your supplier through no fault of their own, making it impossible for them to pay for the steel which they intend to supply.

Factoring and credit insurance can similarly determine whether the local supplier is able to procure steels.

Managing the economics of material supply

Having looked at some of the things that can go wrong in steel supply, the following section focuses on some of the methods that can be employed to build in sufficient contingency/redundancy with regard to steel supply, in order to ensure that the project does not suffer unduly from things beyond your control, and, as is often the case, beyond the control of your selected supplier(s). This is a key point; simply having a contractual obligation with the supplier will rarely be enough, since litigation is unlikely to solve any problems in a time- and cost-effective manner. An essential element in managing the economics of material supply is a partnership understanding in which contractual parties are aware of more than just the key drivers for each other's businesses and are prepared to work together to overcome the issues that will surely arise over the lifetime of a project.

1 Availability of product.
  - dimensions:
    - shape
    - size (foot weights)
    - lengths
  - tolerances.

2 Availability of grade, and certification of grade.
3 Availability of volume.
4 Availability of prefabrication.
5 Surety of delivery.
6 Surety of price.

Availability of product

Dimensions

As mentioned in the Introduction to this chapter, the engineer is obliged to make an assessment on the availability of the chosen materials, and primary contingencies. So, for example, if the structure is to be built largely from hot-rolled sections supplied from a local (regional) stockholder, the engineer should be satisfied that:

- The stockholder has more than one source for each of the section sizes. If this is not the case (perhaps the chosen supplier is the exclusive agent for a particular steel mill's section products) then the question arises as to how the stockholder can guarantee timely delivery of the agreed sizes at the agreed price should their (favoured) supplier not perform. It should be noted that the realisation a favoured supplier is going to be unable to perform is unlikely to happen in time for the stockholder to order fresh production from an alternate source mill. It is, therefore, prudent to ensure wherever possible that the sizes you require are commonly available in the local/regional market. This gives both you and your supplier greater options and contingencies to go elsewhere for product should the necessity arise.
- The required dimensions are frequently available from more than one producing mill, and not subject to minimum order volumes or irregular mill rolling programmes.

In most parts of the world you will not have access to both imperial and metric sizes of rebar, nor is there likely to be an equal availability of more than one series of sections (e.g. Japanese or European).

It is worth noting that the common commercial rule of thumb – that the advantage of using the more common product is that competition tends to drive down the price of that product – is far from assured when it comes to steel. Dominant suppliers tend to operate on the assumption that their clients cannot afford to take the risk of relying on a less common product in case of supply chain disruptions. It may seem counter-intuitive, but it is generally the more ubiquitously available product that is the most expensive. Engineers can think of this 'extra' cost as an insurance premium; a price worth paying for the reassurance that chosen product sizes will be available locally most or all of the time.

Grade

It is rare to find commonly available sizes in anything other than so-called commercial grades or qualities. If the grades you require are different from what is commonly available, it is best to have a contingency design for every line item, which can be employed in the event of a local shortage of the specified non-commercial grade. This should make use of the market's commonly available commercial grades and be in line with the above product dimensions.

An alternative approach often used is to establish a direct relationship with one or more producers of both commercially available grades and those grades less frequently available 'off the shelf'. The purpose of the relationship is not to buy direct – although this could be one development – but primarily so both sides fully appreciate the importance of timely delivery (particularly with regard to the less commercially available product specifications). This approach can work well, but equally, can also fail drastically, for example when:

- the local supplier feels uncomfortable with their customer making direct contact with the producing mill, creating unease or mistrust between the supplier and contractor;
- the producing mill realises the engineer's lack of alternatives and uses this as leverage to solicit more favourable commercial terms – now or by later renegotiation – either directly or through the local supplier.

Origins and national standards

Grade complications can also arise when material lists have been developed exclusively with reference to a single national standard, such as British (BS), German (DIN) or American (ASTM or AISI or SAE) standards, but the only product available at a particular time is certified to a different standard. Within Europe the homogenisation of BS, DIN *et al.* into the EuroNorm (EN) standards has, to some extent, resolved these issues, but it has far from eradicated their occurrence altogether. Furthermore, outside Europe, engineers often face repeated challenges if the local stockholder's inventory of a specific line item has, for example, initially been sourced from Korea, but during the project's lifetime is also sourced from South Africa, Brazil, the US, Ukraine and India, all of which have their own standards.

Certification and origin

As touched on in the Introduction, certification can be a minefield. Despite the producer and origin claims on Mill Test Certificates (mtc's), in many cases the steel itself may have originated as far as a continent away. More than 100 million metric tonnes per annum of (i) round billet (for seamless tubes), (ii) square billet (for all long steel products such as rebar, rod and sections) and (iii) slab (for all flat products, such as plate, sheet and coil) are shipped from producing mills to 're-rollers', where these 'semis' products are merely processed into finished steel shapes. The re-roller's mtc very rarely indicates where the steel was actually made. Indeed, with so much rebar and section made from scrap, the reality is that such steels are most likely to be a blend of steels produced several years previously in a variety of different countries.

Certification levels

If you go into the detail of the EN 10204 standard, you will find that, with most commercial steel grades, the specific piece of steel for which you have an mtc was never itself tested, either for its quality or indeed its dimensions.

Whatever the production routing, all producers ‘blend’ their ingredients to deliver steels within certain specification ranges. At the base level, their mtc’s reflect this blend. In essence, the mtc merely reports that the steel is within the stated range, which is itself permitted under the stated specification. This is as much as can be expected for lower-end product from lower-end producers.

At the next level up, the producer actually has to sample the chemistry of each ‘heat’ of steel. That sample, taken from the 60–250 mt bucket of molten metal, is clearly only representative of the achieved chemistry. With so much production a result of continuous casting, there is undoubtedly a blending that occurs between the steels of each successive heat. The mtc is a good guide to the chemistry of the rebar or section, but not much more.

When it comes to recorded mechanical properties (e.g. tensile, yield, elongation, hardness), mills again have a variety of options concerning levels of testing. At the bottom end ‘typical values’ of the steel produced will be indicated, whereas at the top end every piece of steel is tested individually.

The same is true of dimensions; the mill will set its equipment up to roll product to within a certain range allowed under the standard to which it is being certified. The mtc will tend to record the nominal dimensions only. It is highly unlikely any single piece of steel will exactly match the dimensions stated on the mtc.

The engineer should make a point of understanding the nature of the testing authority, the equipment used, and its maintenance/recalibration regime. For example, if the mill itself is the testing authority, there is a question as to how diligent it is likely to be in its testing regime and maintenance programme. Few mills can afford higher rejection rates of overzealous inspectors. Keeping things in-house – having the inspectors on your payroll – might just lower the rejection rate to an acceptable number.

Most, but by no means all mills create a totally separate company for their inspection services, and ensure their maintenance and recalibration programmes conform to the approved norms. It does not quite constitute outsourcing, but the quasi-independence of such facilities tends to be accepted by most buyers for most steels.

Every buyer is entitled, of course, to demand third-party inspections, which are usually performed on the mill’s equipment, or otherwise on samples taken away to independent laboratories. It is obviously important to be vigilant in assessing the integrity of the chosen surveyor, as well as their ability to recognise whether or not the steel sample they have been given is in fact from the piece of steel you want testing. This is not easy to achieve, and the engineer is going to have to trust a great many people along the way.

Ultimately it is up to the engineer to determine what level of testing is required for the steel being used in the project. They must consider whether the commercial cost (in money and time) of top-level, detailed independent testing is really appropriate for all the steels in question, and whether the resulting paperwork will give an accurate indication of the integrity of each rebar or section.

Packing and protection

Strict conformity to minimum standards of packing and protection can do much to ensure the physical integrity of delivered steels. Mills accustomed to international export tend to ensure their product is appropriately packed for the journey to their local port. Where they frequently make errors is in their assessment of the equipment and conditions at the port of discharge, as well as the subsequent transit and handling of the steel in the destination country. These are factors not necessarily known to the producing mill. To this end it is incumbent on each buyer in the supply chain to ensure the packing and protection is sufficient for the multiple handling and transit prior to arrival with the ultimate consumer. This is often not the case, particularly with lighter weight/strength steels which are prone to damage.

Even reputable mills make out-of-specification/junk steel, while mills of a lower standing sometimes produce first-class steels. Both frequently provide less than adequate packing. The better mills will diligently ensure all non-conforming steel is never allowed out of its premises labelled as ‘prime’ steel, but rather set aside to be graded and sold in a wide variety of sub-prime categories. Commercial pressures or human error can, however, combine to cause problems to parties later in the supply chain. The scope and frequency of claims in the industry suggest that this is a far from rare occurrence.

In practical terms, the most important steps to be taken with regard to origin, certification, etc. are to clearly document what is required from the outset, but also to provide the local supplier with an acceptable degree of source flexibility in order to meet your needs. For example, the engineer could consider the following:

- Supplier lists should not be specific as to the country of origin. Rather, if employed, they should be producing mill-specific, with enough options for the local supplier to pursue depending on market circumstances. Relying on one mill is a recipe for disaster.

- Supplier lists detailing grades as defined by one national standard would do well to list alternative grades as defined by other national standards (with specific amendments if so required), giving the local supplier an acceptable degree of source flexibility to meet your needs.
- Insert a clause enabling you to fund spot checks by appointed third-party agencies at the producing mill, for quality, certification and packing integrity.
- Be accommodating of dual/triple/quadruple certified product. This tends to cost the local stock holder extra, but is sometimes a lifeline which enables them to deliver the same product into the various projects within their jurisdiction that have been specified in different standards.
- Establish a minimum level of certification, perhaps using EN 10204 or its equivalent (nearly every standard's authority issues such a standard). Then engage the supplier in the appropriate courses of remedial action, such as local testing of samples taken from representative pieces at local independent laboratories, should they or their suppliers be unable to come up with mtc's meeting the requisite level of testing/information/independence. Ensure that the certification levels enable full traceability back to the producing mill, and not just to the middle men.
- Establish a minimum level of packing and protection, and perhaps insist on taking delivery of product with the original mill packing still intact to discourage the local supplier from repacking loose or broken bundles, which provides scope for the inclusion of odd pieces of non-conforming steel.
- Recognise that stock-holders themselves will rarely undertake on-site physical or laboratory inspections of their inventory. They are, after all, just middle men passing on product made by others, so you should ensure that you have the resources to impose spot checks on your product at the supplier's facility or at your job site (preferably the former).
- Ensure that those in charge of receiving physical product diligently check conformity and traceability between the supplied steel and accompanying mtc's, perhaps initiating a protocol whereby nothing gets accepted on site without due sign-off of this conformity.
- Rotate the personnel and shift patterns of those in charge of accepting product.

Availability of volume

In the Introduction to this chapter I stated that commercial quality steels in standard sizes tend to be available everywhere, as long as the money is there to pay for them. While this is the case for the most part, the reality is that along the supply chain certain parties will not be prepared to pay enough, and this is when volume shortfalls tend to occur.

Rarely do regional markets run totally dry of product, as there is a whole sector of industry traders who make a living by spotting market shortfalls earlier than most and intelligently arbitrating that supply and demand (and therefore price) imbalance between national or regional steel markets. True non-availability is limited to those products which are (i) only capable of being produced by a few mills around the world (e.g. heavy plate of >5000 mm in width in a sour service, ultra-low sulphur, HIC tested grade), and (ii) unsuited to air-freight (a 5000 × 12 000 mm plate weighing 18 mt cannot be feasibly transported on an average cargo plane). As mentioned before, in general however, the engineer is unlikely to be faced with a genuine non-availability of product. What is more common is that the product will not be available at the price you, or someone further up the chain, is prepared to pay.

I have also discussed previously the gamble most stockholders take when ordering their inventory 3–6 months in advance of receipt, and how it is inevitable that they will occasionally run out of certain sizes or grades, particularly when market prices are falling, recession is looming, or financing is proving to be expensive or in short supply. It is also the case that the stockholder may 'have a run' on some items due to variable, unpredictable market demand, and simply sell out of certain items before stock replenishment occurs. It is evident that local markets (e.g. the south of England) as opposed to single suppliers can run out of certain items. But such events tend to be resolved provided that the sizes or grades are commonly available in the regional market or are made locally, and that one party (the engineer or the local stockholder) is under contractual obligation to pay higher prices to purchase the product ex-stock elsewhere.

It does not help the stockist that mills have rolling programmes demanding minimum quantities before certain sizes or grades will be produced. For example, the producing mill may demand 1200 mt to roll a particular section size, or a minimum of two heats to make a particular grade, and if it does not get orders for those volumes, production/rolling will be delayed. As such, published rolling programmes should be considered primarily as guides, and certainly not as guarantees of production.

There are also problems with international shipping, when certain minimum volumes are required to induce a vessel to call into specific load and discharge ports. If those volumes are not met, then it can be commercially unviable for the shipper to call into those ports. The steel may be ready, but stuck at load port until more tonnage is accumulated, or someone puts enough money on the table to induce the vessel to make the call.

The wisdom of providing design scope to accommodate steels of different dimensions and grade to accommodate availability shortfalls in particular steels during a contract's lifetime has already been discussed earlier in this chapter. Providing for contingency in suppliers is founded on similar logic, and on occasion is inextricably linked to the availability of product. However, providing for contingency in suppliers exacts different demands of the engineer.

It is not unusual, for example, to find suppliers tied into exclusive representation/agency arrangements with source mills; arrangements which also severely limit the relevant mill's capacity to supply its products to the local agent's competitors. Thus, identifying alternative suppliers of the same product specification is not always straightforward.

When the same products are available from a number of suppliers, as is often the case with reinforcing bar, suppliers will frequently seek to tie the buyer into 'all-or-nothing' arrangements, thereby limiting the capacity of the buyer to source from competitors in even the most demanding of circumstances. Often in conjunction with this, the local stockholder may have exclusive arrangements with processors (e.g. cut and bend) or fixing companies. This 'full service' offering is again used to tie projects up with a single local supplier or processor. Should anything untoward happen with one or more companies in this 'closed' relationship, competing suppliers are not always ready to come to the engineer's rescue for fear of retribution in the future. When the timely delivery of required volume is jeopardised, engineers all too often find that they are helpless and unable to get out of supply commitments with their chosen supplier, even though delays and penalties will inevitably ensue.

The key points about volume:

- Ensure that your contractual arrangements provide a clear list of circumstances under which your chosen supplier is obliged and/or you are able to source outside the usual channels, with clear responsibility divisions on costs.
- Be acutely aware that your local supplier will 90% of the time merely be a middle man, and not actually in as much control of the inventory as you may be led to believe. You should, therefore, bear in mind that if the supplier unduly 'loses' under the contractual obligations (to cover for shortfalls), they will almost certainly look to recoup the losses incurred with you by means fair or foul. Just because you have a contract clearly defining obligations, this does not mean you have mitigated against the problem; it will likely raise its head in another form elsewhere on the project unless you have truly instigated a partnership relationship with the supplier.

Availability of prefabrication

If you have outsourced prefabrication, there are some obvious pitfalls to be aware of. For example:

- The prefabricator may not remain in business. In this case it is important to have put in place contingency fabricators.
- The delivery of the correct shape, grade and volume of ordered unfabricated steel to the fabricator may not be reliable (see above).
- You may not have much control over the fabricator's processing function. It is obviously preferable if you are able to identify and do something about shoddy workmanship before it leaves the factory, rather than when it arrives on site.

Some of the most ugly and costly disputes involving fabricators tend not to involve shoddy workmanship, but rather the fabricator leveraging exhaustive 'rents' from their customer once they become aware of just how few alternatives, if any, the customer has. As in any business relationship, if the party with the upper hand learns of their power, they may well abuse it when particular situations arise (this is usually related to an imbalance originating in the commercial arrangements and/or working relationships between the counterparties). The following section considers how the commercial element of contractual arrangements can be kept fair even during times of market volatility.

Surety of delivery/surety of price

The preceding sections have covered most of what needs to be understood about providing surety of timely delivery of the right steel. The following addresses the issues and problems associated with the provision of timely delivery at the right price, that is to say, 'within budget'.

It has already been established that standard-sized steels in the common 'commercial quality' grades tend to be ubiquitously available ex stock, although it may be necessary to pay an increased price should local demand outstrip local supply demand. The world will, in fact, rarely ever run out of such steels.

The first thing to understand with regard to surety of price is that if product shortages occur, they can generally be quickly and easily resolved by throwing money at them. A contingency fund should be provided for this eventuality as it will almost certainly arise.

More important than this, however, is the incompatibility of the traditional fixed price, lump sum contract with true price surety. This incompatibility derives from the nature of the long steel product supply chain, because, as explained above, the market is based on floating prices, no matter what the production routing. The contractor or developer that demands a fixed price lump sum is asking for trouble.

The following scenario illustrates this inherent incompatibility:

- It is September 2007, and a contractor is finalising a lump sum contract for 100 000 mt of rebar to be delivered over 18 months, with peak demand – 60 000 mt – due Q2 and Q3 2008. International rebar prices had been modestly volatile over the preceding 2 years. Absolute prices had ranged between US$450–650 mt CFR international ports, and 90 days volatility had its peak at 17% but averaged 9%.

It would not have been unreasonable for a contractor to secure supply proposals based on raw rebar costing US$600–700 mt and history shows that this is indeed what happened.

However, history also shows us that international rebar prices rose beyond US$1500 mt by the summer of 2008. Let us recall that stockholders are just middle men taking a 3–6 month gamble on steel they hope to sell at a profit. They need to make money when markets rise to ensure they have some cover for when markets fall, leaving them with unsold inventory purchased above local retail prices. It is a margin business and few, if any, have pockets deep enough to endure a loss of US$800–900 mt across their product range. For this reason many lump sum contracts were renegotiated or simply torn up as stockholders could not endure the loss.

In Q4 2008 and Q1 2009, the opposite happened. Lump sum contracts confirmed at summer 2008 prices were renegotiated or simply torn up, as contractors were unable to endure the 'loss' of paying US$1500 mt when the same rebar was readily available for US$1000 mt less. The cost of reneging on the contract was less than the loss incurred on the steel.

Lump sum contracts become unhinged when price volatility in the product supply chain exceeds 'manageable' levels.

Some common methods of addressing this reality are reviewed below:

1. Price escalation clauses.
   - If material input prices increase by 10%, the contractor will absorb them. The developer must absorb increases above 10%.
   - If material input prices increase by 10%, the stockholder must absorb them. The stockholder and contractor will absorb increases above 10% at a ratio of 50:50.
2. Min–max clauses.
   - The contractor will pay the market price (e.g. average per month) within a certain range. Any costs above and/or below the min–max range are shared.

Regardless of how they are divided, such exercises of 'managing' and 'absorbing' costs simply keep the price risk (the risk of prices moving against you) within the steel supply chain. To cover for their share of the price risk, each party in the supply chain duly loads up their onward sales price according to the share of the risk they are taking. Ultimately, the developer pays a hefty price for the steel because each party in the supply chain has successively inflated their onward sales price. The actual prices of scrap, billet and rebar, however, are correlated. Mathematically there is only one price risk, rather than a series of separate price risks. By focusing only on individual price exposure, the links in the steel supply chain have grossly miscalculated and over-charged the cost of that risk, and while the party that pays for that risk is the developer, in actual fact the party ultimately footing the bill is the general public. Now multiply that by all the products with similar supply chains and no tools with which to efficiently hedge away the price risk. Price surety has been achieved perhaps, but at an inflated cost.

There is a different, lower cost way to achieve price surety. Take for a moment the energy supply chain, or that of base metals (aluminium, copper, zinc, lead, tin and nickel). These supply chains use derivatives such as futures, options and swaps to 'sell' the price risk outside the physical supply chain, to competing investment banks and trading companies. In effect, these parties offer 'price risk insurance', and the industrial supply chain pays them an 'insurance premium' to have them remove the price risk – indeed, to offer them fixed prices. Derivatives are merely tools to manage price risk and achieve price certainty in an efficient manner.

When the physical supply chain uses derivatives to achieve price certainty, this is referred to as hedging. The buyers of that risk (investment banks, commodity trading companies etc.) are, like all insurers, speculators. Insurance companies speculate that the cost of all claims will be less than the income from insurance premiums. They may make it sound like a non-gambling business model, but using complex mathematics and historical data to predict the odds does not hide the fundamentals, namely that insurance providers are speculators. Similarly, the buyers of commodity price risk are also speculators.

Once the deal is done, financial performance is assured, irrespective of the volatility in the market price of the goods in question. Derivatives remove the incentive to renegotiate and renege on contracts, because the net sales/purchase price is guaranteed. The prudent use of derivatives instruments is, therefore, the surest way to price certainty in physical supply chains.

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Chapter 5

Concrete: an introduction

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A historical context is given in order to highlight the stages of development of concrete, from 7000 BC to the present time. Using this section, the inherent advantages and disadvantages of concrete as a construction material are given and both reinforced and prestressed concretes and many special types of concrete which can be used in construction are introduced. This chapter also highlights the need for ensuring both the durability and sustainability of concrete as a construction material. The currently used methods are listed and future trends are summarised. All these topics are elaborated in different chapters within the section on Concrete and hence only a brief summary is given in this chapter. No discussion on this versatile material can be confined to the different chapters in the section on Concrete and hence readers are advised to refer to further reading materials listed in each chapter for a thorough understanding of concrete as a construction material for the twenty-first century.

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Historical context

> *That knowledge which clings to one single effect as if it were the whole, without reasons, without foundation in truth, is narrow and therefore trivial. (from Srimad Bhagvad Gita)*

Concrete is a mixture of loose gravel or crushed rock and sand, held together by a cementing material (**Figure 1**). The chemical reaction between cementing material and water, known as hydration, leads to a rock-like mass and its properties are influenced by the materials used and the chemical reaction. The strength of the matrix is governed by the porosity of the hardened mass, which depends on the initial water/cement ratio (w/c) and the extent of the chemical reaction (hydration). Both these influence other properties of concrete, such as workability, dimensional stability, early-age mechanical properties and durability. While studying the different stages of development of concrete, four key periods can be identified:

- ancient concrete (7000 BC to AD 500)
- emergence of Portland cement and reinforced and prestressed concretes (AD 1700–1900)
- modern concretes (since 1900)
- sustainability of concrete (since 2000).

The contribution to the development and use of ancient concrete was made by Egyptians, Greeks and Romans. Structures built during their time are still landmarks to their durability, thanks to the use of combinations of volcanic ash and lime in inter-ground form. The Romans used lightweight concrete for the construction of the Colosseum in AD 82 and the Pantheon in AD 127, both in Rome, Italy. Towards the end of the first century, the Romans became conscious of the cost of manufacturing concrete and started using local materials such as lime and incorporating crushed bricks, tiles and pottery as aggregates. This is now a major area of research for making concrete a sustainable construction material. Quality control was also given emphasis by rigorously following standard regulations and the supervision of the work on site by qualified craftsmen.

Through John Smeaton's experiments in the 1770s, concrete re-emerged as a construction material, with the ability to be used in challenging constructions, such as the Eddystone Lighthouse, Plymouth, England (1756–1759). He manufactured the lime mortar using limestone containing clay impurities (clay and lime in proportions varying from 3:14 to 1:17 to result in better strengths). In 1824, Joseph Aspdin, a brick layer from Leeds in England, obtained a patent for a superior cement resembling Portland stone; thus emerged Portland cement. During the nineteenth century, notable structures, such as the Thames Tunnel in London (1825–1845) and Osborne House on the Isle of Wight (1845–1848), were constructed using Portland cement.

The world's first use of reinforced concrete was in 1848 by Jean-Louis Lambot in France for the construction of a boat. He plastered a layer of fine concrete (mortar) over a network of iron rods and mesh, which is now known as ferrocement. However, the inventor of reinforced concrete was William Wilkinson, a Newcastle builder, who in 1854 embedded a network of flat iron bars or wire ropes in floors and beams of flat or arched concrete. The first reinforced concrete bridge was constructed in 1870 at Homersfield near Bungay in Suffolk, England. Reinforced

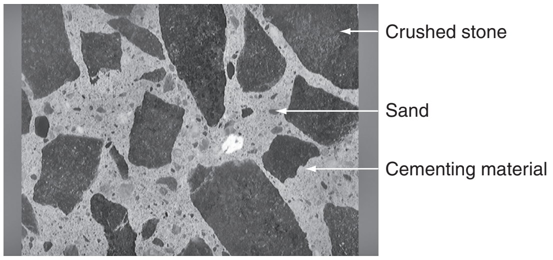

Figure 1 Concrete – the material which rationalised construction

concrete became popular due to its improved fire resistance, and by the beginning of the twentieth century it emerged as the foremost construction material.

However, a major drawback of the reinforced concrete system was cracking in the tension zone under service loading. In order to overcome this difficulty, Eugene Freyssinet, a French pioneer, introduced the concept of prestressed concrete in 1936 at a special meeting of the British Institution of Structural Engineers in London. He suggested that by combining concrete with high-strength prestressing steel, a completely new material possessing properties very different from those of ordinary reinforced concrete could be obtained and this new material would always be in compression and, hence, would not allow tension cracking under service loads. During the post-World War II period, numerous research led to the advancement of this technique for the construction of large-span bridges and other structures.

The use of chemical and mineral admixtures for modifying both fresh and hardened concrete properties led to development of different types of special concretes since 1950s. The concrete mix design procedures were refined during this period. These developments led to the introduction of:

- no slump concrete
- lightweight concrete
- rapid hardening concrete
- heavyweight concrete
- foamed concrete
- pumped concrete
- sprayed concrete
- ferrocement
- fibre-reinforced concrete
- self-compacting concrete.

The application of concrete as a construction material multiplied due to the ease of forming complex shapes and constructing in severe environments, without compromising on quality, safety and serviceability. C. D. Pomeroy stated in 1978:

> *If ordinary Portland cement had been an expensive product it would still have been widely used, but much more highly respected.*

One can justify this statement because the world production of concrete is over 200 million cubic metres per year – that is about 2 cubic metres per year per person in the world, and it is the second largest used material in the world, next to water. The lack of respect for the material contributed to lack of quality assurance in construction, which in turn led to the premature deterioration of reinforced concrete structures. It is estimated that more than 50% of the construction budget of developed countries has been spent on repair and rehabilitation projects due to the premature deterioration of concrete structures. Therefore, ensuring durable structures has been a priority for all structural engineers in the latter part of the twentieth century.

The world consumption of Portland cement increased from less than 2 billion tonnes in 1980 to more than 1.3 billion tonnes in 1996. Not only has this imposed huge demands on concrete-making materials (raw materials used to make cement, natural aggregates and water) but also has increased the release of carbon dioxide at a rate equal to the production of cement. The world is going through a transition phase due to the ever-increasing demands on its natural resources and a similar decrease in availability of these to sustain living on earth; the situation with concrete is exactly the same. Therefore, attention recently has been to make concrete a sustainable construction material, by a combination of rationalisation of cement manufacturing processes, use of waste materials instead of natural resources and reducing the use of cement by either substituting cement with supplementary cementitious materials or using chemically activated low-energy binders using waste materials.

Concrete as a structural material

Concrete is used in constructions nowadays in three different forms (excluding special types), as plain, reinforced and prestressed concrete. Properties of all these forms are controlled by the materials used, the manufacturing processes and the treatments given after their manufacturing. In plain concrete, the properties of interest are:

- compressive strength and modulus of elasticity
- workability
- dimensional stability (bleeding, creep, shrinkage)
- tendency to cracking during service conditions (non-structural and structural cracks)
- durability (resistance to freeze–thaw, salt scaling, chemical reactions such as internal and external sulfate attack, acid attack and alkali attack, alkali–aggregate reaction, etc.).

The durability in general depends on interactions between concrete and the exposure environment. Therefore, concrete needs to be designed to resist a particular service condition. This would also mean that there is a need to analyse the exposure conditions carefully and identify environmental parameters which are likely to interact with the concrete.

In the case of reinforced concrete, as steel reinforcing bars are used to provide tensile resistance to the concrete, additional properties such as those listed below are of interest:

- bond strength between reinforcement and concrete
- fire resistance
- corrosion of reinforcement.

The protection to embedded steel is provided by the cover concrete and, hence, properties of the cover concrete are extremely influential to the durability of reinforced concrete structures. As in the case of plain concrete, the deterioration of reinforced concrete is also influenced by the interactions between the concrete and the service environment. In particular, the corrosion of reinforcement is dependent on the resistance of the cover concrete to the transport of deleterious substances, such as carbon dioxide and chlorides, from the environment to the steel. Therefore, it is essential to limit the tensile crack widths to a minimum so that reinforcement corrosion can be reduced.

As concrete is subjected to compressive stresses using high-tensile steel in prestressed concrete, both creep and shrinkage play a major role in the safety and serviceability of prestressed concrete structures. In addition, corrosion of prestressing tendons needs to be given attention because, as in the case of reinforced concrete, steel tendons could corrode when carbon dioxide or chlorides reach their location.

The outcome of a survey by the author on factors influencing the deterioration of concrete structures (from published case studies) is shown in **Figure 2**. The importance to be given to proper selection of materials, quality of the concrete and the specified cover can be seen in this figure.

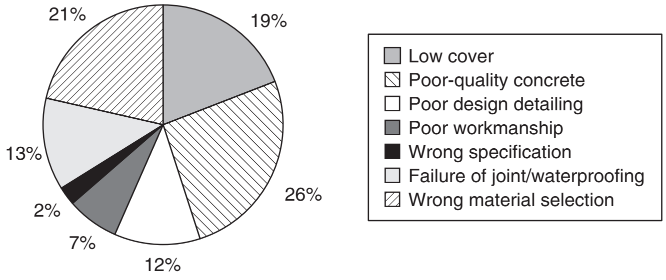

Figure 2 Factors influencing deterioration of concrete structures

Ensuring safe, durable and sustainable concrete structures

The process of providing safe, durable and sustainable concrete structures begins at the conceptual stage of a project itself, when decisions on the type of construction, design and detailing, material selection, method of construction and strategy for continuous monitoring and testing and maintenance management are taken. Structures normally withstand the intended service load if durability is not compromised. Therefore, extra attention needs to be given to durability designs, which involve careful consideration given to the 3 M's, i.e. materials, methods and manual labour (representing both male and female labour), as listed below:

Materials:

- cement (type, quantity and quality), aggregate (type, size, shape and grading), water (quality and quantity)
- supplementary cementitious materials (ground granulated blastfurnace slag, pulverised fuel ash, microsilica, metakaolin, etc.)
- admixtures (chemical and mineral) and additives.

Methods:

- design and detailing
- type of formwork (impermeable vs controlled permeability formwork)
- manufacture of concrete (method of production of concrete, placement, compaction, finish, etc.)
- treatment given after manufacturing (curing regimes, type of surface treatment applied, etc.).

Manual labour:

- quality control
- workmanship.

With the judicial combination of the above 3 M's, it is now possible to manufacture high-performance concretes (that is, concrete that would perform adequately when exposed to a service environment for which it has been designed). This should not be confused with high-strength concrete because high-strength concrete may not give long service life in all exposure environments; for instance, high-strength concrete containing no air-entrainment may not be resistant to freeze–thaw cycles.

The service life of concrete structures comprises two stages, i.e. an initiation phase when material undergoes changes due to interactions with the environment and the propagation phase when active deterioration occurs. Therefore, one of the approaches to ensure the intended service life is to embed sensors which will monitor changes in the material during the initiation phase so that any abnormality can be detected early and appropriate remedial actions put in place before active deterioration occurs. This can be complemented by regular non-destructive testing using in situ methods which are capable of yielding information on the progressive deterioration of a structure. For example, it is known that transport properties, such as absorption, diffusion and permeability, are related to the durability of concrete structures and, hence, these properties may be measured on a regular basis to assess the progressive deterioration of a structure.

Concrete as sustainable construction material

As stated earlier, the production of each tonne of Portland cement releases approximately the same quantity of carbon dioxide to the atmosphere. This accounts for approximately 7% of the world's carbon dioxide emission. However, compared to other construction materials, such as steel, the energy required for the production of concrete is relatively low. The manufacture of steel per tonne requires an energy input of 40 GJ, whereas the equivalent figure for concrete is 1.5 GJ. However, the manufacture of concrete utilises large quantities of natural resources. Therefore, there is a need to conserve concrete-making materials and improve the durability of concrete structures to establish concrete as a sustainable construction material. The former can be achieved either with the use of waste materials as substitutes for natural resources or by resorting to recycling old concrete for the production of new concrete. The latter can be achieved by resorting to the 3 M's listed above in addition to resorting to non-ferrous reinforcement in constructions.

There are numerous applications where concrete could be used for environmental protection as well. For instance, it is an effective medium for the encapsulation of nuclear waste and contaminants from derelict sites. It can also be used to stabilise soils and immobilise heavy metals in soils.

Future trends

Concrete is continuously changing due to developments in chemical admixtures and environmental pressures to reduce the emission of carbon dioxide. Many chemically activated systems, where Portland cement is totally replaced with activated industrial by-products as binders, are emerging at the moment. These are likely to be beneficial to the precast industry and many specialist application areas. A close examination of the world trend in construction materials would reveal the dominance of concrete as a construction material. Therefore, there is a need to change the attitude to this material, as highlighted by C. D. Pomeroy, so that future structures will not only be strong but also safe, durable and sustainable.

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CONTENTS

Chapter 6

Different types of cement used in concrete

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Portland cement has been the most commonly used binder in concrete for a long period since its invention in the late nineteenth century, but due to the beneficial effects of mineral admixtures, such as pulverised fuel ash, ground granulated blastfurnace slag and microsilica, modern cements now contain one of these materials, in addition to small quantities of other types of fine powder used as filler material. Therefore, knowledge of the different types of cements available on the market and their chemical and mineralogical characteristics is important for selecting cements for different exposure conditions. In this chapter, different types of cement specified in the new European Standard on cementitious material are introduced and a discussion on various special types of cements is given.

Introduction

Properties of concrete in both fresh and hardened states depend, to a greater or lesser extent, on the properties of cement used. However, the wide variety of cement types and the use of various types of cementitious and other materials in blended cements may result in confusion regarding the type of cement to be used for a particular work. Therefore, it is important to understand how the various types of cement affect the fresh and hardened properties of concrete. In this chapter an attempt is made to provide a general background and description of various Portland cement types, along with brief discussions on the way they affect properties of the concrete. The general background and description of normal Portland cement is not included. Such information is available in any textbook on concrete, as well as many specialised books on cement; a particularly recommended reference is the ACI Committee 225 *Guide to the Selection and Use of Hydraulic Cements* (ACI Committee 225, 1987). This chapter also introduces two other types of cement, namely high alumina cement and alkali-activated cement.

Main cement types

In order to facilitate the discussion, a list of various generic types of Portland cement produced around the world is summarised in **Table 1**. It is to be noted that the descriptions given in this table are very general and national standards may have specific limits for the contents of constituents and there may be several different 'sub-types' of cement with different levels of pulverised fuel ash (pfa, also known as fly ash) and ground granulated blastfurnace slag (ggbs) and mixes of slag, pfa and limestone powder. The types of cement and their composition, as given in BS EN 197-1: 2000 (BSI, 2000), are summarised in **Table 2**.

The composite cements are those in which a proportion of the Portland clinker is replaced by industrial by-products, such as ggbs and pfa, certain types of volcanic materials (natural pozzolans) or limestone. **Table 3** gives the characteristics of the constituents of composite cements. In addition, microsilica (MS), also known as silica fume, and metakaolin (MK) are also used as supplementary cementitious material in concrete to improve various physical and durability properties of concrete. The characteristics of MS and MK are summarised in **Table 4**.

The mineralogical composition of cements used in special purposes varies, as can be seen from **Table 5**. In this table, the typical chemical compositions of normal, sulfate-resisting and white Portland cement are summarised.

Sulfate-resisting Portland cement

Sulfate-resisting Portland cement (SRPC) is a special type of cement, which is used in conditions where there is a risk of damage to the concrete due to sulfate attack. SRPC is produced by fusing together a precisely controlled blend of very finely ground limestone, shale and iron oxide at high temperatures to form clinker. A small quantity of gypsum is then added to this clinker before grinding to produce the final product. As the tricalcium aluminate phase (C_3A) of cement is attacked by sulfates, the sulfate resistance in SRPC is achieved by adjusting its chemical composition so that the C_3A content is less than 5% (Tikalskya *et al.*, 2002). Due to the lower-level alkalis in SRPC, it is sometimes used to minimise the alkali–silica reactivity in concrete.

Type	Designation	BS EN 197-1 notation	Constituents	Applications
Normal Portland cement	Pure	CEM I	Clinker and calcium sulfate	All types of construction except those exposed to sulfate exposure
Sulfate-resisting Portland cement	Pure	BS EN 197-4 (2004) (BSI, 2000)	Low C_3A clinker and calcium sulfate	Concrete exposed to sulfate attack
White Portland cement	Pure	CEM I	Special low iron content clinker and calcium sulfate	Architectural finishes
Portland pulverized fuel ash cement	Composite	CEM II	Clinker, pulverised fuel ash and calcium sulfate	All types of construction, where concrete is required to have improved sulfate resistance and protection against alkali–silica reaction, mass concrete
Portland slag cement	Composite	CEM III	Clinker, granulated blastfurnace slag and calcium sulfate	All types of construction, where concrete is required to have improved sulfate resistance and protection against alkali–silica reaction, mass concrete
Portland limestone cement	Composite	CEM II	Clinker, limestone and calcium sulfate	All types of construction
Pozzolanic cement	Composite	CEM II	Clinker, natural pozzolan and calcium sulfate	All types of construction, particularly where concrete is to improve the durability
Masonry cement	Composite	BS EN 413-1 (2004) (BSI, 2004)	Clinker, limestone and air-entraining agent	Brickwork, blockwork and rendering

Table 1 Main types of cement produced around the world

The use of SRPC is recommended for the following applications:

- structures in sewage treatment plants;
- structures in coastal areas;
- piers and foundations;
- structures in chemical, fertiliser and food processing industries.

In concrete containing SRPC, a lower amount of C_3A is available for binding the chloride ions (Suryavanshi *et al.*, 1995), therefore the use of SRPC may be disadvantageous if there is a risk of the presence of chloride ions in concrete containing steel reinforcement.

White Portland cement

White Portland cement is similar to normal Portland cement in all respects except its whitish colour. It is somewhat more expensive than the normal Portland cement. This is due to the requirement of substantial modifications in the manufacturing method to obtain the white colour.

China clay, containing little or no iron or manganese oxide, is fired with chalk or limestone to produce white Portland cement (Gani, 1997). As the use of coal results in the addition of ash to the clinker and this can be a potential source of iron impurities, either gas or oil is normally used as fuel for the production of white cement clinker. To avoid contamination, special precautions also have to be taken in the grinding process. All these aspects increase the cost of white Portland cement by about three times the price of normal Portland cement. White Portland cement is generally used for architectural projects, where pure white concrete is specified. It is also used to produce vibrant colour concrete and mortar.

Portland pulverised fuel ash cement

Pulverised fuel ash (pfa), otherwise known as fly ash (fa), is defined in ACI Committee 116 document *Cement and Concrete Terminology* (ACI Committee 116, 1988) as 'the finely divided residue resulting from the combustion of ground or powdered coal, which is transported from the firebox through the boiler by flue gases'. Pulverised fuel ash is usually finer than the normal Portland cement, has a very rounded shape as opposed to the extremely jagged particle shape of cement and is of lower density (Day, 2006) (specific gravity usually 1.9–2.4 compared to 3.15 for cement).

Pulverised fuel ash is used as a constituent of blended cement or in combination with normal Portland cement. In either case the normal proportion of pfa is 20–35% of the total cement (Berry and Malhotra, 1980; Al-Ani and Hughes, 1989; Lam *et al.*, 1998; Joshi and Lohtia, 1999; Han *et al.*, 2003). Portland pfa cement is manufactured by burning a precisely specified mix of raw materials containing lime, silica, alumina and small quantities of other materials to form a clinker and then inter-grinding that clinker with pfa and calcium sulfate, the latter being added to control the setting. Although Portland pfa cement can be used in concrete, mortar, render, screed and grout, it is ideal for self-compacting concrete.

The hydration of Portland pfa cement includes two processes: first, the hydration of cement particles and, second, the pozzolanic reaction between pfa and $Ca(OH)_2$, which is released in the cement hydration. It has been observed by

Cement	BS EN cement notation#	Clinker content %	Content of other main constituents %
Portland cement	CEM I*	95–100	<5
Portland-slag cement	CEM II/A-S	80–94	6–20
	CEM II/B-S*	65–79	21–35
Portland-silica fume cement	CEM II/A-D	90–94	6–10
Portland-pozzolana cement	CEM II/A-P	80–94	6–20
	CEM II/B-P	65–79	21–35
	CEM II/A-Q	80–94	6–20
	CEM II/B-Q	65–79	21–35
Portland-pulverized fuel ash cement	CEM II/A-V	80–94	6–20
	CEM II/B-V*	65–79	21–35
	CEM II/A-W	80–94	6–20
	CEM II/B-W	65–79	21–35
Portland-burnt shale cement	CEM II/A-T	80–94	6–20
	CEM II/B-T	65–79	21–35
Portland-limestone cement	CEM II/A-L	80–94	6–20
	CEM II/A-LL*	80–94	6–20
	CEM II/B-L	65–79	21–35
	CEM II/B-LL	65–79	21–35
Portland-composite cement	CEM II/A-M	80–94	6–20
	CEM II/B-M	65–79	21–35
Blastfurnace cement	CEM III/A*	35–64	36–65
	CEM III/B*	20–34	66–80
	CEM III/C	5–19	81–95
Pozzalanic cement	CEM IV/A	65–89	11–35
	CEM IV/B	45–64	36–55
Composite cement	CEM V/A	40–64	36–60
	CEM V/B	20–39	61–80

Note:# Explanation of the notation used:

CEM II/A–S 42.5 N: CEM II – Main cement type; indicates higher (A), medium (B) and lower (C) proportion of Portland cement clinker; S – sub-type indicates the second main constituent (in this case blastfurnace slag); 42.5N – standard strength class; N – sub-class, N indicates normal early strength and R indicates rapid early strength.

The code letters used to indicate the sub-type and thus the second main constituent are:

S – blastfurnace slag; D – silica fume; P – natural pozzolana; Q – natural calcined pozzolana; V – siliceous fly ash (e.g. pfa); W – calcareous fly ash (e.g. high-lime fly ash); T – burnt shale; L – limestone; M – two or more of the above.

* Cement manufactured in the UK.

Table 2 Types of cement and their composition as in BS EN 197-1: 2000

Constituents	Natural pozzolan	PFA	GGBS	Limestone
Reaction type	Pozzolanic		Latently hydraulic	Hydration modifier
SiO_2 (%)	60–75	38–64	30–37	3
Al_2O_3 (%)	10–20	20–36	9–17	0.5
Fe_2O_3 (%)	1–10	4–18	0.2–2	0.5
CaO (%)	1–5	1–10	34–45	51
MgO (%)	0.2–2	0.5–2	4–13	2
SO_3 (%)	<1	0.3–2.5	0.05–0.2	0.3
Na_2O (%)	0.5–4	0.2–1.5	0.2–1	0.2
K_2O (%)	1–6	04–4	0.3–1	0.1
Loss on ignition (%)	2–12	2–7	0.02–1	42
Specific gravity	2.30–2.80	2.30	2.90	2.70
Specific surface area (m^2/kg)	300–600	300–600	300–500	400–900

Table 3 Nature of composite cement constituents (adapted from Newman and Choo, 2003)

Oxides (%)	Microsilica	Metakaolin
SiO_2	92.0	52.1
Al_2O_3	0.70	41.0
Fe_2O_3	1.23	4.3
CaO	0.30	0.09
MgO	0.21	1.4
SO_3	0.33	–
Na_2O	1.23	0.01
K_2O	0.82	0.62
Loss on ignition	3.18	0.48
Specific gravity	2.20	2.40
Specific surface area (m^2/kg)	15 000–20 000	12 000–15 000

Table 4 Characteristics of microsilica and metakaolin (adapted from Newman and Choo, 2003)

researchers (Aiqin, 1995; Aiqin *et al.*, 1997) that in the case of Portland pfa cement hydration process, both the hydration of cement particles and the pozzolanic reaction of pfa accelerate each other. The $Ca(OH)_2$ released in the cement hydration gives a condition for the pozzolanic reaction of pfa. On the other hand, in the pozzolanic reaction pfa absorbs the $Ca(OH)_2$ and promotes the cement hydration. However, the degree of hydration of the system reduces with the increase in pfa content because the activity of pfa is lower than that of cement (Aiqin *et al.*, 2004).

The following benefits can be obtained with the use of Portland pfa cement in concrete compared with that of the normal Portland cement concrete at equal water-cement ratio (Virtanen, 1983; Idorn and Henrisken, 1984; Malhotra, 1990; Bilodeau and Malhotra, 1992; Bilodeau and Malhotra, 2000):

- improved workability and cohesion;
- reduced bleeding;
- enhanced resistance to sulfate attack;
- enhanced resistance to chloride ingress;
- more effective mitigation of damaging alkali–silica reaction.

The drawbacks of using Portland pfa cement in concrete are as follows (Halstead, 1986; Langey *et al.*, 1992):

- increased setting time;
- reduced early strength;

	Normal	Sulfate-resisting	White
SiO_2	20.4	20.3	24.6
Insoluble residue	0.6	0.4	0.07
Al_2O_3	5.1	3.6	1.9
Fe_2O_3	2.9	5.1	0.3
CaO	64.8	64.3	69.1
MgO	1.3	2.1	0.55
SO_3	2.7	2.2	2.1
Na_2O	0.11	0.10	0.14
K_2O	0.77	0.50	0.02
Loss on ignition	1.3	1.3	0.85
Free lime (%)	1.5	2.0	2.3
LSF (%)	96.6	97.3	95
SR (%)	2.55	2.33	11.2
AR (%)	1.76	0.71	6.3
Calculated mineral composition			
C_3S	57	62	67
C_2S	16	12	20
C_3A	9	0.9	4.5
C_4AF	9	16	0.9

Table 5 Typical composition (%) of normal, sulfate-resisting and white Portland cement (adapted from Newman and Choo, 2003)

- reduced heat generation, which is advantageous in hot weather.

Portland slag cement

Slag (also known as blastfurnace slag) is a by-product from the manufacture of iron in a blastfurnace. The slag is used in granulated form to manufacture either the blended slag cement or slag concrete. Portland slag cement is most widely used in concrete, either as a separate cementitious component or as part of a blended cement. It can be manufactured by two different methods. In the first method, it is manufactured by inter-grinding clinker and granulated slag in the ball mills, while in the second method normal Portland cement and granulated slag are blended to manufacture the Portland slag cement. A separate grinding followed by factory blending of the two powders is used by most of the cement producers in Europe (Newman and Choo, 2003).

Normally, Portland slag cements show a higher fineness than normal Portland cement with the same strength class (Lang, 2002). Therefore, in concrete containing Portland slag cements, a higher water demand can be expected. The setting times of Portland slag cement increase with the increase in the slag content (Carette and Malhotra, 1984; Lang, 2002). This is due to the lower hydraulic properties of Portland slag cements compared to the normal Portland cement. As the heat development in concrete containing Portland slag cement is less compared to the normal Portland cement, it is particularly suitable for mass concrete works. Moreover, as Portland slag cement hydrates more slowly than that of the normal Portland cement, the strength development of Portland slag cement concrete at early age is also lower. However, if sufficient moisture is available, the long-term strength of such concrete could be higher (Newman and Choo, 2003). The improvement in the strength at later age is due to (i) the prolonged hydration reaction of the Portland slag cement and (ii) the denser microstructure that is formed as a result of the slower hydration reaction. However, it is to be noted that the hydration reaction of Portland slag cement is temperature dependent. An increase in the temperature increases the rate of strength gain of Portland slag cement and the opposite is true as the temperature decreases. Concrete containing Portland slag cement if cured properly is likely to be more durable than similar normal Portland cement concretes.

Portland slag cement can be used in concrete applications such as:

- mass concreting in dams and retaining walls;
- pavements and foundations;
- concrete structures exposed to harsh environments, such as wastewater treatment and marine applications.

Portland limestone cement

Portland limestone cement (PLC) contains a controlled level of high purity limestone and can be used as an alternative to normal Portland cement. PLC is produced by milling clinker, limestone and a given amount of gypsum. The European Standard EN 197-1 (BSI, 2000) identifies four types of PLC according to the amount of limestone in it. Types II/A-L and II/A-LL contain 6–20% limestone, whereas types II/B-L and II/B-LL contain 21–35% limestone (**Table 2**). PLC containing limestone as high as 30% has been produced and used in various construction applications in France and Italy for many years (Newman and Choo, 2003).

The use of PLC in concrete has both technical and economical benefits. The technical benefits are the satisfactory physical properties of PLC with a reduction in the water demand and important in the workability (Baron and Dourve, 1987; Schmidt, 1992; Neville, 2003). The economical benefit is achieved by a reduction in the energy consumption in producing PLC (Tsilvis *et al*., 1998).

PLC is used in constructions where there is no risk of chemical or thermal attack. Applications may include ready-mixed concrete, precast concrete, grouts, screeds, concrete blocks, renders and mortars for block and bricklaying. As PLC has a lighter colour than normal Portland cement, it is particularly suitable for decorative and architectural works.

Pozzolanic cement

The ASTM C 618 (ASTM, 1994), in the definition of pozzolan, states the following: 'Pozzolans are siliceous or alumino-siliceous materials which have little or no cementing value by themselves, but when finely ground and in the presence of water, they chemically react with calcium hydroxide at room temperature in order to form compounds with cementing properties.'

Pozzolanic cement is produced by mixing a material known as pozzolan and finely ground calcium hydrate. Pozzolanic cement is of low mechanical strength and its rate of setting is slower than the normal Portland cement. Therefore, it is mainly used in masonry applications. As pozzolanic cement paste has a high content of large pores, the total porosity is higher (Takemoto and Uchikawa, 1980). However, these pores are connected with thinner and more segmented pores and this decreases the permeability (Massazza, 1993). Therefore, the use of pozzolanic cement improves various durability properties of concrete.

Pozzolanic cements can be used for concrete in:

- aggressive environments;
- hydraulic, port and coastal works;
- road and tunnel works;
- geologic consolidation works.

Masonry cement

Masonry cement is produced by mixing normal Portland cement and plasticising materials, such as limestone or hydrated lime. As the name suggests, masonry cement is specially formulated and manufactured to produce masonry mortar for use in brick, block and stone masonry works. It can also used to produce stucco. Mortars produced with masonry cement provide an excellent performance in terms of workability, strength and durability (Dubovoy and Ribar, 1990; Davison, 1981; Zamatis, 1959).

Microsilica as a supplementary cementitious material

Microsilica (MS) is a by-product of the silicon and ferrosilicon industries. Due to its extreme fineness and high silica content, MS is a highly effective pozzolanic material (ACI Committee 226, 1987). The use of MS as a supplementary cementitious material in concrete provides the following benefits (Asgeirsson and Gudmundsson, 1979; Perry and Gillot, 1985; Mehta, 1985; Gautefall, 1986):

- increased compressive strength;
- reduced absorption and permeability;
- very low chloride ion diffusion;
- improved sulfate resistance;
- improved resistance to chemical attack.

The improved behaviour of concrete containing MS is due to a combination of its pozzolanic reaction and the micro-filler effect. The pozzolanic reaction converts the most vulnerable calcium hydroxide that is produced during the hydration of cement into durable calcium silicate hydrates. On the other hand, the extremely fine particles of MS produce the so-called 'micro-filler effect', which distributes the hydration products in a more homogeneous fashion in the available space which makes the matrix much denser.

The main applications of MS in concrete include:

- high-strength concrete;
- shotcretes;
- structures requiring good surface finish.

Metakaolin as a supplementary cementitious material

Metakaolin (MK) is a thermally activated alumino-silicate material obtained by calcining kaolin clay within a temperature range of 650–800°C (Sabir *et al*., 2001). Due to its high pozzolanic properties, the utilisation of MK as a supplementary cementitious material in concrete has received considerable interest in recent years (Wild *et al*., 1996; Coleman and Page, 1997; Frias and Cabrera, 2000; Asbridge *et al*., 2002). An important difference between MK and other supplementary cementitious materials used in concrete is that MK is a primary product, while others are either secondary products or by-products. Therefore, MK can be produced with a controlled process to achieve the desired properties.

The use of MK improves the compressive strength of concrete, especially during the early stages of hydration (Zhang and Malhotra, 1995; Curcio *et al*., 1996; McCabe, 1999). The increase in the strength of MK concrete is due to: (i) the filling effect where MK particles fill the space between cement particles; (ii) acceleration of cement hydration; and (iii) the pozzolanic reaction. However, after an age of 14 days the contribution that MK makes to concrete's strength is reduced (Sabir *et al*., 2001). The white colour of MK also results in a concrete with lighter colour, which is suitable for colour matching in various architectural applications. Nowadays MK is increasingly used to produce high-strength and high-performance concretes with improved durability properties.

Other cement types

High alumina cement

High alumina cement (HAC) is neither a type of normal Portland cement nor a blend of normal Portland cement. It is a special type of cement, made of raw materials, which are different from those used to manufacture normal Portland cement. The main compound in HAC is calcium aluminates, unlike in normal Portland cement in which it is calcium silicates. The main advantages of HAC over normal Portland cement are its high early strength, high temperature resistance and good chemical resistance (Neville, 2006). However, the hydration products of HAC are chemically unstable and undergo a change known as conversion (Taylor, 1990). The conversion increases the density of the cement hydrates, which results in an increase in the porosity of HAC hydration products. The increase in the porosity and the micro-cracks formed during the conversion reduces the strength of HAC concrete. The use of HAC in new structural concrete was effectively banned in the UK following a few well-known collapses in the 1970s. However, later on it was found that the primary cause of such collapses was either the poor construction details or the chemical attack and not the problems with the concrete itself (Mangalbhai, 1990). The use of HAC can show rapid hardening and strong resistance to corrosion if the recommended guidelines (George, 1975; Mangalbhai, 1990) are followed.

Alkali-activated cement

Alkali activation is a process in which the cement particles are initiated to react with the addition of alkali in the early mixing stages. This activation releases certain chemical constituents, which form the new binding phase, along with preparing the particle surface for bonding. The main difference between normal Portland cement and alkali-activated cement is in the hydration product. The major hydration products in the former are calcium silicate hydrate and portlandite, whereas in the latter cements the hydration products are mainly the poorly crystallised alumino-silicate gels containing alkalis. Based on the compositions, alkali-activated cements can be grouped into geo-cements, alkali-activated pulverised fuel ash, alkali-activated slag, and aluminate cements (Shi *et* al., 2006). The use of alkali-activated cements has gained considerable interest during the last two decades, mainly because of its low energy cost, high strength and improved durability performance compared to normal Portland cements (Davidovits, 1982; Davidodvits and Sawyer, 1985). The presence of excess alkali in alkali-activated cement systems is likely to bring about an increase in the susceptibility of these materials to alkali–aggregate reactions. However, it has been demonstrated by researchers (Bakharev *et al.*, 2001; Fernandez-Jimenez and Puertas, 2002; Xie *et al.*, 2003) that the presence of excess alkali does not greatly enhance the occurrence of alkali-aggregate reactions.

Alkali–activated cements have been commercially produced and used in the former Soviet Union, China and some other countries for the following applications:

- concrete pavements
- concrete pipes
- concrete sinks and trenches
- structural concrete
- masonry blocks
- autoclaved aerated concrete
- refractory concrete
- oil-well cement.

Summary

In this chapter, the range of cements available for today's manufacturing of concrete products and structures is summarised. Features of some of the special cements have been elaborated so that users can identify their strengths and limitations. It is important to consider both the functionality of the cement type and its impact on both physical and durability properties when choosing a particular cement type for any specific job application.

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Chapter 7

Hydration of cementitious materials

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The mechanisms of hydration are described in the context of concrete mix design. Properties of hydration products are described with emphasis being placed on those of calcium silicate hydrates. The evolution in the volumes of solids present and in their compositions with hydration is discussed. Relationships between cement composition, mix design proportions and property development in concrete as influenced by hydration are reviewed.

CONTENTS

Overview of concrete

In its simplest form, fresh concrete is a physical mixture of one of any number of types of Portland cement, sand, aggregate and water. Various chemical and mineral admixtures, which affect the properties of the fresh and the hardened concrete and the economics of concrete production, may also be present. Commonly used mineral admixtures include filler materials, such as ground limestone, pozzolans, such as fly ash or silica fume, and ground air-granulated or water-granulated slags. Cements are hydraulic in that they react with water to produce a hardened mass. For Portland cements the primary binder compound produced during hydration is calcium silicate hydrate, or C–S–H in cement chemistry notation. Pozzolans are particulate solids that contain reactive forms of silica or aluminosilicates. Pozzolans may be natural, such as volcanic ash or man-made, such as fly ash, silica fume or calcined clays. Pozzolans are not hydraulic in themselves, but are capable of reacting with the calcium hydroxide produced during cement hydration to form additional C–S–H. Slags are latent hydraulic materials, containing both calcium and silica. Their hydration is facilitated by the highly basic solutions produced as cements hydrate. The term 'cementitious material' is applied to the combination of cement and pozzolan and/or slag that is present in a concrete mix. While chemically inert, limestone fillers are thought to exhibit beneficial effects with regard to space filling and, possibly, in providing templates for the deposition of hydration products. Overview discussions of the principal topic of this chapter can be found in a variety of texts (Bogue, 1955; Lea, 1971; Mindness and Young, 1981; Neville, 1996; Hewlett, 1998; Mehta and Monteiro, 2006).

Chemical admixtures are added to concrete for various purposes (Ramachandran, 1984). Water-reducing admixtures or superplasticisers are used to facilitate concrete placement at low water-to-cementitious materials ratios. Accelerating admixtures are used to increase the rate of set and strength development, particularly when concrete is placed in cold weather. Alternatively, set retarders may be employed to avoid premature stiffening when concrete is placed in hot weather. Durability admixtures include those to entrain air with the objective of distributing small air bubbles throughout the concrete. When the pore solution starts to freeze, the liquid partially filling these bubbles will do so first. That process draws liquid from the cement paste and allows it to freeze in locations where expansion associated with ice formation can be accommodated with minimal damage. Corrosion-inhibiting admixtures are added to afford protection to embedded steel. The most common of these, calcium nitrite, acts by rapidly oxidising ferrous ion produced by chloride-induced corrosion to ferric ion. The latter rapidly reacts with local hydroxyl ions to form a ferric hydroxide precipitate barrier over the steel thereby reducing access of the chloride ions to the metal surface (Bentur *et al.*, 1997).

Cements

Origins

While commonly attributed to the Romans, the development of cement is more likely an innovation of the ancient Greeks. Construction of durable facilities needed to support transportation infrastructure in ancient Naples, a Greek colony, was the same problem faced by Smeaton in the 1780s. Cement capable of hardening under water and resistant to being washed out was required for both the docks of Naples and the Eddystone lighthouse. The Greeks combined volcanic ash with calcined limestone while Smeaton fired limestone and clay to produce impure dicalcium silicate. Although the reactants differ, both hydration reactions produced calcium silicate hydrate.

Modern cements

Modern Portland cements consist of four major compounds. These are impure $3CaO{\cdot}SiO_2$ (tricalcium silicate), $2CaO{\cdot}SiO_2$ (dicalcium silicate), $3CaO{\cdot}Al_2O_3$ (tricalcium aluminate) and $4CaO{\cdot}Al_2O_3{\cdot}Fe_2O_3$ (tetracalcium aluminoferrite). These are produced by firing limestone and clay at approximately 1450°C. It is usual to denote these compounds using cement chemistry notation as C_3S, C_2S, C_3A, C_4AF, respectively. In addition, calcium sulfate is introduced to the cement when the clinker is being ground to cement fineness. The compound compositions of cements are assessed from the proportions of CaO, SiO_2, Al_2O_3, Fe_2O_3 and 'SO_3' by calculation using the Bogue equation (Bogue, 1955; Lea, 1971).

The silicate-containing compounds in Portland cements are primarily responsible for concrete strength development. Tricalcium silicate is more reactive than dicalcium silicate but requires higher kiln temperatures to form. The cement first commercialised by Aspdin in the 1820s relied on the hydration of dicalcium silicate for strength development. Subsequent technological advances have resulted in compositions containing tricalcium silicate as well as in cements being ground to smaller average particle sizes. $3CaO{\cdot}Al_2O_3$ and $4CaO{\cdot}Al_2O_3{\cdot}Fe_2O_3$ are called interstitial phases and their presence is not regarded as meaningfully contributing to the strength development of mature concrete. These compounds are present as the result of the benefit of iron and aluminum oxides fluxing the formation of the silicate-containing phases during cement production. Calcium sulfate is present in finished cement as a result of being added to cement clinker during the grinding process. Other compounds typically present in cement include free lime, alkali oxides Na_2O and K_2O, MgO and a variety of incidental compounds.

The proportions of the compounds present in cements and the fineness to which they are ground are the subjects of various national and international standards (e.g. *ASTM*, 2006a). Traditionally, these standards have been prescriptive and directed towards strength gain and durability attributes. There is, however, a trend towards performance-based specifications, thereby allowing some departure from standards-based restrictions on the selection of cementitious materials and concrete mix designs (see http://www.nrmca.org/P2P/).

The cementitious materials components of concretes can be combined during cement production or can be separately mixed in producing fresh concrete and practices differ between the US and Europe. Intergrinding mineral admixtures with cement is the norm in Europe. Separate addition of pozzolan or slag at the time of concrete production is common in the US. Cements interground with fly ash or slag at a cement production facility are called blended cements in the US to distinguish them from 'pure' Portland cements. ASTM specifications recognise both the proportions and the compositions of the pozzolan or slag used in blended cements. The compositions of fly ashes vary greatly depending on the source of the coal and the manner in which it is fired. Fly ash may contain a relatively high proportion of lime (ASTM Class C ash) and ashes of this type will behave differently in concrete than do the low lime ashes whose compositions are dominated by silica, alumina and iron oxide (ASTM Class F ash) (*ASTM*, 2006b).

Hydration

Overview

Several aspects of the behaviour of major cement compounds confer great technological importance to them. First, they dissolve in water. Second, they lack true solubilities. Thus, it is not possible to form a solution saturated with respect to C_3S. Rather, C_3S will continue to dissolve as long as water is accessible. Third, they dissolve at rates that are sluggish from a chemical perspective, but appropriate for their intended use and the rates of dissolution can be controlled by grinding and by addition of appropriate chemicals. Fourth, they form hydration products that are of lower densities than the parent solids and in doing so they incorporate water. Thus, space originally occupied by water becomes occupied by solids. Finally, it is the property of these hydration products to accommodate to the space available to them rather than to significantly expand or contract. An excellent discussion of hydration can be found in Taylor (1997).

Hydration of the calcium silicates

Overview of hydration reactions

The silicate phases, namely impure tricalcium silicate ($3CaO{\cdot}SiO_2$) called alite and impure dicalcium silicate ($2CaO{\cdot}SiO_2$) called belite, collectively constitute about 80% by weight of Portland cement. It is the hydration reactions of these silicate phases that are primarily responsible for the development of concrete strength. Alite and belite hydrate to form the same hydration products, calcium silicate hydrate, C–S–H and calcium hydroxide ($Ca(OH)_2$). The hydration reactions for these phases may be expressed as follows:

$$3CaO{\cdot}SiO_2 + 5.3H_2O \rightarrow 1.7CaO{\cdot}SiO_2{\cdot}4H_2O + 1.3Ca(OH)_2 \quad (1)$$

$$2CaO{\cdot}SiO_2 + 4.3H_2O \rightarrow 1.7CaO{\cdot}SiO_2{\cdot}4H_2O + 0.3Ca(OH)_2 \quad (2)$$

where the composition of C–S–H is taken to be $1.7CaO{\cdot}SiO_2{\cdot}4H_2O$. It should be noted, however, that

C–S–H is not a compound of fixed composition. The Ca/Si ratio in C–S–H varies between 0.83 and about 2. Older literature frequently cites the Ca/Si ratio to be 1.5 or $3CaO{\cdot}2SiO_2{\cdot}nH_2O$. The water content of C–S–H also varies, decreasing with drying. When equilibrated to 11% relative humidity, the value of n is about 2.1. For so-called D dried C–S–H, which is equilibrated at −79°C, the value of n is 1.3–1.5 (Taylor, 2006).

While the hydration reactions for C_3S and C_2S produce the same products, their rates and the relative proportions of the hydration products formed are quite different. The rate of alite hydration is significantly more rapid than that of belite. The primary hydraulic compound produced by Smeaton in his experiments and in early Portland cements was belite; thus the hydration reaction expressed in equation (2), has its origins in the eighteenth century.

The hydration reactions shown in equations (1) and (2) produce calcium hydroxide. The physical properties that calcium hydroxide imparts to concrete are inferior to those imparted by C–S–H. Consequently, if a portion of this calcium hydroxide can be consumed in the formation of additional C–S–H, improved concrete properties can be realised. This is the basis for the pozzolanic reaction wherein fly ash, silica fume and other pozzolans react to produce additional C–S–H according to the following reaction:

$$1.7Ca(OH)_2 + SiO_2 \rightarrow 1.7CaO{\cdot}SiO_2{\cdot}4H_2O \qquad (3)$$

Thus, in blended cements a third hydration reaction, the pozzolanic reaction, involving C–S–H formation, occurs. The pozzolanic reaction improves both the long-term strength of concrete and reduces permeability. However, this reaction is sluggish and strength gain during the first 28 days or more can be lower.

Because of its importance in determining property development in concrete, the hydration characteristics of C_3S have been extensively studied. From a mechanistic standpoint, C_3S hydration occurs in three stages. The first of these is responsible for the onset of the induction period and is the result of a layer of C–S–H forming on the surfaces of the hydrating particles. The formation of this layer causes hydration to become diffusionally controlled, with the rate of C_3S hydration depending on the rate of diffusional transport through this layer. The rate of hydration during this stage of hydration is slow and does not appear to be strongly dependent on fineness. This period of hydration is called the 'induction period'. Eventually this layer becomes unstable and a morphological transformation to an acicular, cauliflower-like, C–S–H hydration product occurs. The specific reason for this transformation remains a matter of debate; however, there is evidence to suggest that it is associated with a silicate polymerisation reaction.

Once the initial layer of C–S–H has undergone the transformation to an acicular product, the anhydrous C_3S particle surfaces are again exposed to the concrete pore solution. This results in the onset of a period of rapid hydration, referred to as the 'acceleratory period'. A significant fraction of the alite in cement is consumed during the acceleratory period with the proportion depending on the cement fineness. The acceleratory period eventually comes to an end before hydration is complete and the rate of hydration decreases again because the small particles of C_3S have been consumed and the layers of hydration products that have formed around the larger particles thicken. Transition to a 'post-acceleratory period' is complete when the layers of C–S–H surrounding the hydrating particles are sufficiently thick to cause hydration to again become diffusionally controlled. It appears that the hydration rate during the post-acceleratory period is also independent of fineness. Thus, in the hydration of cement, it appears that the primary difference in behaviour between ordinary Portland cement and that ground to high fineness lies in the amount of hydration that occurs during the acceleratory period.

Hydration of tricalcium aluminate

The percentage of the aluminate phase in Portland cements may vary from essentially sero in so-called 'sero C_3A cements' to as high as 14% or more. C_3A is a highly reactive phase. If mixed with water at room temperature, the rate of heat evolution associated with its rapid hydration may be sufficient to cause the water to boil. The rapidity of this reaction leads to a phenomenon in concrete called 'flash set'. Flash set results because the products of aluminate phase hydration cause a stiffening of the fresh concrete. Premature stiffening associated with aluminate hydration makes placement and finishing difficult. The use of gypsum as a means for controlling flash set goes back over a century to the studies of Michaelis in 1870 (Kanare, 1985). While the mechanistic details by which the hydration of the aluminate phase in cement is retarded by calcium sulfate have remained a matter of debate (Brown *et al.*, 1986), it is generally recognised that retardation is associated with the formation of ettringite, $3CaO{\cdot}Al_2O_3{\cdot}3CaSO_4{\cdot}32H_2O$. The reaction of C_3A in the presence of gypsum to form ettringite may be written as follows:

$$3CaO{\cdot}Al_2O_3 + 3CaSO_4{\cdot}2H_2O + 26H_2O \rightarrow 3CaO{\cdot}Al_2O_3{\cdot}3CaSO_4{\cdot}32H_2O \qquad (4)$$

The amount of calcium sulfate interground with cement clinker is carefully proportioned to ensure that ettringite formation does not continue to any significant extent after about one day when the concrete has started to gain strength. The formation of ettringite is an expansive reaction. It is usually undesirable that an expansive reaction occur in hardening concrete because dimensional instability and cracking may result. An exception is shrinkage-compensating cements where slight expansion counteracts the tendency of concrete to shrink when curing.

The expansion associated with ettringite formation can be accommodated when concrete is in a plastic state and the formation of ettringite formation contributes to set. This is because ettringite crystallites are needle-like and the formation of needle-like structures in the plastic cement matrix causes it to stiffen.

Equation (4) shows how 3 moles of calcium sulfate react with one mole of C_3A to form ettringite. A typical molar ratio of $CaSO_4$-to-C_3A in Portland cement is 0.65-to-1. Hence, there is insufficient sulfate in cement to react will all the available C_3A to form ettringite. Sequentially, C_3A first reacts to form ettringite. When the source of calcium sulfate is exhausted, the unhydrated C_3A will then react with previously formed ettringite to produce $3CaO \cdot Al_2O_3 \cdot CaSO_4 \cdot 12H_2O$. This compound is commonly called monosulfate. The reaction is:

$$2[3CaO \cdot Al_2O_3] + 3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 32H_2O + 4H_2O \rightarrow 3[3CaO \cdot Al_2O_3 \cdot CaSO_4 \cdot 12H_2O] \quad (5)$$

and monosulfate is commonly observed in mature concretes.

If sulfate from an external source enters the concrete pore structure, monosulfate can convert back to ettringite. This is the mechanism of classical sulfate attack (Hobbs, 1998; Skalny *et al.*, 2002). Under some circumstances, ettringite can form in hardened concrete in the absence of external sulfate. This can occur when concrete is cured at a high temperature. Under conditions of high-temperature curing, C–S–H and C_3A compete for sulfate. However, some of the sulfate which is incorporated in C–S–H at high temperature is exsolved at low temperatures and becomes available to react with monosulfate to form ettringite. This process is called delayed ettringite formation and has been found to be damaging to steam-cured concrete (Skalny *et al.*, 2002).

There are two reasons why ettringite formation is damaging to hardened concrete. First, the solid volume of the ettringite exceeds solid volume of the monosulfate from which it forms. Second, ettringite needles tend to bridge pores in the hardening concrete. Once ettringite needles have bridged a pore its continued growth will generate a tensile stress on the pore walls. It is these internal tensile stresses that may eventually lead to crack formation.

Ferrite phase hydration

The proportion of the ferrite phase in Portland cements is in the same range as that of the aluminate phase or slightly lower. As a practical matter, the ferrite phase is unreactive and does not meaningfully contribute to concrete strength development.

Relationships between hydration and concrete property development

Overview of physical changes during hydration reactions

While hydration of individual cement compounds has been studied as model systems for the hydration of cement, it is important to state that physical changes occur in concrete which are not very well approximated by individual cement compounds. For the most part these changes have to do with mechanical property development. In general, this relies on filling of internal spaces originally occupied by the mixing water. For a mix having a water-to-cement ratio of 0.5 by weight, the difference in densities of water and cement indicate that about 75% of the volume is occupied by water. As the hydration reactions in cement proceed, the higher-density cement compounds are replaced by lower-density hydrates and the mixing water is consumed. These coupled events result in the diminution in the total volume of the water-filled space. However, as is discussed in greater detail below, the reactions described above do not all occur at the same rates nor do the reaction products distribute themselves homogeneously throughout the space originally filled by the mixing water. In particular, the hydration of C_3A rapidly produces ettringite. Ettringite forms as needle-like crystal which facilitates the conversion concrete from a plastic to a hardened state and ettringite formation significantly contributes to setting. This is occurring to a large extent during a period in which there is not extensive hydration of C_3S.

As C_3S hydration advances, continued infilling of the water-filled space by C–S–H and $Ca(OH)_2$ occurs, eventually resulting a continuous but porous solid. It is during this period that the previously formed ettringite is converted to monosulfate that becomes highly intermixed with the C–S–H. While details regarding the structure of the C–S–H continue to be debated, it is generally viewed as being formed of irregular layers of solid (gel) with nanometer-scale spaces in between filled with water (gel porosity). The distribution of the gel defines the larger remaining porosity, that was originally filled with water, as capillary porosity. Continued hydration both reduces the volume of capillary porosity while increasing its tortuosity thereby reducing the permeability of concrete. The slow hydration of C_2S results in the further increase in strength and reduction in permeability. The pozzolanic reaction contributes to longer-term properties as well.

Volume relationships

In addition to the chemical changes that occur with hydration, hydration is also responsible for the physical changes which affect property development. When describing concrete it is more usual to consider curing than hydration. Proper curing ensures that sufficient water is available to the concrete to support ongoing hydration. Calculation of the proportion of water needed on a weight ratio basis to completely hydrate a typical cement would produce a water-to-cement (w/c) ratio of about 0.26. Because concretes are not normally produced at this w/c ratio, residual porosity is present. This would suggest the concrete constituted at this w/c ratio would produce a solid free of porosity. However, even concrete produced at $w/c = 0.26$ would contain some porosity. Porosity is conferred to concrete in three ways. First, any water not consumed in hydration will produce capillary porosity; second there is gel porosity inherent in C–S–H; and, third, porosity will develop as a result of a process called chemical shrinkage. Capillary porosity is that which remains in the spaces in the fresh concrete originally occupied by water. As hydration proceeds, this space, along with that created as the anhydrous cement grains are consumed, becomes occupied by hydration products. The hydration products in mature concrete are primarily calcium hydroxide, which tends to form preferentially near the surfaces of the aggregate particle, and C–S–H in which monosulfate is dispersed. In well-mixed concrete these hydration products distribute themselves as ribbons of paste between aggregate particles, typically ranging from 100 to 200 µm in thickness. Any of this space which remains unfilled by a hydration product is capillary porosity and it is this type of porosity which controls concrete permeability. A typical variation in capillary porosity with w/c ratio is shown in **Table 1**.

When the w/c ratio is 0.70, even complete hydration is regarded as incapable for segmenting the capillary porosity as a means to reduce permeability.

A third mechanism that results in porosity is associated with the volume changes that occur with hydration. While the conversion of C_3S to C–S–H and $Ca(OH)_2$ according to results an increase in the volume solids present, the total volume decreases. In particular $V(C_3S) + V(H_2O) > V(Ca(OH)_2) + V(C\text{–}S\text{–}H)$. This volume decrease with hydration is called the chemical shrinkage and it occurs because the hydration products occupy less volume than did the reactants. Chemical shrinkage can manifest in two ways: formation of porosity within the concrete or a slight contraction of the concrete element. Thus because chemical shrinkage is inherent in hydration, this analysis shows the importance in minimising the w/c ratio in order to minimise capillary porosity.

w/c ratio by mass	Vol % capillary porosity
0.40	8
0.45	14
0.50	19
0.55	24
0.60	28
0.65	32
0.70	35

Table 1 Variation in capillary porosity with w/c ratio

Maturity

While it is these chemical events that result in strength gain, there is not a linear relationship between the extent of hydration and the extent of the strength gain. During early hydration there is significant consumption of anhydrous cement without meaningful strength development. Only when the hydration products have formed a continuous solid between the aggregate particles does meaningful strength occur. These events are typically characterised in terms of initial set, final set and the 28-day strength. While the 28-day strength will likely remain an important design parameter, it is based on the assumption that the concrete is being cured at about $\pm 8^\circ$C room temperature. However, the hydration phenomena leading to strength development are strongly dependent on temperature and ambient temperature variations may significantly affect the strength gain characteristics of field concrete. The understanding of the relationship between curing time and temperature has led to the maturity concept. This concept is based on the premise that: concrete strength increases with hydration, the rate of hydration increases with increasing temperature, strength can be expressed as a function of a time–temperature combination, and maturity is $\sum$(time interval × temperature). Maturity is expressed in °C-hrs or °C-days and temperature in this context is the difference between the actual temperature and -10° (the datum temperature) at which hydration is regarded to stop. Various equations have been derived to express maturity and compressive strength which is predicted based on a plot of the log of the maturity parameter as the abscissa against compressive strength as the ordinate (Neville, 1996).

Miscellaneous cements

Belite cements

Portland cements are mature industrial commodities, their chemical and physical characteristics are the subjects of various codes and standards, and production of Portland cement is capital intensive. These factors suggest that significant changes in Portland cements will only be realised with difficulty. For these reasons, changes in cementitious materials used in concrete have been evolutionary and can be categorised as follows: increased C_3S contents, finer grinding and the use of mineral and chemical admixtures. However, one change of great potential significance would be the commercial production of belite cements. Belite cement production would produce less CO_2 as only 2 moles of $CaCO_3$ are required to produce a mole of belite, while 3 moles are required to produce a mole of alite. Belite production also offers the opportunity to save energy by reducing kiln temperatures. Because inherently less $Ca(OH)_2$ is produced during hydration, equation (2), as compared with equation (1), changes in the use of mineral admixtures would be anticipated and an altered overall rate of hydration would affect maturity calculations and concrete practice in general. The challenges in realising concretes based on belite cements is both in the identification of means to accelerate its rate of hydration and in facilitating changes to relevant standards.

Shrinkage-compensating, and regulated set cements (ACI, 1970)

In a Portland cement the amount of calcium sulfate interground with the clinker is proportioned to preclude the ettringite formation after about the first day of hydration. Although the expansive reaction associated with ettringite formation can be accommodated while concrete is in a plastic state, the occurrence of an expansive reaction in hardened concrete used in ordinary construction is undesirable. There are, however, exceptions to this and cements have been formulated based, in part, on the formation of ettringite. These cements fall into two general categories: shrinkage-compensating and regulated set cements.

Shrinkage-compensating cements are formulated such that an expansive reaction compensates for the early shrinkage typical in Portland cement concretes. In cements of this type, ettringite is formed as a result of the reaction between gypsum and the compound $4CaO{\cdot}3Al_2O_3{\cdot}SO_3$ according to the reaction:

$$4CaO{\cdot}3Al_2O_3{\cdot}SO_3 + 8CaSO_4{\cdot}2H_2O + 6CaO + 80H_2O \rightarrow 3(3CaO{\cdot}Al_2O_3{\cdot}3CaSO_4{\cdot}32H_2O) \quad (6)$$

Shrinkage-compensating cements are also referred to as expansive cements or self-stressing cements.

Regulated set cements contain the compounds that react rapidly with sources of calcium sulfate to form ettringite. The early strength gain in regulated set cements is associated with ettringite formation. After ettringite formation has reached virtual completion, reaction of the silicate phases results in continued strength gain. In one class of regulated set cements, $11CaO{\cdot}7Al_2O_3{\cdot}CaF_2$ is added. This compound is rapidly hydraulic and ettringite is rapidly formed when a source of calcium sulfate is present. In a second class, the compound $4CaO{\cdot}3Al_2O_3{\cdot}SO_3$ is present but in higher proportion than that used in shrinkage-compensating cements.

High-alumina cement (Neville and Wainwright, 1975)

High-alumina cement is a non-Portland cement composed essentially of monocalcium aluminate, $CaO{\cdot}Al_2O_3$. The hydration reaction in this cement is:

$$CaO{\cdot}Al_2O_3 + 10H_2O \rightarrow CaO{\cdot}Al_2O_3{\cdot}10H_2O \quad (7)$$

High-alumina cement offers the desirable features of high early strength and resistance to chemical attack. The source of resistance to chemical attack is that the calcium aluminate hydrate formed is of low basicity and is resistant to attack by mild acids.

High alumina cements perform adequately providing the exposure condition does not promote the conversion of the monocalcium aluminate hydrate to tricalcium aluminate hexahydrate. This conversion reaction may be written as follows:

$$3(CaO{\cdot}Al_2O_3{\cdot}10H_2O) \rightarrow 3CaO{\cdot}Al_2O_3{\cdot}6H_2O + 2Al_2O_3{\cdot}3H_2O + 18H_2O \quad (8)$$

and results in a significant reduction in the total volume of the solids present. The monocalcium aluminate hydrate phase becomes increasingly unstable with respect to tricalcium aluminate hexahydrate as the temperature is increased above room temperature. This conversion reaction leads to a loss of strength that has resulted in structural failures (BRE, 1975).

Other cements

A variety of cements used for specialised applications, such as oil well cements, refractory cements, etc., have not been considered. In addition, the above discussion of shrinkage-compensating, regulated set, and high-alumina cements is cursory and reference may be made to a variety of sources regarding these types of cements (e.g. Bensted, 1983; Ramachandran *et al.*, 1981).

Concrete mix designs and the opportunity for innovation

There is large body of reference material intended to aid in the design of concrete mixes (e.g. *ACI*, 2006; Kosmatka *et al.*, 2002; *Concrete Manual*, 1998) and various sources of commentary on the standard tests used to evaluate cements, fresh concrete and hardened concrete (Lamond and Pielert, 2006). In designing a concrete mixture for a particular application, three general aspects of a mix design can be controlled. These are compositional control, permeability control, and strength control. Collectively, these parameters determine both the short-term and long-term performance of a concrete and each of them is influenced by hydration phenomena.

At present it is common for prescription to govern mix designs. Various code organisations may specify maximum w/c materials ratios, minimum 28-day strengths, minimum cement contents and the like. It has been argued that a prescriptive specification serves to ensure that a concrete will adequately serve its intended function. Alternatively, it is being argued that concretes are routinely overspecified, which leads to economic waste, and that prescriptive specifications stifle innovation. This latter argument is the basis for evolving performance-based specifications that allow the specifier far more latitude in designing a mix that will be optimal for its purpose both in terms of cost and performance.

Further reading and reference

In providing citations for the topics in this section, reference has been made to standard texts and overview publications, publications by trade associations and publications by the voluntary consensus organisation ASTM. Depending on specific interests, trade associations and technical societies produce a variety of publications. Examples of these include Special Technical Publications (STPs) by ASTM and Special Publications (SPs) by ACI, and topical publications such as those produced by the Materials Science of Concrete and by the Materials Research Society.

Regarding sources cited herein, reference to Bogue and Lee in particular, while published over 50 and 35 years ago, respectively, is not coincidental; these references remain relevant and cite much early literature which is still the basis for presently accepted concepts and practice. References (*ASTM*, 2006a, 2006b; *ACI*, 2006) are revised annually and the five volumes of *ACI Manual of Concrete Practice* (2006) cover a broad range of topics relevant to concrete technology. References (Kosmatka *et al.*, 2002; Lamond and Pielert, 2006) are periodically updated and provide valuable reference to changes in concrete practice and in standard tests for concrete and concrete materials.

In addition to the above sources, a variety of journals publish on topics related to hydration and associated changes in physical and chemical properties of concrete. Representative examples of these are *Cement and Concrete Research*, *Advances in Cement Research*, *ACI Materials Journal*, *Concrete International*, *Magazine of Concrete Research*, *Journal of Materials in Civil Engineering*, *Cement and Concrete Composites*, *Journal of the American Ceramic Society*, *Materials and Structures*, and *Transportation Research Record*.

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Chapter 8

Aggregates for concrete

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CONTENTS

The Standard definition of aggregate is a granular material used in construction; it may be natural, manufactured or recycled. Typically, concrete is comprised of 70% aggregate and the remaining 30% is water, air and cementitious material. However, it is true to say that the fundamental characteristics of concrete are greatly influenced by the physical and chemical properties of the aggregate. There are two factors which determine these characteristics: first, the geological origin and deposition of the material; and second, the physical processing by the aggregate producer. This chapter considers the geological classification of aggregates and the physical and chemical properties resulting from formation and deposition, properties which result from processing into a concrete aggregate and how size, shape, texture, strength and chemical stability impact the properties of concrete and its durability. The chapter also considers lightweight and heavyweight aggregates, both natural and manufactured and recycled aggregates.

Geological origin and deposition

Rocks are divided into three groups defined by the particular mode of deposition:

1. igneous rocks formed by solidification of the earth's internal molten magma;
2. sedimentary rocks formed by accumulation and deposition of fragmentary materials or organic remains; and
3. metamorphic rocks formed by alteration of existing rocks by heat, pressure or other processes in the earth's crust.

A full explanation of the formation and deposition of these rocks can be found at the British geological society website: http://www.geolsoc.org.uk/rockcycle.

Igneous rocks

Igneous rocks fall into two categories intrusive and extrusive. Intrusive rocks are further divided into large intrusions (plutonic) and small intrusions (hypabyssal). Plutonic rocks cool slowly and therefore have a larger grain structure. Hypabyssal rocks cool more quickly and have an intermediate grain size 0.5–2.0 mm. Extrusive or volcanic rocks cool very quickly and are fine grained, less than 0.5 mm and may be amorphous or glassy. Intrusive igneous rocks are subdivided into acidic, intermediate, basic and ultra-basic melts, the acidic melts being material from the continental crust and the more basic rocks are from the mantle beneath the oceanic crust. Acidic melts are rich in silica (greater than 65%), have densities in the range of 2.4 to 2.7 and contain light-coloured (leucratic) minerals, these form the granite series of rocks. Basic melts have lower silica content, typically 45–55%, have densities in the range of 2.9–3.0 and contain dark-coloured [melanocratic] minerals, these are the gabbro series of rocks. Rocks which fall between acidic and basic are described as intermediate and have densities, silica content and colour in the middle of the ranges. **Table 1** shows the classification and properties of igneous rocks. These types of rock are quarried extensively for use in construction and in particular for producing asphalt and concrete aggregate.

Igneous rock types generally make good concrete aggregate. This is because the extraction and processing methods result in angular-shaped material thus providing aggregate interlock. The rough crystalline texture enables strong bonds between the coarse particles and the sand–cement paste and the crystalline characteristics of these rocks provide hard durable aggregate.

Sedimentary rocks

Sedimentary rocks are usually classified into the following three groups (**Table 2**) depending on the mode of deposition:

1. clastic sediments derived from erosion, transport and deposition of existing rocks;
2. chemical sediments produced by precipitation, particularly in super-saturated seas and lakes;
3. organic or biochemical sediments.

Clastic sediments

Clastic sediments include sandstones, shale and conglomerates in addition to unlithified deposits of sands and gravels.

	Acidic	Intermediate	Basic	Ultra basic	Occurrence
% Silica	>65	55–65	45–55	<45	
Quartz content	High	Low	Very low	None	
Colour	Pale	Medium	Dark	Very dark	
Density	2.4–2.7	2.7–2.8	2.8–3.0	3.0	
Coarse grained >2 mm	Granite	Diorite	Gabbro	Peridotite	Batholiths and large sills
Medium grained 2–0.5 mm	Micro-granite	Micro-diorite	Dolerite		Laccoliths, sills and dykes
Fine grained <0.5 mm	Rhyolite	Andesite	Basalt		Lava flows and small dykes

Table 1 Classification and properties of igneous rocks

The latter deposits have been a common source of concrete aggregates for many years, partly due to the ease with which they can be won and processed.

Fluvial action over time leaves river terrace and flood-plain deposits comprised of materials eroded from the upper reaches of the river. The particle size of the material reduces with the distance from the river's source and flood-plain deposits tend to be quite shallow but spread over a wide area. These alluvial deposits are usually well sorted in particle size, clean and rounded to irregular in shape.

Glacial deposits were formed as a result of physical erosion by moving ice sheets. The material transported by the ice was deposited as the ice melted during warm periods in between ice ages. Glacial deposits can be variable in shape, have a high clay and silt content, can be angular in shape and include a higher proportion of weathered rock particles. In contrast, materials transported away by melt water are described as fluvio-glacial deposits; these are characteristically more similar to alluvial sand and gravel deposits.

Alluvial deposits of well-sorted rounded sands and gravels are also found on the sea bed, having been deposited at times when the sea level was significantly lower than today. These marine deposits are also exploited for concrete aggregates.

Material deposited in river deltas tends to be fine-grained sediments and over time these produce sandstones, mudstones and siltstones. As the sea depth increases, the sediments become finer. A rock type frequently used as an aggregate and formed in this environment is greywacke, a type of sandstone having angular grains and a proportion of the mineral feldspar. Clastic rock types are not always the most suitable for concrete aggregate since they can be porous and have high water absorption, consequently durability may be affected.

Chemical sediments

These are formed from chemical precipitation in lakes and seas when the water is in supersaturated condition; the rock types include some limestones, gypsum, rock salt and chert.

Organic or biochemical sediments

These deposits are principally limestone formed by the deposition of calcium carbonate, the rock having dense, durable crystalline structure. Limestones formed from the biochemical deposition of skeletal remains do not make viable concrete aggregates due to high porosity; limestone formed from coral reefs and shell also falls in to this category. Limestone can vary in colour from dark grey to white depending on the inclusion of other minerals;

	Clastic			Organic	Chemical
	Grain size	Unlithified	Lithified		
Rudaceous	>200 mm 60–200 mm 2–60 mm	Boulders Cobbles Gravel	Conglomerate	Calcareous Limestone	Calcareous Limestone Dolomite
Aranaceous	0.6–2 mm 0.2–0.6 mm 0.05–0.2 mm	Coarse sand Medium sand Fine sand	Sandstone	Carbonaceous Coal	Gypsum Ironstone
Argillaceous	0.02–0.06 mm <0.02 mm	Silt Clay	Siltstone Shale Mudstone	Siliceous Chert Diatomaceous earth	Siliceous Chert

Table 2 Simplified classification of sedimentary deposits

Pressure	Dynamic mode	Regional mode	Contact mode	Heat
Low ⟶		Slate		⟵ Low
Moderate ⟶		Schist	Marble	⟵ Moderate
High ⟶	Mylonite	Gneiss Granulite	Hornfels Quartzite	⟵ High

Table 3 Metamorphic rock formation and types

however, it can be simply identified by immersion in dilute hydrochloric acid which results in some effervescence. Flint is also formed by chemical sedimentation and occurs in bands or nodules in chalk deposits. Erosion of soft chalk over time leaves deposits of sand and gravel, principally comprising flint. Such deposits are relatively widespread in the south-east of England.

Metamorphic rocks

Metamorphic rocks are formed when existing rock formations are subjected to heat and/or pressure; these forces alter the composition and texture of the rocks. The mode of formation can be divided into the following three simple groups (**Table 3**):

1. dynamic metamorphism
2. regional metamorphism
3. contact metamorphism.

Dynamic metamorphism is the action of pressure only; as the pressure increases, hard-wearing fine-grained rocks are formed. Changes in texture are due to the realignment of grains along the plane of pressure and this may produce flaky aggregates that break down easily and have high water demand in concrete.

Regional metamorphism is the action of both pressure and heat together; the severity of the combined forces determines the resulting rock type. Contact metamorphism is the action of heat only; the intense heat fuses the rock particles together although, overall, the structure remains the same.

It can be seen from **Table 3** that when mudstones are subjected to increasing heat and pressure the rocks formed are: slate, with flaky layers oriented at right angles to the applied pressure; schist, where the flaky layers are being changed by increasing pressure and new minerals are forming due to the heat; gneiss, where most of the previous texture has now gone and rock grains are beginning to fuse together; and granulite, where none of the original texture remains and grains are totally fused together.

Rock types commonly used for aggregates

Table 4 (BS 812-102: 1989) lists the rock types commonly used for aggregates. Some aggregates may require a cautious approach if there is any indication that they may be susceptible or potentially susceptible to alkali–silica reaction (ASR); in particular siliceous aggregates have been identified in the majority of confirmed cases of ASR in the UK. However, the history of use of a particular source of aggregate should give a safe indication of suitability for making concrete and other measures can be taken such as reducing the total alkali content of the concrete (BRE Digest 330: 2004).

Extraction and processing of natural aggregates

The method of extraction is always dependent on the type of rock and location of the deposit to be worked. Sand and gravel deposits may be dug or dredged if the deposit lies under water or is marine based while rock deposits usually require blasting. Actual processing methods are both similar and different because while rock extraction may require extensive crushing, this is not usually the case with sand and gravels except as a secondary process for oversize material.

Sand and gravel

The nature of the deposit to be worked will determine the type of plant required. Deposits that are principally sand generally only require systems to separate coarse and fine sands and remove materials such as lignite or other organic contaminants. Further, where deposits are principally clean sand that is free of clay seams, for some applications sand is only dry screened and not subjected to any washing or silt removal process.

Wet processing of sand is normally done through gravitational hydraulic classifiers or centrifugal hydraulic (cyclonic) classifiers, these types of plants are widely used in sand processing operations. This process is known as elutriation and is defined as separation of coarse and fine particles by washing, the principle being that if a particle is falling at a particular velocity through water and an upward current has a higher velocity than that rate, the particle will be carried upwards to the overflow. Both types of plant rely on the behaviour of particles in a water stream. In the first case the degree of drag and terminal velocity is affected by the particle size and a vertically rising water stream is used to cause finer fractions to overflow a weir and then directed to a silt lagoon. The coarser fraction is then collected from the system by helical rakes or drag bucket system and is then fed to stockpile via a de-watering system. This type of plant can be used to make sands of varying grades of fineness. The cyclonic type of plant works on very similar principles: the plant creates a vortex within the vessel by pumping the feed of raw material and water into the system at the required pressure and again the finer

Petrological term	Description
Andesite[a]	A fine-grained, usually volcanic, variety of diorite
Arkose	A type of sandstone or gritstone containing over 25% feldspar
Basalt	A fine-grained basic rock, similar in composition to gabbro, usually volcanic
Breccia[b]	Rock consisting of angular, unworn rock fragments, bonded by natural cement
Chalk	A very fine-grained cretaceous limestone, usually white
Chert	Cryptocrystalline silica
Conglomerate[b]	Rock consisting of rounded pebbles bonded by natural cement
Diorite	An intermediate plutonic rock, consisting mainly of plagioclase, with hornblende, augite or biotite
Dolerite	A basic rock, with grain size intermediate between that of gabbro and basalt
Dolomite	A rock or mineral composed of calcium magnesium carbonate
Flint	Cryptocrystalline silica originating as nodules or layers in chalk
Gabbro	A coarse-grained, basic, plutonic rock, consisting essentially of calcic plagioclase and pyroxene, sometimes with olivine
Gneiss	A banded rock produced by intense metamorphic conditions
Granite	An acidic, plutonic rock, consisting essentially of alkali feldspars and quartz
Granulite	A metamorphic rock with granular texture and no preferred orientation of the minerals
Greywacke	An impure type of sandstone or gritstone, composed of poorly sorted fragments of quartz, other minerals and rocks; the coarser grains are usually strongly cemented in a fine matrix
Gritstone	A sandstone with coarse usually angular grains
Hornfels	A thermally metamorphosed rock containing substantial amounts of rock-forming silicate minerals
Limestone	A sedimentary rock, consisting predominantly of calcium carbonate
Marble	A metamorphosed limestone
Microgranite[a]	An acidic rock with grain size intermediate between that of granite and rhyolite
Quartzite	A metamorphic or sedimentary rock, almost entirely composed of quartz grains
Rhyolite[a]	A fine-grained or glassy acidic rock, usually volcanic
Sandstone	A sedimentary rock composed of sand grains naturally cemented together
Schist	A metamorphic rock in which the minerals are arranged in nearly parallel bands or layers. Plate-like or elongate minerals such as mica or hornblende cause fissility in the rock which distinguishes it from gneiss
Slate	A rock derived from argillaceous sediments or volcanic ash by metamorphism, characterised by cleavage planes independent of the original stratification
Syenite	An intermediate plutonic rock consisting mainly of alkali feldspar with plagioclase, hornblende, biotite or augite
Trachyte[a]	A fine-grained, usually volcanic, variety of syenite
Tuff	Consolidated volcanic ash

Notes:

[a] The terms microgranite, rhyolite, andesite and trachyte, as appropriate, are preferred for rocks alternatively described as porphyry or felsite.

[b] Some terms refer to structure or texture only, e.g. breccia or conglomerate and these terms cannot be used alone to provide a full description.

[c] Composed of crystals so fine that they can be resolved only with the aid of a high-power microscope.

Table 4 Rock types commonly used for aggregates (BS 812-102: 1989) (reproduced with kind permission from BSI)

particles are carried by a rising water current to the vortex finder while the coarser material falls in a downward current.

Where the deposit also has a significant proportion of gravel; more intensive processing is required, the extracted material is passed over a grizzly screen to remove oversize material then put through a washing process, first to break down agglomerates and separate the sand, material finer than 4–5 mm in dimension, from the coarse material. The fine material is then processed as described above to remove deleterious materials and make the required grades of sand. The gravel is then passed over a screen deck to remove oversize particles and separate the rest into single-size material; this process may include further washing to remove clay and silt. The oversized material is then returned through the process via a tertiary crusher as shown in **Figures 1(a)**, **1(b)** and **1(c)**. It should be noted that sea-dredged material is generally clean, and the main reason for a washing plant is to reduce the chloride level in the aggregates to within recognised acceptable limits.

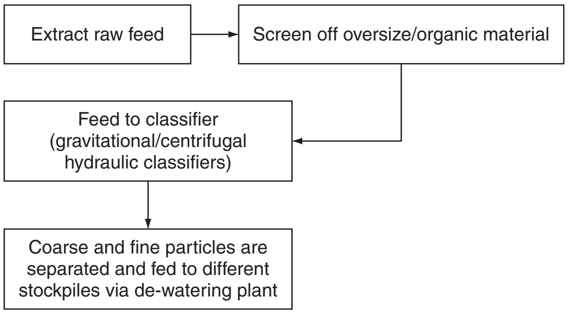

Figure 1(a) Flow diagrams of typical processes for sand and gravel production (land based – principally sand)

Crushed rock aggregate

The production of crushed rock aggregate is a more energy-intensive process and requires a significantly higher capital investment than a sand and gravel operation. The flow diagram for a basic crushed rock quarry is shown in **Figure 2**. The term for the reduction of blasted rock, or gravel, to usable sizes is communition; the resulting aggregate and its properties are influenced by the natural properties of the rock and also the type of crusher(s) used.

The primary crusher is usually either a jaw or a gyratory type, its principal purpose being to reduce material to a more transportable size for the secondary circuit. In limestone quarries impact crushers may be used but not for hard rock since maintenance costs would be too high.

Secondary crushers can be jaw, cone or impact types and the decision is normally dependent on the hardness of the stone. Both jaw and cone types will produce aggregates with a higher flakiness index when compared to impact crushers; the latter tends to produce aggregates of ideal angular shape. The actual equipment choice will depend almost entirely on the end uses of the crushed rock and also the economic viability of the operation; certainly a

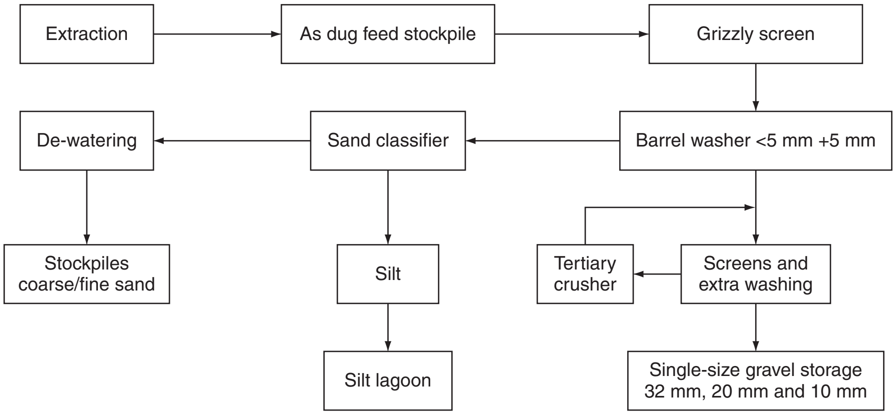

Figure 1(b) Flow diagrams of typical processes for sand and gravel production (land based – sand and gravel)

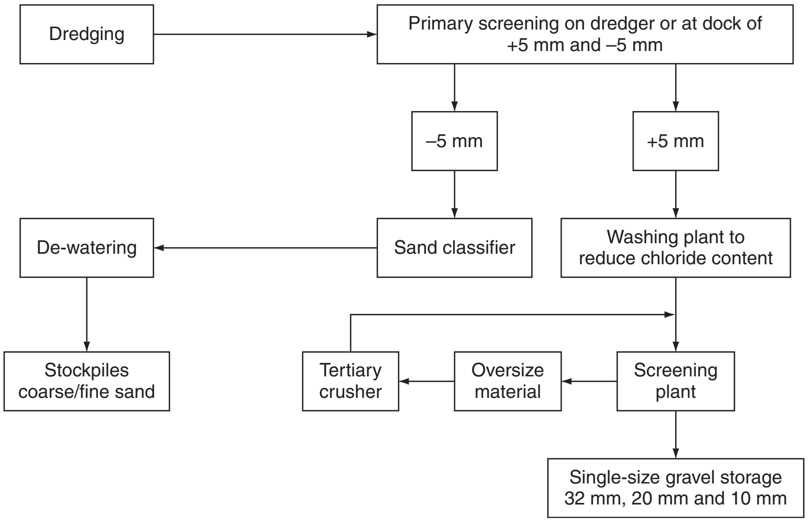

Figure 1c Flow diagrams of typical processes for sand and gravel production (sea-dredged sand and gravel)

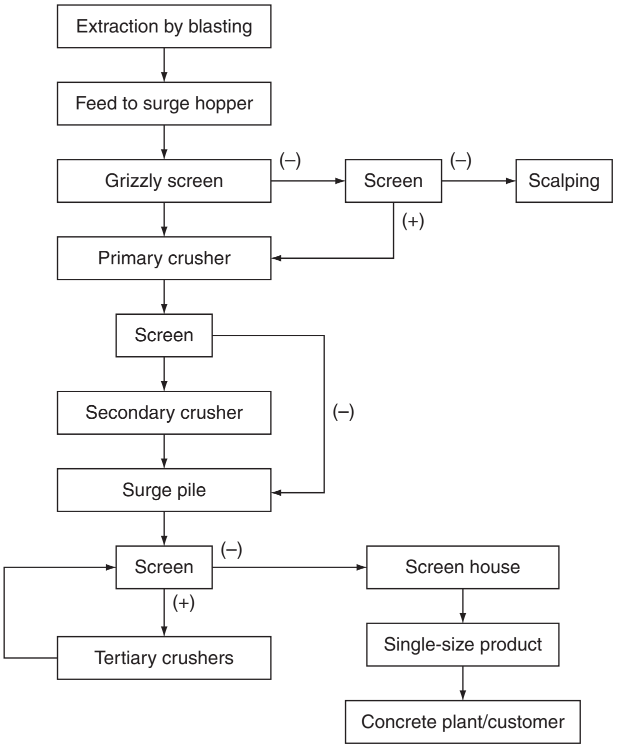

Figure 2 Flow diagram for a basic crushed rock quarry

more angular shape allows concrete design to be more economic – generally as flakiness reduces then so does the cementitious content necessary to produce a given strength or to meet a specified water/cement (w/c) ratio.

As with all quarrying operations sustainability both economic and environmental, requires that as much of the extracted material as possible is used for downstream products: for every tonne of coarse aggregate produced there is also a significant quantity of crushed rock fines. The latter material is now widely used in concrete to replace or reduce natural sand content; substitution up to 100% may be possible dependent on the end use of the concrete. It has been found that these materials can significantly increase strength for a given cementitious content due to the densification of the concrete matrix by the fine fraction (particles less than 0.063 mm in maximum dimension).

Influence of aggregate characteristics on properties of concrete

Apart from affecting the fresh and hardened properties of concrete, aggregates strongly influence its mix proportions and economy. There are various characteristics of aggregates. BS EN 12620 + A1: 2008 specifies the characteristics of aggregates and filler aggregates obtained by processing natural, manufactured or recycled materials and mixes of these materials for use in concrete. The characteristics of aggregates discussed in this section are those that are of prime importance in concrete production.

Shape and surface texture

Particle shape and surface texture of aggregates can have marked effects on the properties of both fresh and hardened concrete (Mehta and Monteiro, 2005). However, shape and surface texture influence the properties of freshly mixed concrete more than the properties of hardened concrete. These effects could be beneficial where the predominant particle shape is equi-dimensional and detrimental where it is flaky and/or elongated (Neville, 1996). Flakiness of coarse aggregate can have an adverse influence on the workability and mobility of concrete and may cause blockage in pump pipelines due to particle interference. Experiments have shown that the workability of concrete as measured by the compacting factor is reduced by 10% if the shape of the particles is changed from rounded to angular. The strength of concrete also decreases with the increase in aggregate flakiness, with flexural strength being more affected than the compressive strength. Flakiness in fine aggregate can lead to problems of 'bleeding' (Mehta and Monteiro, 2005) and segregation in the fresh mix leading to reduced strength and durability of the resultant hardened concrete.

The main influence of surface texture is on the bond between the aggregate and the cement paste in hardened concrete. Surface texture is generally considered to have greater influence on concrete flexural strengths, which are frequently found to reduce with increasing particle smoothness. However, inadequate surface texture can also adversely affect the compressive strength in high-strength concretes (strength >50 MPa) (Neville, 1996).

Grading and maximum size

Grading refers to the particle-size distribution of aggregates. Grading limits and maximum aggregate size are specified because grading and size affect the amount of aggregate used as well as cement and water requirements, workability, pumpability, and durability of concrete. In general, if the w/c ratio is chosen correctly, a wide range in grading can be used without a major effect on strength. Where concrete of particularly high strength, uniformity or aesthetic quality is required, it is usually beneficial to specify that single-sized coarse aggregates be used. When gap-graded aggregate are specified, certain particle sizes of aggregate are omitted from the size continuum. Gap-graded aggregates are used to obtain uniform textures in exposed aggregate concrete. In general, continuously graded aggregates tend to provide relatively straightforward and easy-to-place concretes, whereas gap-graded concretes usually require appreciably more expertise in mix design, production, handling and placing (Collepardi,

Material	Impact on fresh concrete	Impact on hardened concrete
Lignite	Normally low but some lignite/organic materials may locally retard the setting of Portland cements	Mainly visual but on flat work with time, surface pop-outs may occur and affect surface durability
Iron pyrites	None	Brown staining on vertical cast surfaces may be visually unwelcome but have no effect on durability
Shale	Cause water demand to increase for a given consistence value	Reduced strength
Clay agglomerates	Increased water demand	May shrink on drying and leave surface cavities; reduced bond between coarse aggregate and sand/cement paste
Chlorides	None	Increased risk of corrosion damage to reinforcement

Table 5 Deleterious materials in aggregates

2006). For these reasons, the labour costs of placing gap-graded concretes may sometimes be greater, but this may be justified where their special properties can be used to advantage; for example, where high mechanical stability is required immediately after compaction, or where increased resistance to scour is required in inter-tidal work.

The maximum size of aggregate permitted by BS EN 12620 + A1: 2008 for use in concrete is 40 mm. However, the size most frequently used is 20 mm since consideration has to be given to the cover and bar spacing in reinforced concrete. In some cases 10 mm maximum size aggregate may be used, especially when the structure size is limited or the preference is for a very cohesive concrete for use in a slip-form operation. Although 40 mm maximum size is permitted in pavement quality concrete, it is normally used in the bottom of the two-layer construction with 20 mm aggregate used in the top layer. The concrete placing and finishing method to be employed also plays a very important part in deciding the maximum size to use. Of prime importance is the consistency of the maximum size within the limits of BS EN 12620 + A1: 2008 since a change can influence the water demand of concrete and, hence, affect both the strength and the durability.

Moisture content

The moisture content in aggregate is one of the most important parameters that can affect the quality of concrete in terms of workability, compressive strength, porosity and durability (Neville, 1996; Mehta and Monteiro, 2005; Collepardi, 2006). The moisture content in aggregate affects the mixing water content in concrete in two ways:

1. If the moisture content of aggregate is higher than the saturated surface dry (SSD) condition, then the amount of mixing water in the mix is reduced.
2. If the moisture content of aggregate is lower than the SSD condition, then the amount of mixing water is increased.

In order to avoid changes in the performance of concrete due to the variation in the moisture content in aggregates, it is important to make the necessary adjustments in water content. This can be done by either subtracting or adding water to the mix.

Resistance to fragmentation

It is important that aggregates are strong enough to avoid breakdown due to physical attrition while handling and during the concrete production process. The concrete Standard BS 8500-2: 2006, has a default maximum Los Angeles Index, as classified in BS EN 12620 + A1: 2008, of LA_{40} but also provides a route for establishment of suitability for aggregates having a higher value by trials or history of satisfactory use.

Deleterious material

There are some substances and materials that occur in natural deposits that have a significant influence on concrete properties and durability (refer to **Table 5**). Aggregate producers have always tried to minimise the presence of deleterious materials in processed aggregates but it is impossible to be certain that all materials will be removed; in particular lignite (brown coal) and other organic matter is present in many sand and gravel deposits and it is almost impossible to remove it entirely (Neville, 1996).

Lightweight and heavyweight aggregates

Natural mineral aggregates such as sand and gravel have a bulk density of 1520–1680 kg/m^3 and they produce normal-weight concrete with a density of approximately 2400 kg/m^3. However, for special purposes, aggregates with lighter or heavier density can be used to make correspondingly lightweight and heavyweight concretes. Generally, the aggregates with bulk densities less than 1120 kg/m^3 are called lightweight, and those having densities more than 2080 kg/m^3 are called heavyweight.

Lightweight concretes are defined as those having an oven-dry density of less than 2000 kg/m^3 and heavyweight concretes are those having an oven-dry density greater than 2600 kg/m^3 (BS EN 206-1: 2000).

Both lightweight and heavyweight aggregates may be simply classified into natural or artificial aggregates. Pumice is an example of a natural lightweight aggregate and is used mostly in the production of concrete blocks. Expanded clay, shale or slate, sintered pulverised fuel ash and foamed blastfurnace slag are all artificial lightweight aggregates. They are all capable of producing concrete strengths in excess of 25 MPa with densities ranging from 1500–2000 kg/m^3. Lightweight aggregates are used in a variety of precast products, such as lintels, kerbs, panels, stairs and bespoke units. They can also be used for structural applications, with strengths equivalent to normal-weight concrete. The benefits of using lightweight aggregate concrete include (Mindess *et al.,* 2002; Lamond and Pielert, 2006):

- reduction in dead loads, making savings in foundations and reinforcement
- improved thermal properties
- improved fire resistance
- savings in transporting and handling precast units on site
- reduction in formwork and propping.

For heavyweight aggregates, the most widely used natural materials are barytes and the iron ores such as haematite and magnetite, whereas the artificial materials are based on iron shot or lead shot. Heavyweight concretes are used for the prevention of seepage from radioactive structures and X-ray equipment in hospitals to avoid the harmful effect of radiation on the human body and living organisms; they are also widely used for concrete gravity blankets on undersea pipes and for concrete counterweights.

Recycled aggregates

Recycled aggregates are produced from the processing of previously used construction materials, such as concrete or masonry. Recycled concrete is a viable source of aggregate and can be satisfactorily used in granular sub-bases, soil–cement and in new concrete. In many situations, recycled aggregates can produce concrete that is as strong as concrete made with natural aggregates. However, this depends on the specified strength of the concrete, the source of the recycled aggregate and the proportion of recycled materials used to replace the natural aggregates. If concrete is not exposed to a severe environment (such as seawater or freezing and thawing), concrete containing recycled aggregates can provide a similar level of durability to concrete containing natural aggregates.

The Standard for recycled aggregate for use in concrete is now BS EN 12620: 2000 + A1: 2008, this standard defines them as ‘aggregate resulting from the processing of material previously used in construction’. BS 8500-2: 2006 permits the use of recycled coarse aggregate in a wide range of concretes and generally conformance with the above aggregate Standard confirms acceptability. The concrete Standard specifies the limits on the permitted composition of recycled coarse aggregates as well as giving guidance on their use in.

Concluding remarks

Concrete normally contains less than 30% binder, while the rest is primarily aggregates. Therefore, their properties affect the properties of concrete. However, the focus of concrete science and technology in the last half century has mostly been on binder component. As aggregates play an important role, to confer better dimensional stability and wear resistance to concrete in addition to their use as an economical filler, there is a need to focus on the aggregate component of concrete in the coming decades; this applies particularly to more marginal materials and expanding the use of reclaimed and/or recycled aggregates.

Acknowledgement

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Chapter 9

Water in concrete

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Water is a very important ingredient in the manufacture of concrete. It is mainly used to impart workability to concrete and to hydrate the cementitious components. Water also has a significant role in almost all mechanisms of deterioration of concrete, by acting as a carrier for the transport of aggressive agents into the concrete and being involved directly in many forms of deterioration. That is, water which contributes to the creation of concrete is also, to a large extent, responsible for its destruction. Therefore, an understanding of the role of water in concrete is essential for choosing suitable mix proportions and ensuring durability. In this chapter, the role of water and various damages caused by it are briefly discussed. The appropriate standards are introduced to highlight tolerable concentrations of impurities in mixing water in concrete. As there is a scarcity of water in many parts of the world, a brief discussion on the use of seawater in concrete is also given in this chapter.

Role of water

Water plays an important role in the manufacture of concrete. It acts as a lubricant for cement, coarse aggregate, fine aggregate and some additives, so that these materials can mix completely and evenly. Too much water in concrete can reduce its strength, while too little water may produce an unworkable concrete. As concrete must be both strong and workable, a careful balance of the quantity of water added is required while making concrete. Although water provides plasticity to the concrete in its fresh state so that it can be poured in a form, its real importance is in the hardening process. Therefore, the role of water in concrete can be best described by discussing its importance in hydration and curing.

The water causes the hardening of concrete through a process called hydration. Hydration is a chemical reaction in which the major compounds in cement form chemical bonds with water molecules and become hydration products (Taylor, 1997; Bensted and Barnes, 2002). For the hydration to proceed smoothly, a certain amount of water is needed. The amount of water used is commonly described by the water/cement ratio (w/c), which indicates the mass of water added to the mass of cement used. Nowadays water/cementitious materials ratio (w/cm) is specified as concrete contains more than one type of binder material (Neville, 1996), but the discussion in this chapter is based on w/c. A minimum w/c of 0.25 is theoretically required to hydrate all the cement particles. However, a certain amount of water ($w/c = 0.15$) is physically absorbed by cement phases and as such is not available for hydration. Thus, a w/c of 0.40 represents the minimum water content to achieve full hydration (Neville, 1996). The w/c can be reduced to as low as 0.18 or 0.20 if superplasticisers are used (Paillere, 1994) to improve the rheological characteristics of concrete. In the context of this chapter, the role of superplasticisers and the interaction between water and superplasticisers are not discussed. If more water is added, the surplus amount is not used in the chemical reaction and remains as free water in the cementitious structure to form capillary pores. On the other hand, when the amount of water is low ($w/c < 0.40$, without any superplasticiser), sufficient quantity of water may not be available for all the cement particles to hydrate.

The continuous supply of water to concrete by means of curing is essential for the progress of hydration. In fact, after concrete is placed, an adequate moisture content and temperature must be maintained during the early ages so that the concrete can develop the desired properties for which it is designed (Newman and Choo, 2003; Mindess *et al.*, 2002). Curing has a strong influence on the properties of hardened concrete, such as strength, abrasion resistance, volume stability and resistance to both cyclic freezing and thawing and de-icing salts.

Concrete deterioration caused by water

Although water is an essential element in producing concrete, there are circumstances where it can also contribute to the deterioration of concrete. Two important effects of water in concrete are bleeding and drying shrinkage (Neville, 1996). The upward migration of water and its accumulation at the surface of concrete is called bleeding. This not only creates a placing problem but also reduces the strength and durability of concrete near the surface (Lamond and Pielert, 2006). Excessive bleeding on one hand delays the finishing, whereas, on the other hand, lack of bleed-water on concrete surfaces may lead to plastic shrinkage cracking. The loss of water from hardened concrete may lead to drying shrinkage

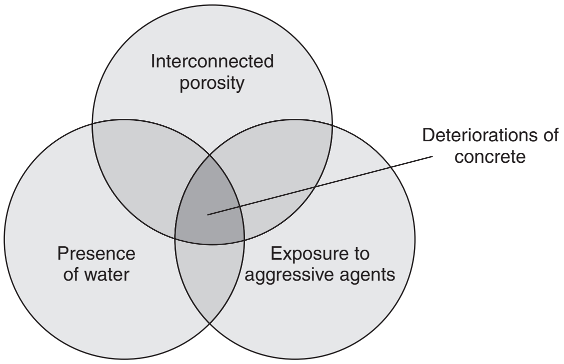

Figure 1 Ternary representation of concrete deterioration (adapted from Collepardi, 2006)

as well. Under certain circumstances, drying shrinkage can lead to shrinkage cracking, which is one of the most common causes of problems in concrete structures exposed to air (Neville, 1996).

Water also acts as a carrier for the transport of aggressive substances, such as salts, into the concrete. The deterioration of concrete can be attributed to the coexistence of three principal elements, viz., interconnected porosity, exposure to aggressive agents and presence of water (Collepardi, 2006). This is represented in **Figure 1**, where each element corresponds to a circle. When all the three elements are present, the area in the middle, where the three circles overlap, corresponds to the situations of a serious risk for different forms of deterioration. For instance, if corrosion of steel in concrete is considered, the interconnected porosity is responsible for the transport of carbon dioxide and chlorides to the level of rebars. Both carbon dioxide and chloride ions are transported through concrete due to their higher concentrations in the exposure environment. The corrosion of rebar occurs when oxygen and water are available at the rebar. Therefore, if water is not present, the corrosion reaction will not occur. That is, in general, deterioration will not happen in concretes which are not exposed to water, even if potential aggressive agents are present in the environment.

Some examples of concrete deterioration caused by water in concrete are (Neville, 1996; Mehta and Monteiro, 2006; Collepardi, 2006):

- Porous, water-saturated concrete having inadequate strength and entrained air is prone to scaling, which is a deterioration mechanism caused by freezing of water in concrete.
- Concrete containing alkali-reactive aggregates is subject to deleterious expansion due to water.
- Water passing over a concrete surface with a high velocity can erode the surface over time.
- The pore water in concrete acts as an electrolyte that permits the development of an electrochemical cell in the process of corrosion; that is, water is necessary for corrosion of reinforcement to proceed.
- Water is an essential element in the carbonation of concrete, which reduces the alkalinity of the hydrated cement paste and can lead to the corrosion of the reinforcement.

Impurities in water affecting concrete properties

Water needs to be pure in order to prevent side reactions from occurring, which may either weaken the concrete or interfere with the hydration process. Therefore, in order to produce concretes with satisfactory performance, both quantity and quality of water should be given emphasis. Excessive impurities in mixing water may not only affect the setting time and concrete strength but also may cause efflorescence, staining, corrosion of reinforcement, volume instability and reduced durability. More details on how impurities in mixing water affect various properties of concrete can be found in Lamond and Pielert (2006). Specifications usually set limits on chloride, sulfate, alkali

Impurity	Maximum tolerable concentrations	Influence on concrete
Sodium and potassium carbonates and bicarbonates Calcium and magnesium bicarbonates	1000 ppm 400 ppm of bicarbonate ion	Decrease setting times
Sodium chloride Calcium chloride Magnesium chloride	20 000 ppm 2% by mass of cement in plain concrete 40 000 ppm	Decrease setting times, increase early strength, reduce ultimate strength; may lead to rebar corrosion
Sodium sulfate Magnesium sulfate	10 000 ppm 40 000 ppm	May increase early strength, but reduce later strength
Iron salts Phosphates, arsenates, borates Salts of Zn, Cu, Pb, Mn, Sn	40 000 ppm 500 ppm 500 ppm	Retard setting
Sodium hydroxide Sodium sulphide	500 ppm 100 ppm	Affect setting behaviour

Table 1 Tolerable concentrations of impurities in concrete mixing water (adapted from Mindess *et al*., 2002; Lamond and Pielert, 2006)

and solid content in mixing water unless tests can be performed to determine the effects of impurities on various properties. The concentration limits for various impurities are given in **Table 1**.

Seawater in concrete

Seawater contains dissolved salts, mostly sulfates and chlorides of sodium and magnesium. Therefore, owing to the accelerating effects of the chloride ions (Mindess *et al.*, 2002), a rapid setting and early strength development can be expected (Liebs, 1949). However, the strength at later ages (after 28 days) may be lower because of the higher amounts of sulfate, which prolongs the crystallisation of ettringite (Liebs, 1949; Stenour, 1960). There are several references (Hadley, 1935; Japan Society of Civil Engineers, 1986; Mohammed *et al.*, 2004) which indicate that seawater could be used in mixing plain concrete without any problems at later periods. However, the presence of chloride ions in seawater increases the risk of corrosion of steel and, hence, most researchers (Neville, 2006; O'Connor, 1994; Kosmatka and Panarese, 1995) are of the opinion that seawater should not be used as a mixing water in reinforced concrete. As seawater causes efflorescence and may cause problems with decorative finishes (Stenour, 1960), it is also recommended to avoid using seawater in architectural concretes.

Testing of water

It has already been stated that the quality of mixing water for the production of concrete can influence the setting time, the strength development of concrete and the protection of the reinforcement against corrosion. Therefore, in assessing the suitability of water of unknown quality for the production of concrete, both the composition of the water and the application of the concrete to be produced should be considered. BS EN 1008: 2002 specifies the requirements for water that is suitable for making concrete which conforms to BS EN 206-1: 2000 and describes methods for assessing its suitability. Whenever there is any doubt on the quality of water to be used in mixing concrete, water has to be tested as per the Standards.

Concluding remarks

It is not possible to produce concrete without water. After casting, the application of external water to contribute to curing is also beneficial to concrete, but there are circumstances where it can contribute to the deterioration of concrete. The environmental conditions are the key factors in determining whether water will have beneficial or deleterious effects on concrete. Therefore, in order to understand the nature and behaviour of concrete, it should be kept in mind that water on the one hand is an essential element in producing concrete; but on the other hand, it is the water which is mainly responsible for various deterioration mechanisms in concrete.

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Chapter 10

Admixtures

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Admixtures have become a normal constituent of concrete, used to modify the properties in the fresh or the hardened state. The main admixture types are discussed with an overview of their mode of action and information on the applications for each type.

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Admixtures

Admixtures are chemicals added to concrete, mortar or grout to modify the properties of the mix in the plastic and/or the hardened state. They are normally supplied as an aqueous solution to facilitate accurate addition through a dispenser.

The quantity added is usually based on the cement content and for most admixtures is in the range 0.2 to 2.0% by weight, although the European Standard EN 934 allows up to 5%. In terms of active chemical, 2.0% of a typical admixture with less than 40% actives equates to less than 0.15% of chemical in a concrete mix. Even at this low content they have a powerful effect, modifying the water requirement, setting time and other properties. At higher dosages (above 1.5% on cement weight) it may be necessary to take account of the water in the admixture when calculating water/cement (w/c) ratio.

Most types of admixtures are covered by a European or National standard that requires them to meet basic performance requirements, provide information on properties that can be used to check uniformity of supply and to have a factory production control system that gives assured quality. In the UK the two main admixture standards are BS 8443 and the European Standard EN 934. Those admixtures conforming to EN 934 can be CE marked. In other areas of the world, the American ASTM admixture standards ASTM C 494 and C 1017 are frequently used.

Admixture use has increased significantly in the last 30 years and they can now be found in most concrete. In the UK it is estimated that more than 85% of all ready-mixed, site-mixed and precast concrete contains an admixture. The benefits are economic and environmental as well as technical and for most concrete mixes, if no admixture is being used, the question should be why not?

Admixture types and their applications are well documented and the UK Concrete Society Technical Report 18: ‘A Guide to the Selection of Admixtures for Concrete’, revised in 2002, remains a good reference source, although it is becoming slightly dated with respect to superplasticiser use and especially on self-compacting concrete applications where Concrete Society Technical Report 62 and the ‘European Guidelines for Self-compacting Concrete’ give better guidance.

The main areas of admixture application can be summarised as:

- economic – optimised mix design
- placing – appropriate workability (consistence), compactability, cohesion, setting and strength development
- problem solving – concrete that will meet special needs for placing or performance
- durability – helping to ensure that concrete fulfils its design requirements for its intended life
- environmental – helping to reduce the health and safety aspects of concrete during placing and environmental impact during its life cycle.

Admixtures can be broadly split by function and then by type as follows:

- dispersant types
  - plasticisers
  - superplasticisers
- set control types
  - set retarding
  - set accelerating
  - hardening accelerating
  - sprayed concrete accelerating
- air entrainment
  - concrete
  - mortar
- special purpose
  - water resisting (waterproofing)
  - underwater concreting

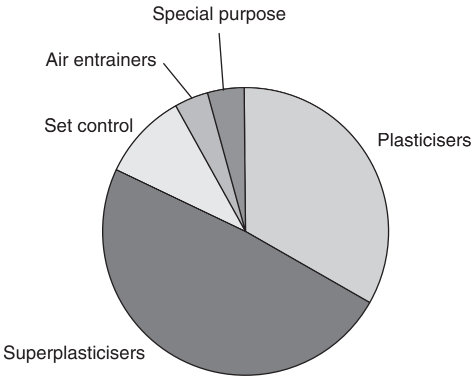

Figure 1 European admixture sales volume by type (EFCA, 2005)

- rheology control – segregation reducing
- semi-dry concrete
- polymers
- foaming.

Over 75% of admixture production is of dispersant types sold as plasticisers (water reducers) or superplasticisers (high-range water reducers). The split between these two types varies considerably between countries. Hot countries such as the Middle East will use predominantly superplasticisers but in cooler climates the split is more mixed and usually relates to local practice in relation to workability (consistence) as measured by slump or flow but also to the optimisation of cement content for economic and environmental reasons.

Figure 1 shows the split in European admixture sales between the main categories and highlights the dominant position of the dispersant types. This figure would suggest that superplasticisers are the most important type but this is not the case in all countries, as shown in **Figure 2**.

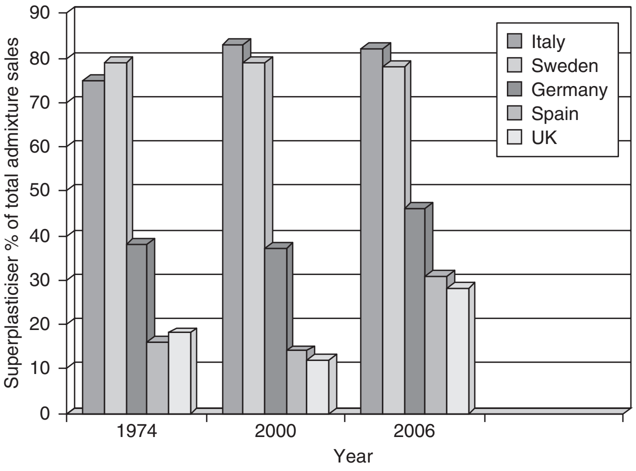

Figure 2 The superplasticiser sale as a percentage of total admixture sales by European country over the period 1994 to 2006 (EFCA, 2005)

The trend is to greater use of superplasticisers, but in countries such as Germany, Spain and the UK normal plasticisers still make up the larger proportion of sales.

Dispersing admixtures

Dispersing admixtures work by deflocculating the cement particles in a concrete, mortar or grout mix. The effect is most noticeable on the cement particles but other cementitious additions are also dispersed and the dosage is normally based on the total cementitous content. Cement particles will naturally agglomerate into larger flocs or clumps, which trap some of the mixing water within the clump. Large clumps of particles will also find it harder to move past each other in a given space than would be the case if they were still individual particles. This results in the need to add more water than is needed for cement hydration in order to move the clumps further apart and provide the necessary fluidity to allow the mix to be placed and fully compacted. However, additional water has a deleterious effect on the strength, durability and other structural properties and so should be avoided.

Dispersing admixtures function by adsorption onto the cement surfaces where they then repel each other and hence the cement particles, dispersing them and also preventing them from re-forming into immobile clumps or flocs. This has the effect of increasing the fluidity of the mix for a given water content/water:cement ratio (see **Figure 3**). By following the zeta potential (Uchikawa, Hanehara and Sawaki, 1997; Hewlett and Young; Uchikawa; Hanehara, 1997; Sakai and Daimon, 1997) it can be deduced that the initial admixture adsorption onto the positively charged cement particle is electrostatic but the subsequent inter-particle repulsion may be electrostatic or steric depending on the admixture type.

By dispersing the cement particles, these admixtures can be used in a number of ways:

- *Increase the fluidity* or consistence of the mix, making it easier to place (they are usually called plasticisers when used in this mode).
- *Reduce the water content*, reducing permeability and improving strength and other mechanical properties (they are usually called water reducers when used in this mode).

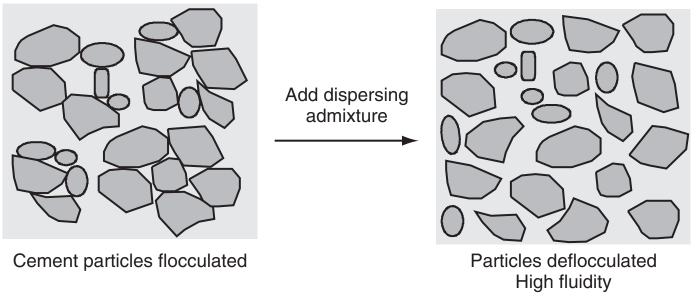

Figure 3 Admixture-induced dispersion of cement particles

- *Reduce the water and cement* content to optimise material cost and lower the environmental impact.
- With appropriate dosage, a combination of the above three applications.

These properties and their applications are discussed in more detail in the following subsections and are related to the main dispersing admixture types. Further information can also be found in The Concrete Society Technical Report No. 18 (Concrete Society, 2002).

Normal plasticising (water-reducing) admixtures

Dispersing admixtures of this type typically give 8–10% water reduction at equal consistence. Increasing the dosage will not give significantly greater water reduction but may result in some unacceptable secondary effects such as retardation or air entrainment. Conventional mix design methodology will usually give satisfactory results when using normal plasticisers.

Plasticised admixtures are usually based on lignosulfonate, a by-product of the timber for paper industry. The lignosulfonate may be supplemented with other carbohydrate-based materials or low levels of superplasticiser as well as other modifiers including air detraining and set control agents. Normal plasticisers are usually most effective in the dose range 0.3–0.6% on cementitous weight and are less expensive than superplasticising admixtures. Because they are natural by-products they are also regarded as having a relatively low embedded carbon content which is important in relation to environmental impact and sustainability. More highly processed and fractionated lignosulfonates are available that give enhanced properties and reduced secondary effects.

The combination of low dose and cost makes normal plasticisers very cost effective for a wide range of general concretes supplied by ready-mixed producers. By reducing the water, typically by 10%, without affecting consistence, an increase in strength and improvement in other properties such as durability can be obtained. Alternatively, an equivalent percentage of cementitous material can also be removed while maintaining w/c ratio and hence strength. This reduces not only the overall cost but also the embedded carbon footprint of the concrete. Factors such as water use and embedded carbon are becoming of increasing importance in relation to sustainability of construction materials and admixture use is therefore an important tool in achieving this for concrete.

Lignosulfonate-based admixtures frequently give a small increase in entrained air content, typically in the order of 1%. On the one hand, this can lead to a slight reduction in strength but on the other, air will increase cohesion in the mix, reducing bleed and segregation. This can be a positive advantage as it permits the producer to widen the range of constituents that will produce an acceptable mix. An example could be allowing a more local sand source to be used that is less well graded but reduces cost and carbon transport miles. A cohesive mix that is free of bleed and segregation is usually easier to place and will produce a good closed finish to the surface.

Many contractors specify a 50 mm slump concrete in the belief that this will give them the lowest cost, however, this will be difficult to place and finish and may lead to the concrete gang adding uncontrolled water on site or to poor compaction resulting in voids and honeycombing in the hardened concrete. The addition of water on site should always be avoided as this will reduce the concrete strength and durability. The concrete should be specified with a workability that is appropriate to the application and 75–100 mm slump is the minimum for most hand placed jobs. The addition of a normal plasticiser by the concrete producer has an insignificant effect on the concrete cost but can easily take the slump up to 100 to 120 mm, giving easier placing and finishing with no detriment to the properties.

A higher initial workability, combined with the slight retarding effect inherent in most normal plasticisers, may help to give some increase in workability retention, prolonging the period over which the concrete can be placed and compacted. However, if workability retention is a significant factor, a superplasticising admixture should be considered (see Sections on Superplasticised flowing concrete and on Set control admixtures).

Superplasticising (high-range water-reducing) admixtures

Dispersing admixtures of this type typically give 15–30% water reduction at equal consistence. Superplasticisers are responsive to dosage and even at higher dosage are reasonably free of unacceptable secondary effects such as retardation or air entrainment.

Superplasticisers first became commercially available in the early 1970s and were based on either sulfonated melamine formaldehyde condensates (SMFC) or sulfonated naphthalene formaldehyde condensates (SNFC). These were either used as single chemicals or, especially in the case of the SNFCs, blended with lignosulfonate and other chemical modifiers to produce enhanced or targeted properties including improved workability retention and better cohesion. Superplasticisers were quickly adopted as the main admixture in warmer climates, especially the Middle East and in some European countries but in others they were only used in specific applications where their higher performance was essential to obtaining the required concrete properties. In the UK, in 2000, they made up just 12% of admixture use.

In the 1990s a new type of superplasticiser was introduced based on polycarboxylate ether (PCE). The earlier

SMFC and SNFC types worked by electrostatic repulsion due to the high negative charge density carried on their surface. This allowed them to attach to positive charges on the cement particles but then the residual negative charges on the admixture acted as ‘like poles’ on a magnet to repel and disperse the particles. The PCE types work mainly by steric effect; there is a low-density negative charge allowing them to adsorb onto the cement particles but the molecules also carry long solvated side chains that prevent adjacent cement particles from coming close and forming flocs or clumps. Some additional information on the mechanism of superplasticiser action can be found in CAA Technical Sheet ATS 2 (Admixtures, website).

The advantages of PCEs include working over a wider dosage range, giving a more powerful dispersing effect and producing a degree of cohesion to the mix that makes them particularly suitable for very fluid mixes. Furthermore, unlike the earlier types that could only be modified by blending, the chemical structure of PCEs can be tuned to bring out certain targeted properties. Their introduction led to a new surge of interest in superplasticisers including the development of self-compacting concrete (SCC). In Europe, PCEs were quickly adopted, especially in precast and as a result by 2007 superplasticiser use in the UK had climbed from 12 to 29% of total admixture use and continues to increase. PCEs have not replaced SNFC-based superplasticisers but complemented them by increasing the range of applications. SNFC-based superplasticisers are generally easier to use and more forgiving of materials and production and still find wide use in superplasticised mixes for general purposes.

Superplasticisers are all based on synthetic chemicals and are typically dosed at 0.8–1.6% by weight on cement content. They are more expensive than normal plasticising admixtures and together with the higher dose are not cost effective in reducing cement content. Their main applications include:

- high-range water reduction for:
    - early strength
    - later age strength
    - reduced permeability (enhanced durability)
- high-flow for:
    - dense reinforcement
    - extended workability retention
    - high-flow almost self-levelling flooring
- self-compacting concrete for:
    - compaction without vibration
    - flow into voids with limited access/ability to vibrate
    - self-levelling slabs
    - high-quality surface finish.

Each of these applications is discussed in more detail in the following subsections. Further information can also be found in Concrete Society Technical Report No’s 18 (2002) and 62 (2005) and in the European Guidelines for SCC (EFCA, 2002).

The distinction between applications for high flow and SCC given above may seem small, but while high flow can be achieved with the older superplasticiser types and with a conventional mix design, SCC requires a PCE admixture and special mix design with well controlled constituent quality (see Section on Self-compacting concrete).

High-range water reduction (HRWR)

The water reduction with these admixtures is typically in the range 15–30% but at the top end of the dosage this can exceed 30% and in the case of PCEs it can be over 40%.

A typical concrete without admixture will contain 200–230 litres of water per cubic metre but only about 80 litres is actually needed to fully hydrate the cement. The remainder is there to give sufficient workability to the mix to allow placing and full compaction. On hardening, this extra 120–150 litres of water (12–15% of the concrete by volume) becomes void space in the form of capillary pores. This results in increased permeability to water and to aggressive chemicals including chlorides and carbonation as well as the potential for drying shrinkage and cracking as the water evaporates.

If a high-range water reducer is used to reduce the water by even a modest 20%, this reduces the void space to 8–10%. The capillary pores are smaller and less continuous so permeability is reduced and durability increased. The cement particles are also closer together, improving strength for a given level level of hydration. The effect is particularly apparent at early ages and at lower temperatures. **Figure 4** shows typical strength results at 20°C for

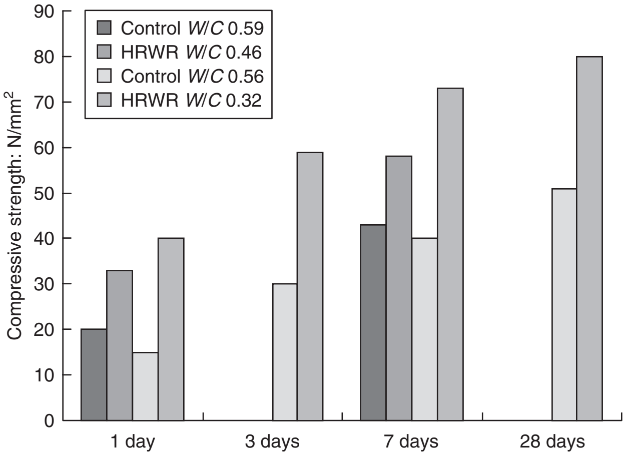

Figure 4 Typical effect of HRWR on compressive strength development N/mm^2 of two cements (CAA, 2006)

two cements showing strength development of an HRWR mix compared with a control from 1 to 28 days.

Comparisons are more difficult at very earlier ages as the control may have little or no strength but with a CEM 1 cement and an HRWR, strengths in the range 10–20 N/mm^2 can be obtained at 12 h at 20°C.

Early strength is particularly important for precast where the ability to demould and reuse the mould in a 24 h time window is often a critical requirement and while heating can be used to accelerate strength development it is often unnecessary if an HRWR admixture is used and the cost and sustainability consequences of the heating can be avoided. Early strength may also be needed in repair situations such as roads and airports where access/use will be restricted until the concrete has gained sufficient strength to take the required load and abrasion. In this situation a HRWR may be used with an accelerating admixture (see Section on Set control admixtures).

At later ages, high-range water-reducing admixtures can be used to achieve strengths in the range 80–120 N/mm^2 or more and such concrete is finding increasing use for thin section walls, high-rise structures and supporting columns in open structures.

Durability can be related to the ability of deleterious materials to penetrate the concrete, usually via the pore structure. The ability of HRWR to significantly reduce the water content of the concrete and hence both the volume and continuity of the pores can have a significant and advantageous impact on durability-related properties such as permeability and chloride diffusion.

Figure 5 shows the relationship between water permeability in m/s and water:cement ratio, using water reducers and high-range water reducers to progressively lower the water content of the concrete mix. The points are actual results with an imposed trend line. It can be seen that by using a water:cement ratio of below 0.50 and preferably below 0.45, the permeability is substantially reduced. A low permeability not only makes the concrete more water resistant, it also helps to reduce penetration of aggressive waterborne chemicals.

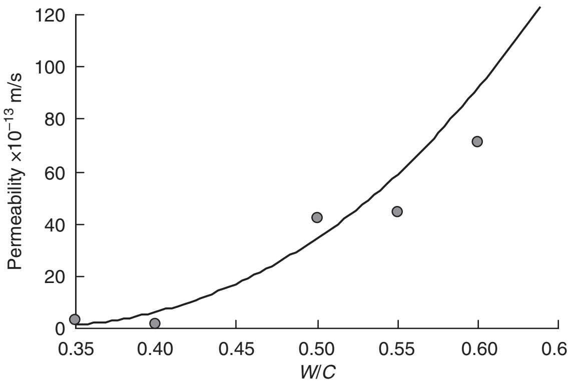

Figure 5 Typical permeability results against water:cement ratio using water-reducing admixtures (UK Cement Admixtures Association, website)

Mix	Cement CEM I	w/c ratio	28-day strength: N/mm^2	Chloride diff. $\times 10^{-12}$ m^2/s
Control	305	0.61	31.5	51.0
HRWR	305	0.51	45.0	22.0
Control	375	0.51	44.5	28.0
HRWR	360	0.41	58.0	8.1
HRWR	450	0.34	72.0	5.4

Table 1 Typical chloride diffusion results against water:cement ratio and cement (CEM I) content using an HRWR admixture (UK Cement Admixtures Association, website)

Chloride penetration is one of the main causes of corrosion failure in steel-reinforced concrete and high-range water reducing admixtures can play a significant role in reducing the water:cement ratio and hence the chloride diffusion coefficient of the concrete (see **Table 1**).

It is becoming increasingly common to find specifications for concrete in aggressive environments that are required to meet levels for water and gas permeability, chloride diffusion and other durability parameters. Meeting these specifications will require careful selection of all the constituents but admixtures will always be required to reduce water content.

Superplasticised flowing concrete

Superplasticisers can, at moderate dose, take a concrete from a 50 to 75 mm slump up to a collapsed slump of 180 to 220 mm with no loss of other properties. Some adjustment to the mix may be required to ensure that there is no bleed or segregation. It is normally sufficient to just increase the proportion of fine aggregate by around 5% but if this fails then the use of, or increase in finer material such as fly ash may also be necessary. Although very workable, some vibration is still needed to ensure adequate compaction but this should not be overdone or it may cause the mix to segregate.

Concrete in this consistence range finds extensive use for structural concrete where the density of the reinforcement and the scale of the application make placing and effective vibration difficult. As well as providing the high workability, the dosage of the admixture will probably have been adjusted to allow some water reduction to give a water:cement ratio that ensures good durability and adequate strength. This type of application would normally be pumped into place and flowing concrete can be pumped to significant heights and over long distances.

Flowing concrete has been used in almost self-levelling flooring applications but the advent of the laser screed machines and of SCC have largely superseded this application except perhaps for some small residential floor slabs.

Figure 6 Superplasticised high-workability flowing concrete being used with dense reinforcement. Note that it is not quite self-levelling and a vibrator is still needed to ensure full compaction round the reinforcement

Figure 7 Self-compacting concrete used in vibrator-free precast. Note the slow creeping flow that enables full encapsulation of the reinforcing bars but without aggregate separation and blocking behind the bar. See also the self-levelling with no build-up, even at the point of discharge, and the lack of segregation so that aggregate can still be seen at the top surface with no indication of bleed water or grout separation on the leading edge

Workability retention is important on many large sites as the time from production to delivery and then to placing can be considerable and frequently subject to unforeseen delays. The problem is even more pronounced at high ambient temperatures. All concrete loses workability with time and if the water:cement ratio is also low, workability loss can be very quick, but concrete with a high initial workability will retain a useable consistence for longer than those starting at a lower slump. To maximise workability retention the superplasticiser should be used to provide an initial high consistence (typically >180 mm slump) but note that some superplasticisers provide better retention than others so advice should be sought from the manufacturer where this is a critical requirement. The dosage may need to take account of the fact that the superplasticiser is being used to reduce water:cement ratio for strength and durability as well as providing high initial workability. Where the placing sequence could result in delays in placing and compacting adjacent concrete pours, a retarding admixture or retarding superplasticiser may additionally be required in order to avoid cold joints between pours (see Section on Set control admixtures).

Self-compacting concrete (SCC)

It was the development of polycarboxylate ether (PCE) superplasticisers with their unique rheological properties that permitted the development of SCC. However, the admixture alone will not confer SCC properties on a conventional concrete mix. To be self-compacting the concrete must have very high flow combined with no aggregate or paste segregation and no tendency for the aggregate to block as it passes through reinforcement (see **Figure 7**). The high flow is provided by the admixture but to ensure the other two properties the aggregate needs to be very carefully graded and the total powder below 0.063 mm needs to be higher than normal. This should not be achieved with cement alone but requires higher levels of additions such as fly ash, ground granulated blastfurnace slag and/or limestone fines.

Segregation resistance is an area of particular concern for users of SCC and where it proves difficult to achieve with the available constituents, segregation-reducing admixtures, often called VMAs, have been developed that can be used in conjunction with the superplasticiser. Advice should be sought from the manufacturer on the use and compatibility of these two admixtures. Segregation-reducing admixtures make the SCC more robust to changes in the constituent materials and to control at the mixing plant but are not a panacea for poor constituents, mixing and control at the production plant and sales suggest that a majority of producers do not need to use them. Further information on VMAs is available from an EFNARC/EFCA document *Guidelines for Viscosity Modifying Admixtures for Concrete* available for free download on the publications page of both the trade association websites.

The mix design of SCC is more complicated and more critical than for most other concrete. Trials are essential as is good production control and it is strongly recommended that advice and supply is sought from admixture suppliers and concrete producers with a proven track record in this type of concrete.

Further information on SCC, including how to specify it and the special test methods developed to assess it can be found in the *European Guidelines for Self-compacting Concrete* (EFCA, website) produced by the European trade associations for concrete and its constituents. It is available for free download on the publications page of all the trade association websites, including http://www.efca.info. The actual specification and test methods are currently being incorporated into the European concrete standard EN 206.

SCC is now used by most structural precast where it offers the following benefits:

- significant reduction in noise levels in the working environment (no vibration)
- good strength development profile
- improved surface finish and the picking up of any detailing on the mould
- ability to flow into and fully fill more intricate mould shapes including under cuts.

On sites it is used for:

- difficult access or complicated shapes
- high-quality surface finishes
- reduced placing noise on sensitive sites.

On site it offers:

- quieter concreting/potential for night working
- no vibration white finger
- reduced labour requirement
- difficult access and ability to flow into difficult or inaccessible voids
- filling by pump from the bottom of the shutter
- high-quality surface finishes
- self-levelling slabs with minimal finishing.

Some additional care is needed on site with SCC to ensure the formwork is tight against leakage and there are no defects as these will be readily picked up and highlighted in the otherwise high-quality final concrete surface finish. Due to the total absence of bleed, quick and careful curing is needed on all exposed top surfaces or the concrete may start to dry out, lose surface strength and form early-age plastic shrinkage cracks.

Set control admixtures

This group of admixtures includes the following types:

- set retarding
- set accelerating
- hardening accelerating
- sprayed concrete accelerating.

Set control admixtures make up a small proportion of sales (see **Figure 1**) and are restricted to relatively specialist applications. The hardening accelerators have been largely replaced by high-range water reducers, some of which have been modified to also accelerate the setting and strength development.

Set-accelerating admixtures for concrete are mainly inorganic chemicals and find use in very cold conditions or when very early stiffening is needed in repair situations. The chemical of choice for set accelerators used to be calcium chloride (which also acts as a hardening accelerator) but due to it enhancing steel corrosion, chloride is now banned from all applications where there will be embedded steel. The alternatives are generally less effective which probably explains their low level of use. They are usually most effective when used in combination with an appropriate high-range water-reducing admixture and find most of their use in rapid repair situations were there is a strength requirement for access/use soon after placing. Additional information can be found in CAA technical sheet ATS 4 (CAA, website).

Set retarding admixtures are important, especially on large pours in warmer climates where there is a significant risk of cold joints forming within the structure. Retarding admixtures are available separately but are often sold as retarding plasticising or retarding superplasticising. This is because a high initial consistence is likely to be required for workability retention. The retarder does not have a very significant effect on workability retention but it will delay the initial setting, by a few hours to several days depending on the retarder type and dose. On large pours there can be a long delay between placing one pour and the next adjacent one. This may be due to the complexity of the pouring sequence or to delays such as delivery or pump breakdown. The warmer the weather, the shorter the tolerable delay. If the first pour stiffens beyond initial set before the adjacent concrete is poured, it will be difficult to get a good monolithic joint and in the extreme case where the concrete has passed final set there will be no bonded joint, called a cold joint. Even if the first pour has lost all workability, provided it has not actually passed initial set, it will under vibration still be possible to form a good joint to subsequent pours. Additional information can be found in CAA technical sheet ATS 3 (CAA, website).

Special set-accelerating admixtures are used in sprayed concrete applications and are covered by European Standard EN 934-5: 1996. Superplasticisers are also used to give initial consistence and low water : cement ratio in the fresh concrete and there are a number of other special admixtures that have been developed for this application. The stiffening/set-accelerating admixtures are added with the air at the spray nozzle to provide the following benefits:

- very fast setting (stiffening)
- early-age strength development
- increased layer thickness

- improved overhead spraying performance
- reduced sagging.

The fastest setting accelerator types are the aluminates and silicates which stiffen in seconds but these are very alkaline and consequently have serious health and safety implications so have been largely superseded by alkali-free admixtures, mainly based on aluminium salts. These give slower stiffening and early strength than the aluminates but are preferred because they are not alkaline, give a better working environment and give better final strength and durability to the concrete. The dosage of sprayed concrete accelerators is high at 5–10% of cement weight and their performance is affected by the cement chemistry. These sprayed concrete accelerators are often used in conjunction with concrete that contains a special retarding workability retention admixture to facilitate the long pumping distances and slower rates of placing that are typically encountered. Sprayed concrete is a specialist area and advice should always be sought from the admixture manufacturer on the suitability of his products for the type of cement and other conditions applicable to the site. Additional information can be found in the CAA technical sheet ATS 8 (CAA, website) and from EFNARC Guidelines for Sprayed Concrete (EFNARC, website).

Air-entraining admixtures

Air entrainment is usually considered for its ability to impart freeze–thaw resistance to saturated concrete and should always be specified for external slabs and paving as well as any other external concrete that may be saturated when subject to frost action. However, air can give other benefits to concrete and mortar. Additional information can be found in CAA technical sheet ATS 5 (CAA, website).

Optimum freeze–thaw properties require a large number of very small bubbles. Standards often specify an air void spacing factor of less than 0.2 mm and this refers to no point in the cement paste being more than 0.2 mm from an air void. The air bubbles intersect the capillary system within the concrete and may actually help to stop it becoming saturated. Even in almost saturated concrete the bubbles remain dry due to the capillary action sucking any water out of the bubble and into the capillaries. If water in the capillaries freezes, the air voids then act to relieve the pressure generated by the expansion of the freezing water. The air voids also reduce the osmotic pressures caused by the use of de-icing salts and which could otherwise cause cracking and surface scaling.

Small amounts of air will improve cohesion, reducing bleed and bulk the paste in lean mixes to give easier finishing. Air bubbles also lubricate the aggregate and can reduce the water requirement and significantly enhance compaction in dry lean concrete or in semi-dry concrete used for some precast concrete products such as bricks, blocks and pavers.

Cohesion generated by the air while assisting with compaction, also helps to reduce slumping when vibration stops. This can be useful in some slip-form applications such as the placing of central road barriers where the still wet concrete is required to stand to full height and retain its shape as soon as it leaves the slip-form.

Air entrainment can lead to lower strength so levels of air should be kept to the minimum appropriate for the particular application. For cohesion this could be as little as 1–2% but for freeze–thaw resistance, 4–6% will normally be needed, see EN 206-1: 2001 for specifications as these are related to aggregate size. Air can be lost during pumping and also if the concrete is overvibrated, so care and checking is needed at the point of placing.

Air entrainment is often used in bedding mortars for masonry where up to 18% air is entrained. It may be called a mortar plasticiser in this application because it does reduce the water demand in this type of high sand content mortar mix but the air also reduces density and gives improved cohesion, resulting in better handling properties. They are covered by European Standard EN 934-3: 2003 and additional information can be found in CAA technical sheet ATS 7 (CAA, website).

Special purpose admixtures

Admixture producers have used their expertise to meet specific needs within the concrete industry and a range of other admixtures has been developed to help concrete meet special requirements within the construction industry. The range includes:

- Admixtures that help concrete to resist water ingress: these may be hydrophobic and effective against rain and occasional submersion. There are also pore-blocking products that will help to resist water under a significant head. These are water-resisting admixtures also called waterproofers. Additional information can be found in CAA technical sheet ATS 6 (CAA, website).
- Underwater admixtures help to prevent concrete from losing cement and fine aggregate while being placed under water or when it is subject to moving water before it can set. Additional information can be found in CAA technical sheet ATS 17 (ATS, website) and the EFNARC/EFCA *Guidelines for Viscosity Modifying Admixtures for Concrete* (EFNARC or EFCA, website).
- Settlement-reducing admixtures, also called VMAs, used mainly with self-compacting concrete to reduce settlement and bleed. Additional information can also be found in EFNARC/EFCA *Guidelines for Viscosity Modifying Admixtures for Concrete* (EFNARC or EFCA, website).
- Shrinkage-reducing admixtures help to prevent drying shrinkage cracking. They can be used to increase bay sizes in concrete floors and so reducing problems associated

with joints. Another example of use is in radiation areas where dense, crack-free concrete is required. Additional information can be found in CAA technical sheet ATS 15 (CAA, website).

- Corrosion-inhibiting admixtures can be used to increase the corrosion threshold of embedded steel, especially where chloride ingress may be a problem. These are integral admixtures added at the mixer and should not be confused with migratory corrosion inhibitors added to the hardened concrete. Additional information can be found in the CAA technical sheet ATS 9 (CAA, website).
- Mortar admixtures, as well as being air entrainers, can also give retardation of the set and retention of mix consistence so that the mortar can be used over an extended period, typically 36 h. However, when applied to an absorbent brick or block, water and admixture are sucked out, promoting earlier setting. Both air-entraining and retarding mortar admixtures are covered by European Standard EN 934-3: 2003. Additional information can be found in CAA technical sheet ATS 7 (CAA, website).
- Other special purpose admixtures include pumping aids, washwater admixtures for in-truck recycling of residual concrete, foam concrete admixtures, polymer dispersion admixtures, semi-dry concrete admixtures. Further information on these can be found in the CAA admixture technical sheets (CAA, website).

Health, safety and environment

Admixtures are usually sold to the concrete producers as aqueous solutions with less than 40% active chemical and most admixtures require no hazard labelling such as Harmful or Toxic under current EU legislation. All admixtures are supplied with a Material Safety Data Sheet (SDS) which details the action to take in the event of a spillage or accidental ingestion, skin or eye contact.

The quantity of active chemical in the hardened concrete is very small and is essentially bound into the concrete. This means that there is little or no leaching and most admixtures have been approved for use in drinking water applications such as reservoirs, pipes and tanks.

Admixtures will generally enhance the sustainability of concrete by helping to reduce the overall level of embedded carbon and increase the durability.

Overdosing: any dosage that is significantly in excess of that intended (overdosing) can, in certain circumstances, result in mix segregation, retardation and/or a degree of air entrainment. The higher the intended dosage within the manufacturer's stated range, the more significant will be the effect of any overdosing. For non-retarding admixtures the retardation is unlikely to be more than 24 h even at considerable overdose but extra curing is advisable to prevent the surface drying out before setting and revibration up to the time of initial set will help to prevent plastic settlement cracking. If overdose results in a significant increase in entrained air, this will reduce the final strength of the concrete. Most admixture manufacturers will provide information on the specific consequences of overdosing for each admixture type.

Technical information and support

Admixture producers have always been at the cutting edge of innovation in the concrete industry and are pleased to discuss specific problems and help to meet the requirements of new and innovative designs or difficult placing situations. Most producers provide services that include trials and on-site support if necessary. The admixture trade associations including the UK Cement Admixture Association (CAA) http://www.admixtures.org.uk or the German Deutsche Bauchemie (DB) http://www.deutsche-bauchemie.de have a lot of useful information for free download on their websites and links to their admixture-producing members. Other admixture trade associations can be reached through the European Federation of Concrete Admixture Associations (EFCA) http://www.efca.info.

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CONTENTS

Chapter 11

Methods of concrete manufacturing and curing

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The initial stages of concrete preparation, placement, compaction and curing are central to the performance of Portland cement concrete in different applications. This chapter reviews various technological aspects that affect concrete performance, such as the requirements for concrete mixing and transportation, formwork types, casting methods, and the effects of vibration, as well as different methods of concrete curing. Special attention is given to novel concrete types and advanced industrial technologies.

Introduction

Concrete technology

Historically, construction has been performed with elementary building units such as natural stone, timber, straw bales, clay blocks, bricks and, later, structural steel. These units have been held together by mortar, connecting elements such as nails, bolts, rivets or welding. With the appearance of concrete circa 1800, a new type of construction method – 'cast in place' – was realised. The advantage of this method is in its ability to produce large-scale structures cast as a single unit without any connecting elements (except expansion joints). With the invention of reinforced concrete such technology helped realise in the most efficient way the most challenging projects of the twentieth century, such as the 2.6 million m^3 Hoover Dam (USA, 1936), 540 m Ostankino Tower (Russia, 1967), 380 m Gullfaks C offshore platform (Norway, 1990), 12.9 km Confederation Bridge (Canada, 1997), 452 m Petronas Towers (Malaysia, 1998). The most important advantages of concrete include the ability to form the shape of a desired configuration, to incorporate the reinforcement elements, the ability to transform to a strong, durable, stone-like material, as well as low cost and availability of local raw materials, resulting in a sustainable and highly desirable product.

Another aspect of modern concrete technology is related to the industrial manufacturing process, or precast concrete. This process involves the production of reinforced concrete elements in controlled factory conditions through the technological steps from concrete mixing to pre-assembling of manufactured elements, and, finally, to the delivery and installation at the construction site. The advantage of the process is that the element's shape, geometry, dimensions, position and location of the reinforcement, as well as the processes related to concrete placement, compaction and curing, are executed under precise quality control. It is obvious that prefabrication is the major trend for manufacturing of high-quality modular construction elements; however, cast-in-place concrete remains the most effective for large-scale projects.

In spite of their differences, these two main manufacturing processes involve the same technological steps:

- preparation of concrete mixture;
- transportation and placement of fresh concrete into the mould(s) with preinstalled reinforcement;
- compaction and finishing of fresh concrete;
- curing of the concrete element/structure;
- formwork removal, inspection and final finishing.

Production and casting of fresh concrete

Batching and mixing

Preparation of a fresh concrete mixture starts with weighting or dosing the individual components of concrete according to the mix design, known as batching. A particular volume of concrete materials mixed is called batch. Mixing of concrete can be done in several ways, depending on the volume of the batch and the job requirements: by hand either in a metal tray, in a wheelbarrow, or by using machine mixing in a drum, pan, single- or twin-shaft mixers. The objective of the mixing procedure is wetting and uniform distribution of all the components of the mixture and the production of a homogeneous mixture with required workability (ACI Committee 304).

The components of the mix can be metered or dosed by weight or volume, depending on the equipment available. The majority of industrial applications involve concrete batching in the plant that can be stationary, field or mobile. Such plants usually have separate bins for different

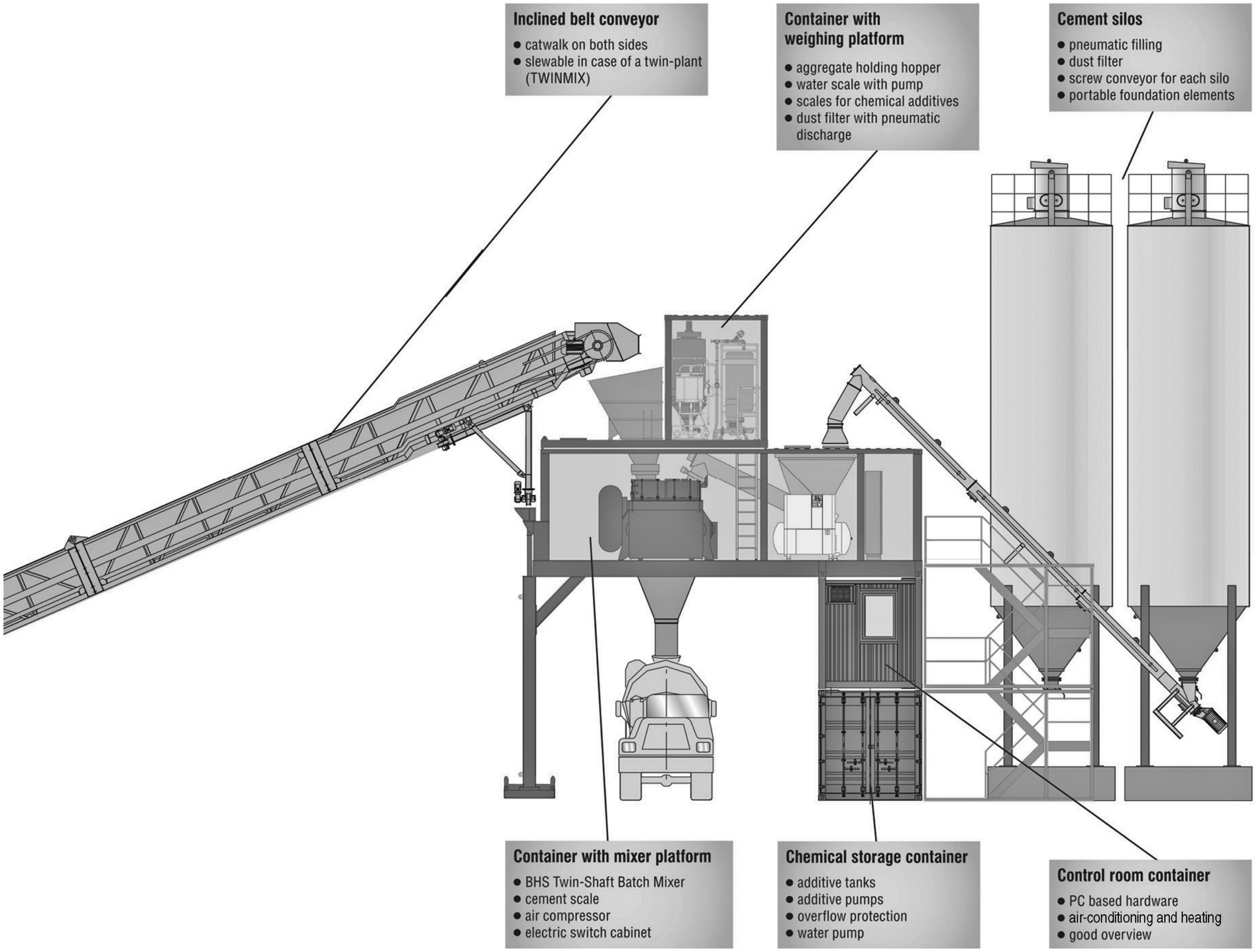


Figure 1 Layout of ready-mixed concrete plant (courtesy of BHS-Sonthofen GmbH)

aggregate types and separate silos for cement(s) and mineral additive(s) such as fly ash or silica fume. Powder materials such as cement and mineral additives are weighed in the aspirated bin and transferred directly to the concrete mixer (**Figure 1**). Aggregates are dosed one by one into the weight conveyor (or, sometimes, into an intermediate weight silo installed atop a mixer) and delivered into the mixer by belt conveyor or skip. Water is batched by volume or by weight, depending on the mixing plant set-up. Recently, chemical admixtures such as plasticisers and superplasticisers have become common in concrete mixtures; these are used in a liquid form, dosed by volume, and pumped into the mixer following the assigned mixing procedure. Silica fume (SF), a standard constituent of high-performance concrete (HPC), is usually available in a densified form, which can be handled by the cement or fly ash dosing equipment. However, the disagglomeration of tiny SF granules to the level of individual particles cannot be effectively handled by common concrete mixing equipment. Therefore, premixing of densified SF with part of the mixing water and, optionally, with a superplasticiser (Dalstol, 1995; Sobolev and Soboleva, 2003) is recommended for the production of high-quality HPC. Such SF slurry is prepared in a separate mixer (usually colloidal or grout mixer), can be stored in the agitating tank and can be dosed using the water metering method. Sometimes, fly ash (low-calcium, type F) is also introduced into a mix in a form of slurry using a similar approach.

In addition to effective mixing, the major challenge for preparing good-quality concrete is related to accurate metering of the components used and, especially, accounting for the moisture content of the aggregates.

Figure 2 Large-capacity drum mixer

The aggregate moisture content fluctuates frequently, which can alter the prescribed mix design and, therefore, the performance of manufactured concrete. With online moisture metres, the quantities of water introduced with aggregates can be evaluated and subtracted from the required amounts of mixing water for each batch, assuring the overall uniformity of manufactured concrete. Other critical aspects of modern ready-mixed concrete technology are related to control of workability, air content and temperature of the fresh concrete supplied from the facility.

Commonly, the concrete mixed in a stationary plant can be delivered to the job site by a truck. This operation is the core of the ready-mixed concrete industry. The concrete delivery trucks have a drum mixer with a capacity of up to 10 m^3 mounted on the truck's frame and connected to the transmission, so the drum can rotate en route and at the site (**Figure 2**). Often, the ready-mixed concrete plant provides only batching (or proportioning) of the ingredients, while mixing is done using a truck mixer en route, so concrete is delivered freshly mixed. This concrete must be used within 90 minutes or before 300 revolutions of the drum after the addition of water (Somayaji, 2001). During hot weather or long distances, water and chemical admixtures are metered and added at the construction site; this helps to minimise the slump loss, but requires additional quality control measures to ensure that all the components are metered properly and the final mix meets the specifications.

Given the diversity of manufactured concrete mixtures, batch mix processing provides a flexible on-demand supply of the mixture with the required quality and quantity. The volume and capacity of mixers vary depending on the job requirements and the equipment specifications. There are drum mixers of up to 12 m^3 capacity, but the common capacity of pan mixers is less than 3 m^3. Taking into consideration that the complete mixing cycle, including loading, mixing and discharging, may be up to 15 minutes (with typical mixing times of 2–5 minutes), these mixers are capable of manufacturing up to 50 m^3 of concrete per hour. In some projects requiring relatively large quantities (more than 50 m^3 per hour) of uniform mix of the same quality, continuous mixing plants can be an option. These operations include large precast or dry mortar facilities. Due to the specifics of the process, horizontal or vertical shaft mixers are commonly used for continuous mixing (Autocon innovative continuous processing systems).

Transportation at the jobsite

Within a relatively short range, fresh concrete is delivered to the placement point by wheelbarrows, carts, chutes, moving bunkers, belt conveyors, cranes or pumping. Cranes and pumps are among the standard equipment for large-scale and high-rise applications. Precast applications commonly use bunkers or belt conveyors to deliver concrete from the mixing plant into the production area. Further delivery and placement is realised with the help of moving bunkers equipped with a screw or belt conveyor; these provide the precise placement of fresh concrete into the moulds.

Pumping operations require concrete of high workability, usually with a slump of 100 mm or higher and also with an increased proportion of small-size particulate fraction (i.e. sand, cement or mineral additives). Often, pumping-aid admixtures are used to preserve the homogeneity of the mixture and to facilitate the concrete flow. Concrete with round or smooth-edge coarse aggregates such as gravel results in an easier flow when pumping; however, crushed stone is commonly used for the design of pumped concrete. Combined application of crushed stone as a coarse aggregate (particle size of 12.5–25 mm) and gravel as a mid-size aggregate (size of 5–12.5 mm) was found to be an effective solution for design of pumped concrete (Sobolev, 1997). Modern pumps have the capacity of up to 100 m^3 per hour and can be used to deliver concrete to a distance of up to 750 m horizontally and up to 400 m vertically (Somayaji, 2001). The efficiency and feasibility of the pumping operations depends on the volume of concrete to be placed, since the pipes must be lubricated by mortar in the beginning of the operation and also must be cleaned at the end (**Figure 3**).

Concrete formwork and moulds

Depending on the job requirements, the concrete formwork can be made of different materials such as:

- wood or plywood;
- steel;
- aluminium (with protective finish);
- fibreglass;

Figure 3 Pumping and placement of superplasticised concrete (Park Lafayette Tower, Milwaukee)

- ABS (acrylonitrile butadiene styrene) plastic;
- concrete;
- styrofoam.

To meet the architectural concrete specifications, the formwork can be surfaced with formliners, usually made of elastomeric materials such as polyurethane or silicon, but also can be polyethylene, polypropylene, Styrofoam, fibreglass or other plastic material.

The formwork is usually designed to withstand the lateral pressure of fresh concrete as well as the loads originating from concrete vibration, crew operation and machines. For concrete with high workability (slump of more than 175 mm) and, especially, for self-compacting concrete, this pressure is equal to the hydraulic pressure of fresh concrete (ACI Committee 347, 2004):

$$P = \rho g h$$

where

P lateral pressure, kPa;
ρ density of concrete, kg/m^3;
g gravitational constant, 9.81 N/kg; and
h depth of plastic concrete from top of placement to point of consideration, m.

For concrete of a slump of 175 mm or less, this pressure can be reduced following the recommendations of ACI 347. The moulds in the precast industry are often subjected to heavy vibration, transport loads (by crane, railway or tilting operations) and exposure to thermal gradients (during accelerated curing). Therefore, all these factors must be considered in the design of precast moulds.

To provide trouble-free form removal, as well as to preserve the form material and avoid discoloration of the concrete surface, different types of form coatings and release agents are used. These are commonly based on hydrocarbon derivatives such as petroleum oils, paraffins, waxes, silicone oils or vegetable oils (the oil-based products are often used in a form of water emulsion). The main idea of such surface treatment is to avoid the bond, chemical interaction, or to prevent moisture exchange between the concrete and formwork material. However, there are special polymer-based materials (applied as formliners on the surface of the form) that allow to absorb or filtrate water from fresh concrete. Permeable polymer coatings, non-woven geotextile and woven polypropylene fabric, were successfully used in these applications (Basheer *et al.*, 1995; Long *et al.*, 1995; West, 2003). The application of a controlled permeability formwork (CPF) allows the filtration of the surplus mixing water and entrapped air (**Figure 4**). Different types of CPF were used for production of architectural panels (**Figure 5**). The resulting surfaces were reported to be bubble free and hard, with higher strength and better abrasion resistance due to densification of concrete. The application of a CPF liner results in (Basheer *et al.*, 1995):

- a significantly reduced water/cement (w/c) ratio near to the surface and enhanced surface strength;
- a greatly reduced incidence of blow holes;
- much greater resistance to carbonation and chloride ingress;
- greatly improved performance under freezing and thawing conditions.

Slipforms are used for continuous concrete cast operations; they are pulled or raised (at certain intervals) as concrete is placed. They are used for continuous cast of pavements, canals, tunnels, precast hollow-core planks, such as

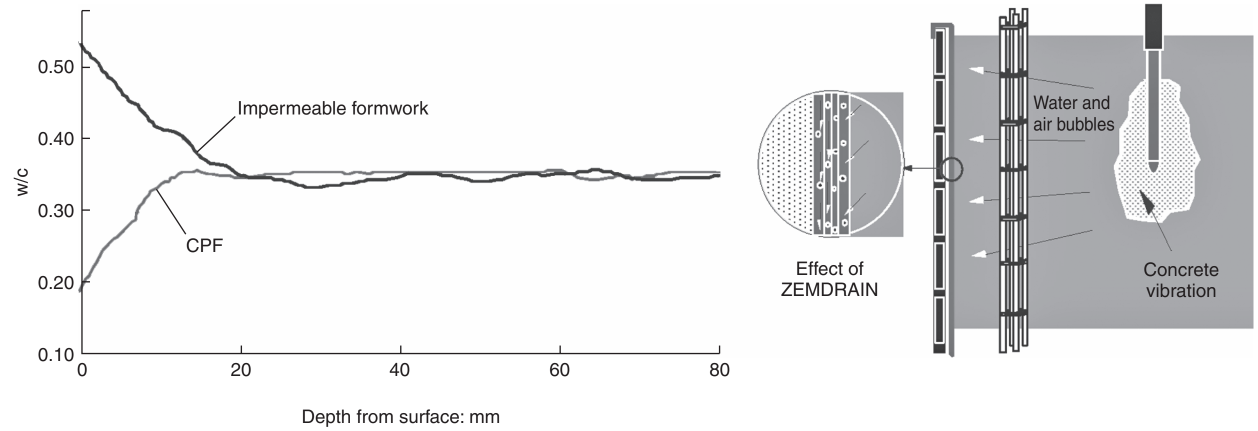


Figure 4 Densification of concrete due to application of controlled permeability formwork (reprinted from A. E. Long *et al*., Second CANMET/ACI International Symposium on Advances in Concrete Technology ©1995 ACI)

presented in **Figure 6** (horizontal slipforms), or to form the walls of buildings or silos (vertical slipforms).

Although formwork is removed from the concrete member as soon as the concrete develops sufficient strength, two types of forms remain untouched and constitute the part of cast concrete element: insulated formwork and decorative concrete panels. Insulated concrete formwork (ICF), composed of two sheets of 40–50 mm expanded polystyrene (EPS) or polypropylene connected by a plastic web holder, is used to brace the ICF blocks together and grip the reinforcement bars in the design position. After the placement and initial hardening of concrete, the surfaces of the resulting sandwich wall are finished by plastering or stucco. Due to the exceptional heat insulating properties of EPS, the resulting wall meets all green and energy star ratings; however, the health issues related to possible styrene or volatile organic compounds (VOCs) emissions into the interior of the building (especially at elevated temperatures) have not been investigated or reported. Decorative concrete (or fibre concrete) tiles are thin, sometimes only 10–25 mm thick, panel elements with a decorative surface finish that are used as a permanent formwork (**Figure 7**). These tiles of relatively large area (from 0.25–0.55 m^2) are connected to the steel frame (that can be a part of scaffolding or a part of a wall structure), then sealed and, finally, the space between the tiles is filled with concrete, usually lightweight aggregate based or foamed concrete.

Figure 5 Precast wall panel produced using the fabric formed mould (M. West Fabric-formed concrete members, Concrete International, 2003)

Placing and finishing

Concrete must be placed at its proper location as soon as it becomes available from the mixing facility. A number of concrete applications require certain aesthetic or functional finishes on the concrete surface. The process of finishing usually starts when water evaporates from the concrete surface and involves screeding, floating and travelling, resulting in a smooth or rough surface. Broom finish is of particular interest for pavement applications since it provides a non-slip surface (Somayaji, 2001; Long *et al*., 1995). There are different techniques to achieve the architectural concrete finish such as stamped or exposed aggregate concrete

Figure 6 Production of Spancrete® hollowcore plank (courtesy of Kim Walker, Spancrete Group, Inc., USA)

(Sobolev *et al.*, 2004). Vacuuming of flat concrete surfaces was found to be a very effective post-finishing treatment for the construction of concrete floors (Hatanaka *et al.*, 2008).

Review of casting methods

The numerous methods to cast concrete require a concrete mix of certain fresh properties in a state characterised by consistency, mobility, compactability and finishability (Long *et al.*, 1995; Ahverdov, 1981). These are broadly represented by a single integral parameter called workability. Forming and compaction are the two basic processes involved in any concrete casting. Accordingly, the mobility or formability represents the potential of fresh concrete to flow and fill the form of a specified shape under the compaction gradients. The compactability can be described as the ability of the material to consolidate without segregation under its own weight, pressure or vibration; this process must result in a dense product with very little volume of entrapped air (less than 3%) (Ahverdov, 1981; ACI Committee 309).

The most popular concrete casting method involves the placement of a plastic concrete mixture into the mould or form, with subsequent compaction using internal or external vibration (**Figure 8(a)–8(c)**). Vibration is required to distribute the concrete mix within the form and to provide the required compaction degree and the uniformity of material with very little amount of entrapped air, usually not more than 3% (ACI Committee 309; ACI 309R-96). The vibration process and concrete mix must be designed to work together so that the vibration intensity matches the workability level and proportioning of aggregates (and vice versa). The deviations from the optimal designs can result in a honeycombing (air pockets in congested reinforcement zones, when workability level or vibration intensity is insufficient) or segregation (when the settlement of coarse segregates occurs from over-vibration), **Figure 9** (Ozyildirim, 2005). A relatively new type of concrete called self-compacting concrete (SCC) does not require any vibration for its proper placement and compaction, **Figure 9**. Due to the application of relatively high volumes of fine fraction composed of cement and mineral additives and the use of chemical admixtures (such as superplasticisers and viscosity-modifying admixtures), SCC fills moulds without segregation or honeycombing (Wallevik, 2006). Different types of water-soluble polymers or nanosilica particles are often used as viscosity-modifying

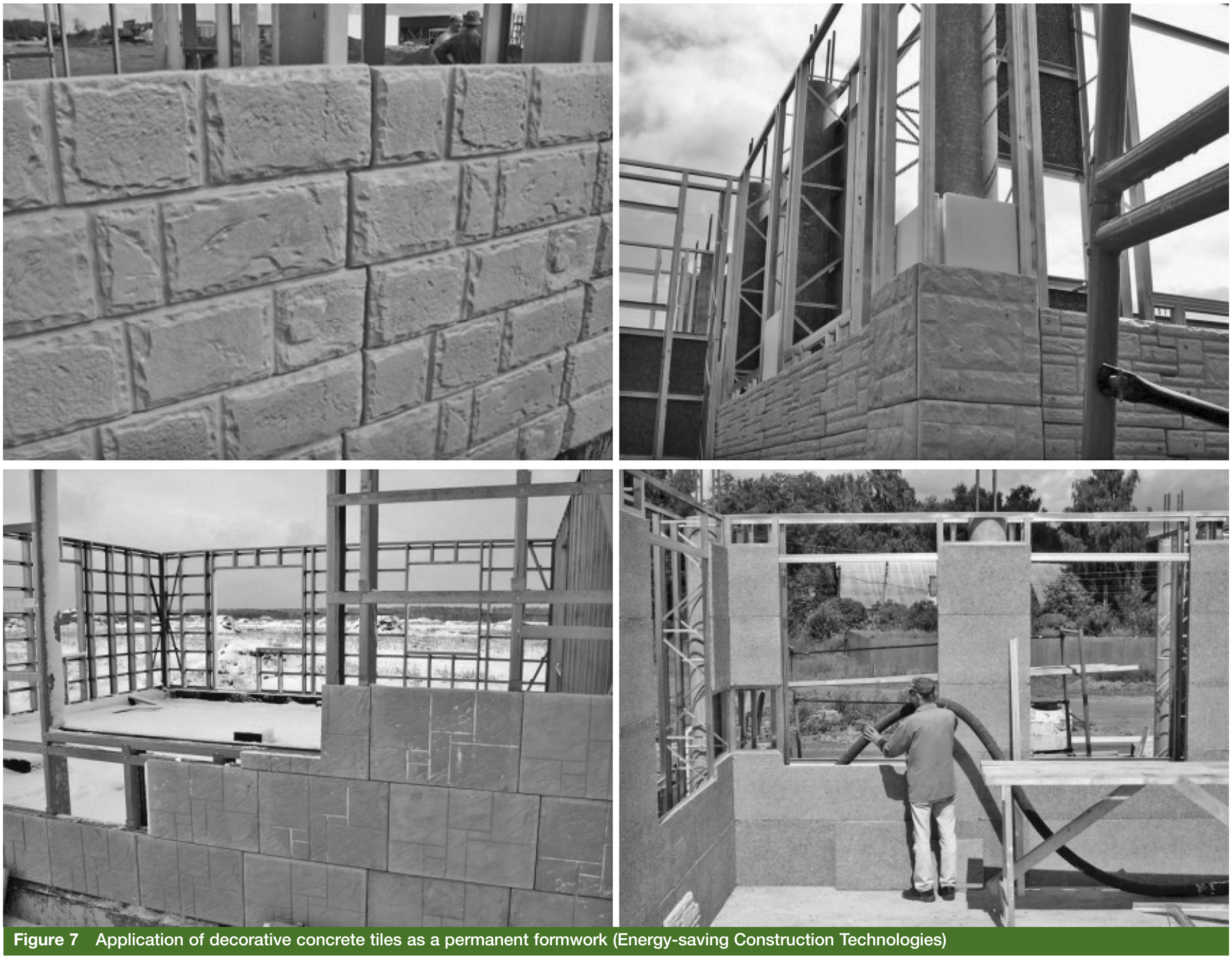


Figure 7 Application of decorative concrete tiles as a permanent formwork (Energy-saving Construction Technologies)

admixtures (Collepardi *et al.*, 2004; Green, 2006). A more expensive sibling of conventional concrete, SCC provides a quite effective high-tech solution in projects where vibration is not desired because of the noise or vibration restrictions or wherever high-speed concrete placement is necessary.

However, the most effective casting methods (in respect of strength per unit of cement used) involve heavy vibration of relatively dry, or low- and zero-slump concrete mixtures. Such concrete is designed for the least amount of cementitious paste and for a very low w/c ratio. Due to the limitations related to vibration levels and the application of rigid moulds, this method is mainly used for precast concrete**, Figure 8(d)**. Often, the industrial process is realised with a combination of high-intensity vibration and surcharge pressure with the aim of immediate mould release, **Figure 8(e)**. This approach is used for a number of mass products such as non-reinforced building blocks and paving stones. Hollow-core slabs and wall panels, including those based on lightweight concrete, are among the precast reinforced concrete elements manufactured using this method, **Figure 8(f)**. Roller-compacted concrete is another application of this technology.

Another industrial method relies on the centrifugal compaction of concrete mix. The process involves the placement and distribution of concrete mix within the cylindrical mould at low revolutions and consequently spinning the mould at high speed with acceleration reaching up to 30 g. Due to a high compaction gradient, excessive water (sometimes with lightweight fractions of the mix) migrates to the inner surface of the hollow concrete element and is discharged at the end of the process. Centrifugal compaction results in denser, stronger, impermeable and durable concrete elements. Such a process has become the standard to produce reinforced hollow-core concrete pipes, poles, masts and piles.

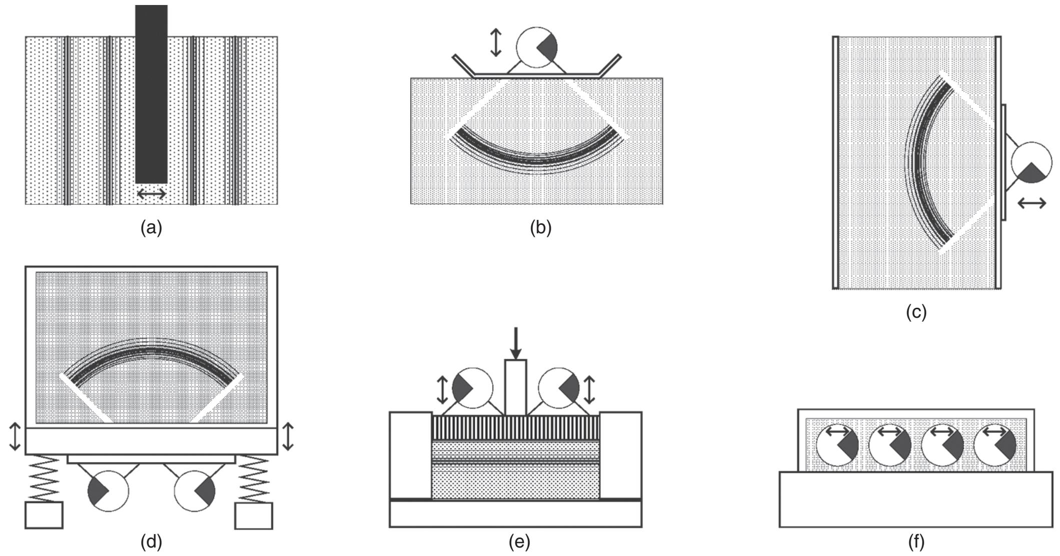


Figure 8 Vibratory methods: (a) internal vibration; (b) surface vibration; (c) form vibration; (d) table vibration; (e) vibro-pressing; (f) compaction of hollowcore plank (adapted from ACI Committee 309)

Effect of vibration and vibro-compaction

Vibration is a very effective method to liquefy the zero-slump or low workability concrete mixtures and also to compact plastic mixtures (ACI Committee 309). The compaction effect is achieved through reducing viscosity of the cement paste. Vibro-compaction involves the space rearrangement of aggregate particles to achieve the densest packing and forming of the element shape; the tixotropic fluidification of cement paste; de-aeration, damping and compaction of cement paste with the formation of a 'compact' gel structure (Ahverdov, 1981). During vibration, the internal friction of fresh concrete is reduced from 20 kPa to 1 kPa; this allows formation of the concrete element to the desired shape (ACI Committee 309). Depending on the frequency, on the amplitude, and the intensity of applied vibration, the required compaction degree can be achieved. Consolidation of fresh concrete requires the acceleration of 0.5–4 g at minimal amplitude of 0.04 mm (ACI Committee 309).

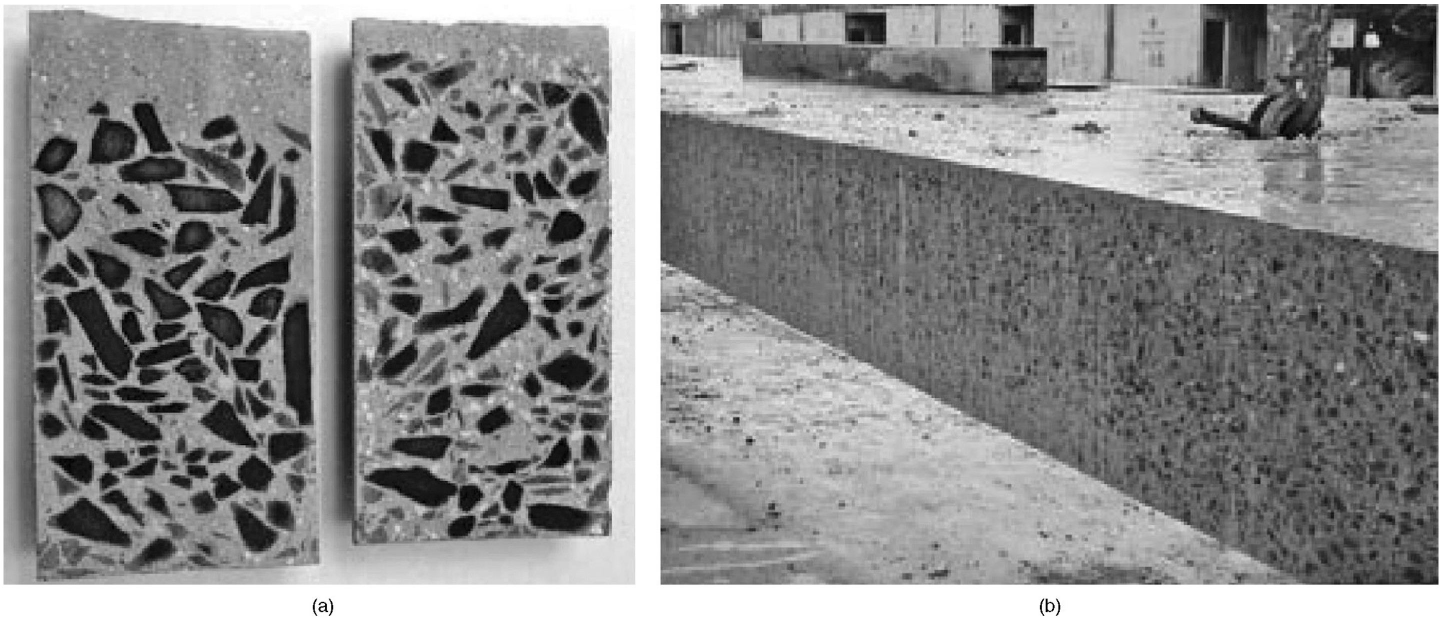

(a)
(b)

Figure 9 The effect of vibration on concrete segregation: (a) over-vibrated concrete; and (b) self-compacting concrete placed without vibration (C. Ozyildirim, Consolidation concerns continue, Concrete International, 2005)

Under vibration, the behaviour of sero-slump and plastic concrete mixtures is different because of the difference in composition and structure of these concretes. Zero-slump concrete is composed of large quantities of aggregates, often proportioned for the 'best' packing or even 'gap-gradings', with a relatively large fraction of coarse-size aggregate fraction (Sobolev and Amirjanov, 2004). As a result, densification of such mixtures is governed by the process of particulate material compaction (similar to the compaction of soil); this concrete requires vibration with high amplitudes and relatively low frequency (about 30–50 Hz) (Ahverdov, 1981). The complete compaction of such mixtures also requires the fluidification of cement paste (an essential step for a cement paste with low w/c of less than 0.4). The voids between the compacted aggregates are filled with cement paste liquefied by high-frequency vibration (up to 100 Hz) at a relatively small amplitude (Ahverdov, 1981). Such vibro-compaction (optionally, with surcharge load) is a very effective method for compaction of stiff concrete used in the precast industry (especially for the elements with instant form removal) and road construction (including slipforms and roller-compacted concrete).

For plastic concrete mixtures, the relative quantity of cement paste is high, so aggregates are suspended within the cement phase; therefore, the concrete flow and compactability are governed by the viscosity of cement paste. Here, vibration helps drop the fluidity of cement phase and, therefore, allows for the flow of aggregates under the vibration or gravity gradients. This case is the most common 'no-hassle' casting process; however, it requires special care to avoid segregation of concrete mixture due to overvibration (especially, for members of relatively high height, such as columns), **Figure 7**.

ACI Committee 309 summarises the conditions for the vibration consolidation of fresh concrete as follows (ACI Committee 309):

- minimum acceleration for concrete of normal consistencies;
- minimum dynamic pressure for very stiff concrete;
- minimum vibratory amplitude for any given mixture;
- minimum vibratory energy for all mixtures.

Depending on different applications, several vibratory methods are used to compact fresh concrete (**Figure 8**) (ACI Committee 309):

- *Internal vibration*: an internal vibrator immersed in the fresh concrete that generates rapidly recurring circular compression waves (**Figures 8(a), 8(f)**). The wave acceleration and amplitudes rapidly decrease with increasing distance from the vibrator, **Figure 10** (ACI Committee 309).
- *Surface vibration*: vibrating screeds capable of consolidating concrete slabs of up to 200 mm thickness (**Figure 8(b)**).

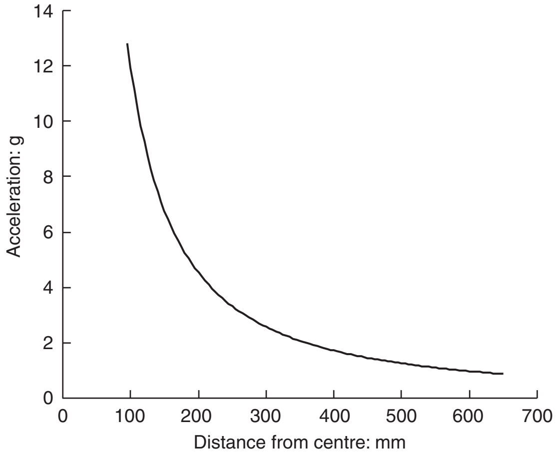


Figure 10 Efficiency of the internal vibration (adapted from ACI Committee 309)

- *Form vibration*: vibrators attached to the side of the mould or formwork at a distance of 1.5–2.5 m apart (**Figures 8(c), 8(e)**); to ensure efficiency, the compaction process is used at a minimum acceleration of up to 3 g.
- *Table vibration or shaking*: the most effective and complicated to design since the process involves interaction between the vibrating table, the forms and the concrete mix (**Figure 8(d)**). The compaction efficiency depends on the acceleration of the table, so accelerations up to 10 g and the frequency range of 50–100 Hz are used in many industrial applications.

Curing of cast-in-place concrete

The effect of curing conditions

Placing and compaction of concrete mix must be accompanied by adequate curing. Curing can be defined as temperature and moisture control procedures ensuring complete or favourable hydration of cement (ACI Committee 308 a and b). Hydration, the chemical reaction of cement with water, is responsible for concrete strength development, reduction of capillary porosity and permeability, and, therefore, is essential for the production of durable concrete with adequate mechanical characteristics (Bentz and Stutzman, 2006). Often, curing is referred to as 'normal curing', which is exposure to favourable environmental conditions with relative humidity of 95% and temperature of 20–23°C (differs depending on the governing standard). In practice, such conditions can be achieved and controlled only in a laboratory (e.g. curing rooms or environmental chambers). The 'real life' concrete is mixed, placed, and exposed to a wider range of environmental conditions (with temperatures ranging from −10 to +40°C and relative humidity (RH) of 30–95%). Hypothetically, in some particular projects, the combination of these environmental factors might be favourable to maintain the required conditions for concrete curing; however, the vast majority of applications require conditioned curing (Neville, 2000).

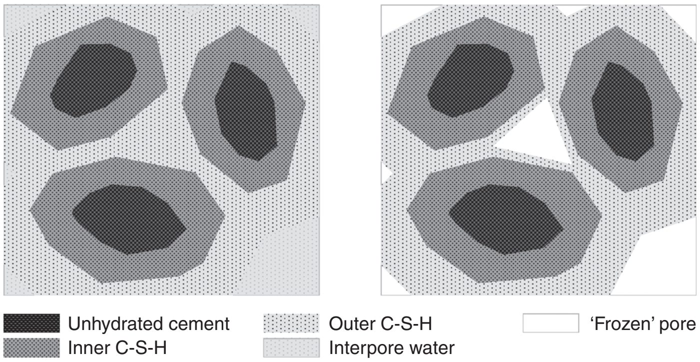


Figure 11 Microstructure model of hydrated cement paste (adapted from D. P. Bentz and P. E. Stutzman, Curing, hydration and microstructure of cement paste, ACI Materials Journal, 2006)

The most critical aspect of concrete curing is related to sufficient water supply for hydration of cement. Loss of water from hardened concrete adversely affects all the essential concrete properties: strength, shrinkage, abrasion resistance, permeability and other properties related to durability (Neville, 2000; ACI Committee 308 a, b; Bentz and Stutzman, 2006; Hooton *et al.*, 2002; Patel *et al.*, 1988; Aïtcin, 1998). In fact, lack of water available for hydration results in poor-quality concrete that will not be able to meet the project specifications and, in essence, may result in the failure of the entire project. Reduced service life and expensive repairs are common characteristics of projects with poorly cured concrete (Hooton *et al.*, 1947; Patel *et al.*, 1988).

In order to achieve the adequate curing, it is important to keep the concrete saturated until the water-filled space between the cement grains is sealed by the hydration products, **Figure 11**, left (Bentz and Stutzman, 2006). The hydration of cement is significantly reduced when the RH of capillary pores reaches 80% or less (ACI Committee 304; Powers, 1947; Patel *et al.*, 1988). In this case, a large-size 'frozen' pore network is formed, **Figure 11**, right. The RH within the concrete and favourable conditions for cement hydration depends on the following factors (ACI Committee 304):

- relative humidity and temperature of the ambient air;
- wind velocity;
- intensity of solar radiation.

The prescription of adequate curing depends also on the w/c ratio: at w/c = 0.42 and higher, the amount of mixing water is sufficient for continuous (and, eventually, complete) hydration of cement; in this case water loss from the hardened concrete must be prevented. For concrete with w/c of less than 0.42, considerable amounts of water are consumed by the hydrating cement; this results in undesirable self-desiccation (Aïtcin, 1998). Therefore, the latter case does require some additional water supply to the hardened concrete. Since the majority of modern concretes (including high-performance, high-strength, etc.) are designed for a w/c of less than 0.42 (not only because of the target strength but also to meet the durability requirements), water curing by external or internal method must be specified and meticulously executed. For example, recent applications of high-performance concrete in Canada use extensive water spraying (Aïtcin, 1998).

The magnitude of moisture loss from hardened concrete depends on the porosity of concrete and its air-void structure, so concrete with a large volume of entrained air, lightweight, or foamed concrete will have higher water loss compared to normal-weight concrete. Due to higher susceptibility to water evaporation, incomplete curing affects the performance of concrete with higher w/c and lower strength development, such as concrete with mineral additives (Neville, 2000). The same exposure conditions affect the structures with different surface-to-volume ratios (called surface module) in a different way. For bulky structures with low surface module (less than 0.1), moisture is lost only from relatively thin, (about 30 mm) surface layers; occasionally, the affected depth reaches 50 mm. This concrete layer is usually not considered in structural design; however, the existing codes rely on the adequate properties of this cover to protect the steel reinforcement. Therefore, the low permeability for aggressive chemicals, CO_2 and high pH are the key design parameters of such a layer.

The strength of specimens cured in fog or water ('normal' laboratory curing) is used as a basis for concrete evaluation, so such concrete is often compared with the performance of concrete cured in other environmental conditions (Neville, 2000). The strength of air-cured concrete can be as low as 65% the strength of water-cured concrete (Price, 1951; Moreno *et al.*, 2007), **Figure 12**. The duration of curing must be not less than three days; in this case the strength loss is only about 20% (for normal-weight concrete at w/c = 0.5). The duration of adequate curing depends on a number of factors (such as temperature, RH, w/c ratio, application of chemical or mineral additives); and for w/c = 0.5 it is considered to be seven days (AASHTO, 1998; Neville, 2000). ACI 318 Building Code requires a seven-day curing period for concrete based on Type I cement (ACI 318-95). In this case the difference between the strength of field and 28-day moist cured concretes is negligible. It should be noted that moist-cured concrete exhibits continuous strength development up to 180 days, while the air-cured concrete (even after the adequate moist curing) experiences up to 10% strength loss during the same period (Neville, 2000). Based on the concrete two-day to 28-day strength ratio, the guidelines for minimal curing duration were developed by RILEM (**Table 1**) (ACI 318-95; Van der Molen, 1979; European Committee for Standardization, 1997; Poole, 2006; USACE CWGS 03300).

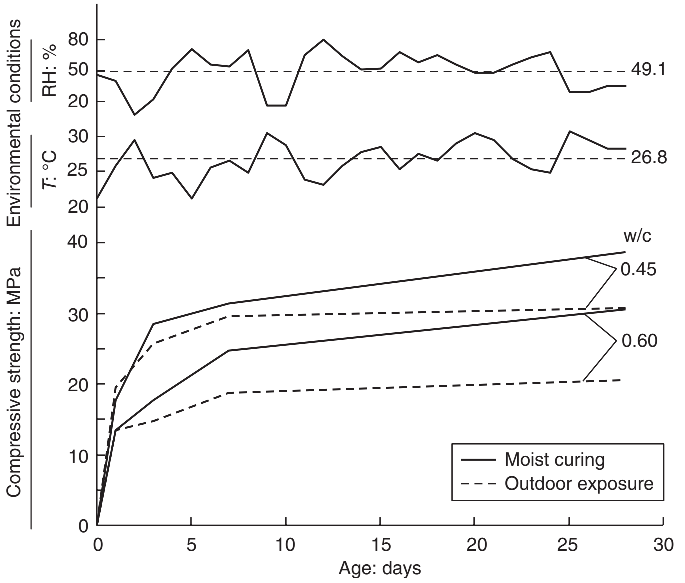


Figure 12 Effect of curing conditions on strength of concrete (adapted from Moreno *et al*., 2007)

Types of curing methods

It is generally considered that the formwork is adequate to prevent moisture loss from the vertical surfaces, but all open surfaces are prone to moisture loss and, therefore, require protection cover or water supply. There are two groups of curing methods (Price, 1951; AASHTO, 1998; Neville, 2000; Kovler and Jensen, 2005; Toy, 2005, 2006; Moreno *et al*., 2007):

- *External*: traditionally applied as: (a) water curing (i.e. water pounding, spraying, misting, or saturated coverings); and (b) sealed curing (when moisture loss is prevented by a water barrier such as curing membrane or water-resistant sheeting).
- *Internal*: with (a) water supply from lightweight aggregates or super-absorbent polymers; and (b) internal sealing with water-soluble chemicals.

The most common methods involve spraying of water, or the application of wet burlap mats, wet sand or sawdust on the surface of concrete. It has been suggested that the quantity of curing water is roughly estimated to be the same as the mixing water. Importantly, water used for curing must be essentially the same quality as the mixing water to avoid discoloration and stains; it must be free from the substances that attack the concrete or reinforcement.

Surface concrete temperature (T), °C	Minimum curing period (days) for two-day to 28-day strength development ratio (r)			
	$r \geqslant 0.50$ rapid	$r \geqslant 0.30$ medium	$r \geqslant 0.15$ slow	$r < 0.15$ very slow
$T \geqslant 25$	1.0	1.5	2.0	3.0
$25 > T \geqslant 15$	1.0	2.0	3.0	5.0
$15 > T \geqslant 10$	2.0	4.0	7.0	10.0
$10 > T \geqslant 5$	3.0	6.0	10.0	15.0

Table 1 Minimum curing period for EN 206 exposure classes other than XO and XC1[37]. Note: XO and XC1 refers to European Committee for Standardization, 1997 (Toy S. Poole, Guide for Curing Portland Cement Concrete Pavements, Vol. 2, 2006)

Membrane curing methods that involve the formation of a water barrier or sealant are effective to avoid moisture loss from concrete and are extensively used to avoid plastic shrinkage. Such membranes are based on hydrocarbon resins, acrylic of butadiene latexes, or wax emulsions and are often applied by spraying on the surface of concrete. The spraying is usually applied after bleeding is stopped, but before the surface has dried out (Neville, 2000; ACI Committee 308 b); for concrete with negligible bleeding (such as silica fume concrete), such a membrane must be applied immediately. Importantly, the curing aid should not be sprayed on dried concrete; such concrete must be wetted prior to application of the curing compound.

Internal water curing (or water entrainment) is a relatively new method, which has actually been realised as long as concrete with lightweight aggregates (LWA) is used (Kovler and Jensen, 2005; Campbell and Tobin, 1967). In such concrete, LWA are usually pre-saturated with water (to achieve the saturated surface dry (SSD) condition) to avoid the workability loss (when part of the mixing water is consumed by porous aggregates). For example, Campbell and Tobin reported on the beneficial effect of LWA on curing of field concrete (Campbell and Tobin, 1967). The application of small quantities (5–7% of cement by weight) of sand-size LWA were found to be very effective achieving internal water curing and, consequently, to provide better conditions for cement hydration and to mitigate the autogeneous shrinkage. Different types of natural (pumice) and artificial (such as expanded clay aggregate or keramzite) LWAs were used for internal curing (Yeginobali *et al.*, 1998; Duran-Herrera *et al.*, 2007; Villarreal and Crocker, 2007).

Superabsorbent polymers (SAPs) were proposed in order to realise the internal curing of concrete (Kovler and Jensen, 2005). The absorbing capacity of SAPs in cement paste is about 20 times their weight, and the application of 0.3–0.6% of SAPs (by weight of cement) is sufficient to provide the adequate curing of concrete. Application of SAPs (which are used in a dry form) is also a very effective method to design an air void system of hardened cement paste. It is important to mention that the concrete strength loss due to the application of LWA or SAPs (due to the inclusion of additional voids) is compensated by the strength gain due to better hydration (Kovler and Jensen, 2005). Internal sealing is achieved with water-retention additives (usually water-soluble polymers), which reduce water evaporation from the surface of concrete (Kovler and Jensen, 2005).

Control of moisture loss

Excessive moisture loss from concrete, such as 1 kg/m^2 per hour and higher, results in a loss of integrity of the concrete surface layer and leads to plastic shrinkage cracking. In 1960, the Ready Mixed Concrete Association (USA) suggested the nomograph that became the standard for estimation of the evaporation rate, **Figure 13** (ACI Committee 305; Kosmatka *et al*., 2002; Snell and Radnaabazar, 2007).

Alternatively, the following equation may be used to calculate the evaporation rate (Snell and Radnaabazar, 2007):

$$E = 5[(T_c + 18)^{2.5} - r(T_a + 18)^{2.5}](v + 4) \times 10^{-6}$$

where

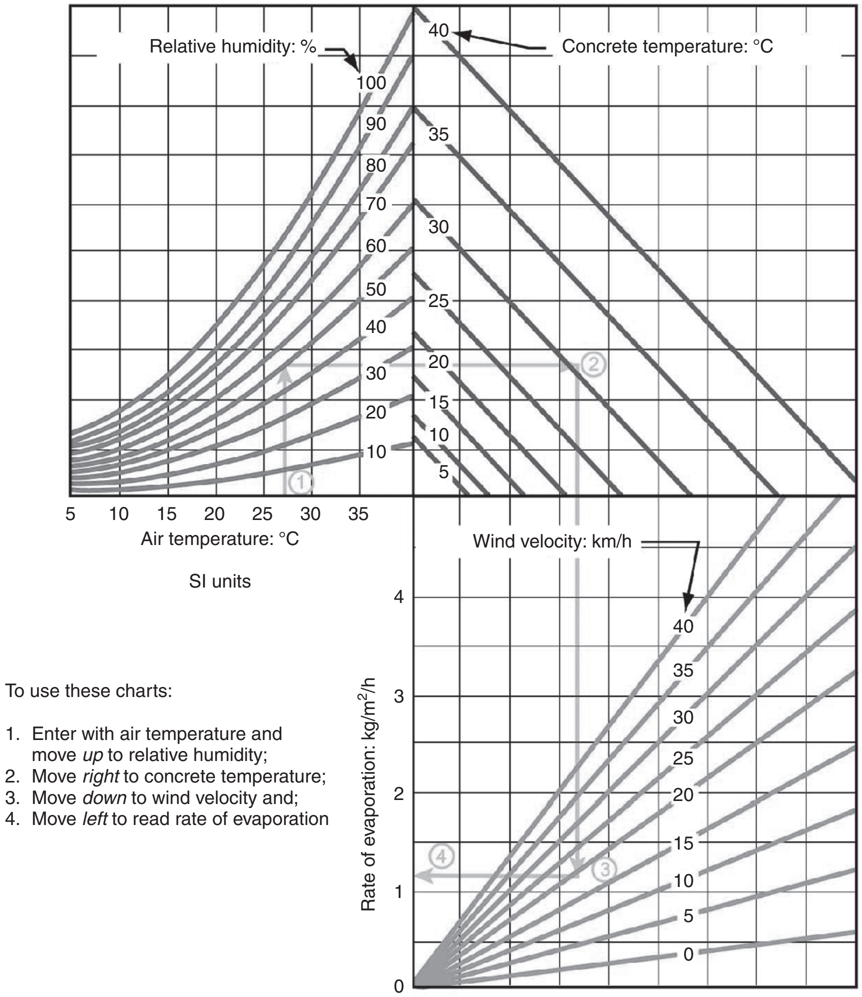
E evaporation rate, kg/m^2/h;
T_c, T_a temperature of concrete and air, respectively, °C;
r relative humidity in per cent/100;
v wind velocity at the level of 1 m above ground, km/h.

The site-specific parameters used in this equation can be obtained from portable weather stations.

Concrete bleeding provides the supply of additional water to the surface and helps to avoid the premature drying of the concrete surface. For concrete with a low bleeding rate (such as concrete with a high cement factor, with fly ash or silica fume), the critical evaporation rate is proportionally reduced; so these are known for their susceptibility to plastic shrinkage cracking. Synthetic fibres also reduce bleeding and these are very effective for controlling the cracking. Surface finishing is specified when bleeding water is evaporated, in order to avoid the formation of weak surface layers (Villarreal and Crocker, 2007).

Hot weather concreting

Hot weather concreting deals with mixing, delivery, placement and curing of concrete at elevated ambient temperatures of up to 35–40°C (ACI Committee 305; Schindler, 2004). If no adequate measures are taken, concrete



To use these charts:

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

Figure 13 Evaporation rate of water from a concrete surface (Snell and Radnaabazar, 2007; Kosmatka *et al*., 2002; ACI Committee 305) produced under such conditions will demonstrate the reduced setting times and quick slump loss; therefore, the transportation and placement of concrete in such conditions will require some corrections of mix design. A simple addition of water is the least effective measure to obtain the required workability level, since additional quantities of cement will be necessary to keep the w/c ratio constant. Chemical additives, such as plasticisers and super-plasticisers with a retarding effect or in combination with an effective retarding admixture, as well as secondary (two-stage) addition of these admixtures, can restore the workability required for concrete placement. The application of pozzolanic additives can be considered another safe method for avoiding the rapid hydration and loss of slump at elevated ambient temperatures.

Hot weather concreting is always concerned with undesired high moisture evaporation rates (ACI Committee 305; Neville, 2000; Kosmatka *et al.*, 2002; Snell and Radnaabazar, 2007). Here all the measures to keep the evaporation rate below the level of 1 kg/m^2 per hour are paramount to avoid plastic shrinkage cracking. These cracks are 0.1–3 mm in width and vary widely in their depth and length. Since high wind velocity (from 4.5 m/s) affects the evaporation, windshields and sun shades are effective measures controling the plastic shrinkage cracking.

Based on the concrete temperature and RH, ACI 305R-99 has set the following limits for concrete operations (ACI Committee 305):

- 41°C – 90%;
- 35°C – 70%;
- 24°C – 30%.

Another possibility for mitigating the problem of hot weather concreting is to reduce the temperature of the concrete, which may be achieved by adding crushed ice as a replacement for mixing water or by cooling the aggregates with dry ice or liquid nitrogen. The placement of concrete during the coolest period (night or early morning) was also recommended (Somayaji, 2001; Neville, 2000). In spite of accelerated hydration of cement and, subsequently, expected rapid strength development at elevated ambient temperatures, the lack of sufficient moisture supply (in the case of inadequate curing) and the rapid structure formation result in concrete of lower 28-day strength, **Figure 12**.

Cold weather concreting

Very little external moisture supply is required at low temperature operations. The main objective of cold weather concreting must be to maintain the sufficient temperature of concrete, so it keeps on developing strength (Neville, 2000). Temperatures of less than 20°C delay the hydration reaction of cement and, therefore, reduce the early strength of concrete. However, slower hydration at temperatures of 4–18°C has a beneficial effect on the formation of uniform microstructure (ACI Committee 305), and, therefore, results in better final strength of hydrated cement paste (if allowed to cure at 20°C afterwards). For example, concrete cured at 13°C, in spite of lower early strength, demonstrated the same strength as concrete cured at 23°C for 28 days; when exposed at 23°C after 28-day age, such concrete showed 20% strength increase over the reference, **Figure 14** (ACI Committee 306).

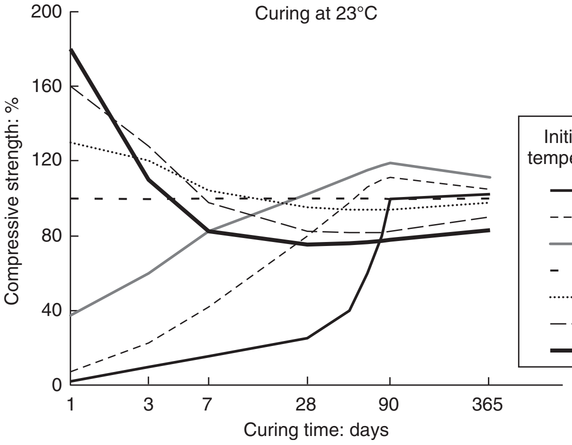


Figure 14 Effect of temperature on strength development (adapted from Price, 1951; ACI Committee 517)

Freezing of fresh concrete at an early age leads to formation of ice and subsequent expansion and destruction of concrete; therefore, it must be avoided. Concrete can be placed only if its temperature is more than 13°C in the case of thin sections and more than 5°C in the case of massive concrete (ACI Committee 306). This can be achieved by heating of the concrete ingredients (with hot water or steam), heat insulation and application of electrical blankets or propane heaters. Application of antifreeze admixtures (which are also accelerating admixtures, such as sodium nitrites and nitrates) that lower the freezing point of the mix and facilitate the strength development was found effective for cold weather concreting (Ramachandran, 1984). Importantly, calcium chloride should not be used as an antifreeze admixture since it initiates the corrosion of steel reinforcement.

Accelerated curing

Methods of accelerated curing

Like any chemical reaction, cement hydration is affected by temperature, so curing at elevated temperatures (up to 95°C) is often used to accelerate the strength development of concrete (**Figure 15**) and, consequently, to speed up the production process in the precast industry. High-temperature curing is always executed after the final setting of concrete is achieved, usually, not earlier than 2 hours. In order to avoid thermal cracking and strength loss, the

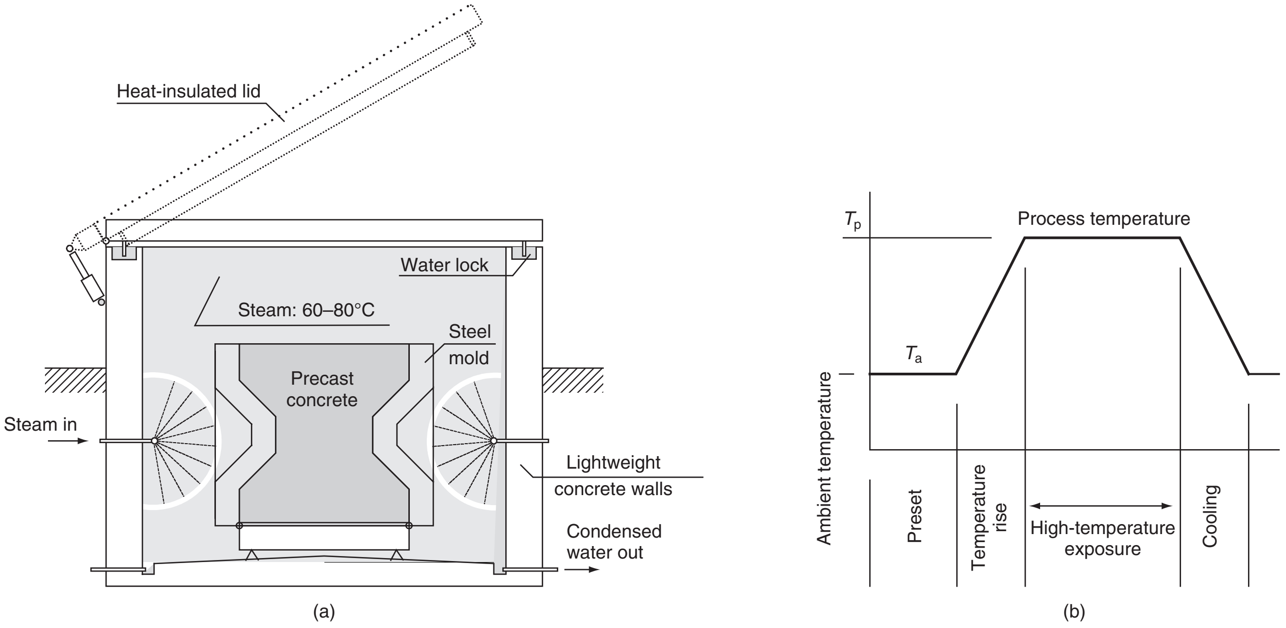


Figure 15 The design of the chamber for (a) accelerated curing of precast concrete and (b) typical curing regime

temperature increase must be relatively slow, of the order of 15–20°C per hour. Importantly, the high-temperature exposure period must be followed by cooling (**Figure 15**).

Accelerated curing of lightweight concrete must be executed even at lower thermal gradients, since the additional stresses due to the expansion of entrained air are generated. Alternatively, completely closed molds (with a top cover) are used to restrain the expansion of lightweight concrete. Temperatures of 80°C and higher are occasionally associated with delayed formation of the ettringite phase and a negative effect on durability; therefore, the accelerated curing is realised in steam media at 70–80°C; such process treatment helps to achieve about 70% of 28-day strength in 6–12 hours (Odler and Chen, 1996; ACI Committee 517).

The major drawback of accelerated curing is related to rapid hydration and formation of dense hydrated layers on the surface of cement particles, which prevent the subsequent diffusion of nearly hydrated products into the interstitial space and formation of 'frozen' pores (Neville, 2000; Verbeck and Helmuth, 1968). The hydrated cement paste has a non-uniform structure with denser agglomerates near the hydrating particles and porous C-S-H gel between the cement grains (**Figure 11**). Such a structure with weaker zones adversely affects the strength of the final product (vs. cured in laboratory conditions). As a result, accelerated curing is always a trade-off between the concrete rapid strength gain and loss of its ultimate strength.

The accelerated process is commonly realised in curing chambers (**Figure 15**) or thermo-forms with steam, hot water or oil used as heating agents. Electrical current or induction in the electro-magnetic field has also been used for heating and accelerated curing of concrete (Sokov *et al.*, 1990). Sometimes, steam chambers are operated at small pressure, up to 0.05 MPa, which allows the use of higher temperatures without adverse effects on concrete strength. A very critical parameter of curing at elevated temperatures is related to heating and heat transfer to concrete members. From one side, such a process must provide the homogenous heating of a structural member, and, from another side, the thermal stresses within the concrete element due to the temperature increase must be compensated by adequate strength development. Special care must be taken to ensure uniform distribution and recirculation of the steam within the chamber (especially over the height), ensuring even heat transfer to the element. The moulds must be designed to be sufficiently rigid to accommodate the thermal stresses without excessive deformations that may lead to changes in the dimensions and cracking of a reinforced concrete member. Autoclaving or high-pressure steam curing in the pressure vessel at a steam pressure of up to 0.8 MPa and a temperature of up to 180°C is commonly used to produce lightweight aerated concrete and lime-silica sand bricks. The application of autoclaving in curing concrete members is limited and used when high early strength, up to 100 MPa, improved resistance to sulfate attack, and reduced drying shrinkage are required.

Summary

The information presented in this chapter demonstrates that multiple factors related to the very initial stages of concrete preparation, placement, compaction and curing are essential for satisfactory concrete performance and, along with the initial design, determine the overall service life of structures. Technological aspects of mixing and processing super-plasticised, high-performance and self-compacting concretes are discussed. Different types of concrete formwork, including novel types of architectural concrete panels, controlled permeability, and insulated formworks are introduced. Several vibratory methods and compaction mechanisms governing the behaviour of sero-slump and plastic concrete mixtures are summarised.

This chapter demonstrates that the production of durable concrete with adequate mechanical characteristics and continuous strength development requires adequate curing. It emphasises that the loss of water from 'green' concrete adversely affects all essential concrete properties: strength, shrinkage, abrasion resistance, permeability and other properties related to durability. The strength of air-cured concrete at reduced humidity can be as low as 65% of the strength of water-cured concrete. The minimal curing requirements set by the ACI 318 Building Code and RILEM are presented.

Factors affecting the moisture loss from concrete and different curing methods such as traditionally applied external curing with water spraying, saturated coverings or curing membranes, as well as novel internal curing methods with lightweight aggregates and superabsorbent polymers have been discussed. Also discussed are the scientific background and the best practices related to curing of concrete at elevated ambient temperatures, cold weather concreting, and accelerated curing.

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CONTENTS

Chapter 12

Properties of fresh concrete

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Concrete in freshly mixed condition can be poured, pumped and moulded into a variety of forms and shapes. This chapter integrates information about numerous workability tests proposed and used by the concrete industry, testing laboratories and researchers, and the influence of constituent materials on workability. The workability test methods for concrete presented here cover a broad range of workability, from dry concrete to highly flowable concrete. Most test methods to measure the workability of traditional concrete have been standardised and the pre-European standards to measure the filling ability, passing ability and segregation of self-compacting concrete are included. In each section the test methods are described and one subsection is dedicated to the rheology of concrete. The air content and the bleeding and segregation of fresh concrete are also covered. A section at the end briefly describes the effect of types and properties of aggregates, water content and water-to-cementitious materials ratio, and the effect of chemical admixtures on workability.

Introduction

The 'plastic' nature of concrete in its fresh state enables it to be used for construction of complex structural shapes, which satisfy aesthetic requirements and provide the highest levels of structural efficiency. It is one of its most significant and unique characteristics in comparison with other major construction materials. The concrete industry had recognised the need to monitor concrete workability to ensure that concrete can be properly placed and compacted, and can achieve adequate hardened strength. Several test methods to measure the workability have been developed for research, mix proportioning and field use. With the exception of the widely used slump test, some test methods that have been studied extensively have generally failed to gain widespread acceptance. Even with the increase in knowledge in concrete rheology, no test has convinced the concrete industry to replace the slump test. The new emerging technologies of concrete, such as high-performance concrete and self-compacting concrete, are susceptible to small changes in mix proportions, which make monitoring the workability even more critical. The concrete industry is still faced with the challenge of developing a field test to measure quickly and accurately the relevant rheological parameters of concrete.

Workability of concrete

The term *workability* is broadly defined; no single test method measures all aspects of workability. The American Concrete Institute describes workability as 'the property of freshly mixed concrete or mortar which determines the ease and homogeneity with which it can be mixed, placed, consolidated, and finished' (ACI, 2000). Neville (1995) defines workability as 'the amount of useful internal work necessary to produce full compaction'. Workability depends not just on the properties of fresh concrete but also the nature of the application. The focus of workability measurement has changed many times over the years. For example, when the slump test was developed in the early twentieth century, concrete researchers were just beginning to recognise the importance of water content in predicting concrete strength (Abrams, 1922).

The scope of this chapter covers the test methods and apparatus for assessment of workability of fresh concrete. The focus is firmly on practical methods enabling a rapid and cost-effective assessment on a site or during production. However, tests that are suitable for laboratory use, namely for development of mix designs with specific requirements for workability, are also included. The chapter is concerned with the whole range of concretes, from traditional concrete, which had to be compacted by vibration, to self-compacting concrete, which can be designed to achieve practically any performance specification for its hardened state, to high-performance concretes. Some of the tests described are also capable of measuring rheological characteristics of the fresh mixes. The workability of test methods can be divided into categories based on several classification schemes. Tattersall (1991) broadly divided the assessment of workability into three classes which is summarised in **Table 1**.

Test methods to measure the workability of traditional concrete

Slump cone test

The test is widely used to assess the workability of medium-range fresh concrete mixes (BS EN 12350-2: 2000). The max-

Class I Qualitative	Workability, flowablity, compactability, finishability, pumpability, etc.	To be used only in a general descriptive way without any attempt to quantify
Class II Quantitative: empirical	Slump, compactor factor, Vebe, flow table, etc.	To be used as a simple quantitative statement of behaviour in a particular set of circumstances
Class III Quantitative: fundamental	Viscosity, mobility, fluidity, yield stress, etc.	To be used strictly in conformity with standard definitions

Table 1 Classes of workability measurement (adapted from Tattersall, 1991)

imum aggregate size is limited to approximately 40 mm. The slump test is used more often as a means of rapid and continuous checking to ensure uniformity of fresh concrete production or supply, rather than solely as a test for the assessment of workability. The test measures the difference in height of a fresh concrete sample moulded into the shape of a truncated cone before and after the mould has been removed. The internal dimensions of the mould, as shown in **Figure 1**, are 200 mm diameter at the base, 100 mm diameter at the top and a height of 300 mm. The mould is filled with concrete in three equal layers, and each layer is tamped 25 times with the tamping rod, while distributing the strokes uniformly over the surface of the concrete. One slump test requires a volume of 5.5 litres to fill the mould. Approximately 6 litres or a minimum mass of 14 kg of ordinary fresh concrete are therefore needed for a single test. The specifications of slump test are summarised in **Table 2**.

Four classes of workability have been mentioned in the European Standard BS EN 206-1: 2000 for concrete when assessed by the slump test (**Table 3**).

Compacting factor

The compacting factor test aims to measure the degree of compaction caused by an application of a standard amount of work to a sample of fresh concrete (**Figure 2**). The test is suitable for the assessment of fresh mixes of ‘medium’ workability. Its use covers approximately the same range of workability as the slump test. The preferred range of application is for mixes producing compacting factors between 0.70 and 0.95.

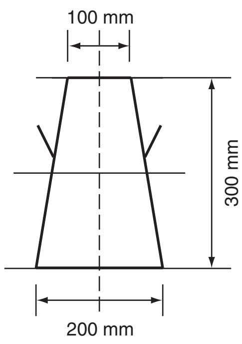


Figure 1 Dimensions of the mould for the slump test

a.	True slump	The sample of concrete settles but it retains a discernible symmetrical shape. There is a measurable difference between the height of the slump mould and the height of the slumped sample of fresh concrete.
b.	No slump	There is no measurable difference between the height of the mould and the height of the concrete sample after the removal of the mould (*zero slump*).
c.	Collapsed slump	The slumped concrete sample spreads out on the base and ceases to retain any clear, discernible traces of its original conical shape. *Collapsed slump* is recorded, without any numerical values or measurements.
d.	Shear slump	Part of the slumped concrete sample collapses; the remaining part retains the slumped shape. In such cases another slump test is carried out. If the additional test produces a true slump, the value is recorded and the shear slump is disregarded. If the additional test produces another shear slump it is recorded as the test result. It indicates a non-cohesive mix which may be unsuitable for assessment by the slump test.

Table 2 Specifications of slump

The compacting factor (CF, two decimal places) is obtained by using the following equation:

$$CF = \frac{m_\mathrm{p}}{m_\mathrm{f}}$$

where m_p is the weight of partially compact concrete and $\mathrm{m_f}$ is weight of fully compact concrete.

Vebe test

The test is based on the principle of measuring the time required to remould a sample of fresh concrete, cast in the shape of a slump test mould, into a cylindrical container of a larger diameter (BS EN 12350-3, 2000). Vibration and pressure assist the remoulding. The first stage of the test follows the procedure for the sump test, with a slump test being carried out inside the cylindrical test container (**Figure 3**). The test is applicable to low-workability, stiff, dry mixes, which show very low slump (less than 20 mm) or sero slump. The Vebe test is the most widely applied test in the production of precast, prestressed and some other high-strength and metallic-fibre-reinforced concrete

Class	Slump: mm*
S1	10 to 40
S2	50 to 90
S3	100 to 150
S4	160 to 210
S5	⩾220

Note: * The slump test result is rounded to the nearest 10 mm

Table 3 Classification of slump according to BS EN 206-1: 2000

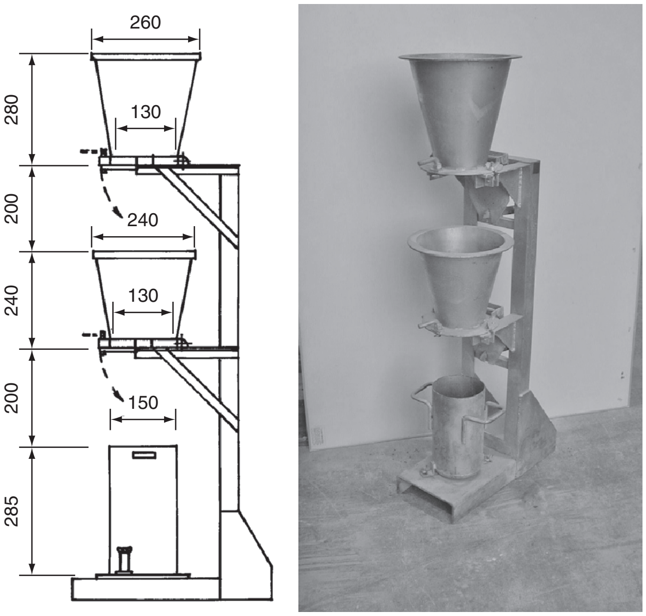


Figure 2 The compacting factor test apparatus: basic dimensions

products where low workability concrete mixes are used. The results of the Vebe test are assumed to reflect the amount of energy required to remould and compact a constant quantity of concrete by vibration.

The classification of concrete following the Vebe test according to BS EN 206-1: 2000 is given in **Table 4**.

Vebe classes

Class	Vebe in seconds
V0	⩾31
V1	30 to 21
V2	20 to 11
V3	10 to 6
V4	5 to 3

Table 4 Vebe classification (adapted from BS EN 206-1: 2000)

Compaction test

The test measures the volume of a concrete sample in a standard container before and after full compaction, preferably by vibration (Bartos *et al*., 2002). The distance from the surface of the compacted concrete to the upper edge of the container is used to determine the degree of compactability. The test is sometimes referred to as the compaction index test. The test is suitable for mixes with an apparent workability varying from very low to high. It is suitable for concrete with a maximum aggregate size of up to 63 mm and for lightweight aggregate concretes. The test is not suitable for mixes of very high workability or flowing concretes. The test is standardised by BS EN 12350-4: 2000. The apparatus comprises a rigid metal container of a square cross-section with a total volume of 16 litres (**Figure 4**). One test requires a minimum volume of approx. 18 litres or a mass of approx. 43 kg of ordinary concrete. The test is inexpensive, with very easily portable apparatus and the range of apparent workabilities which can be assessed by the test is not significantly different from that of the slump test. The classification of the degrees of compactibility according to BS EN 206-1: 2000 is indicated in **Table 5**.

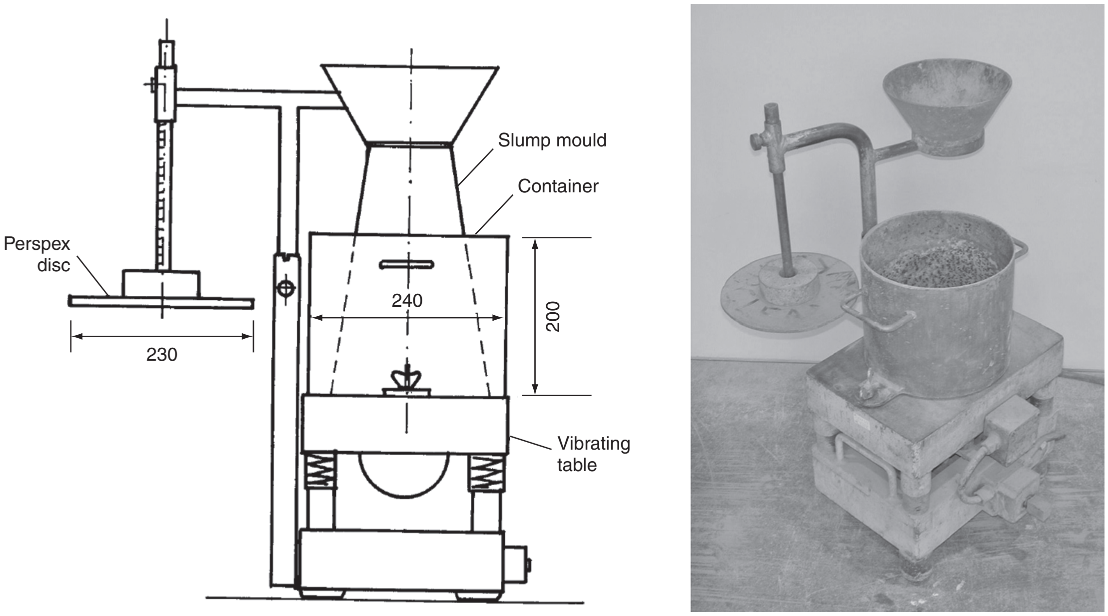


Figure 3 The apparatus for the Vebe test – basic parts and dimensions

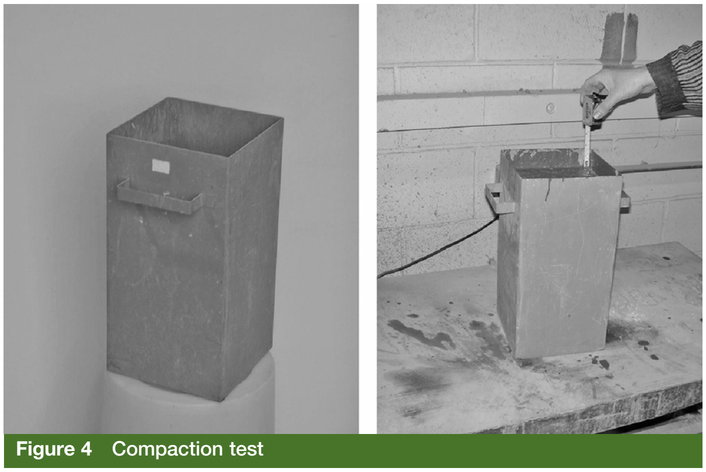


Figure 4 Compaction test

Class	Degrees of compactibility
C0	⩾1.46
C1	1.45 to 1.26
C2	1.25 to 1.11
C3	1.10 to 1.04
C4	<1.04*

Note: * Applied only to lightweight concrete

Table 5 Degree of compactibility (adapted from BS EN 206-1: 2000)

Flow/spread table test

This test measures the spread of a sample of fresh concrete after it has been moulded into the shape of a truncated cone and allowed to slump following the removal of the mould. The slumped concrete is then subjected to a controlled amount of jolting (BS EN 12350-5: 2000). The test was originally developed for the assessment of workability of medium-range concrete mixes and remained in use for such purposes in several European countries (Bartos *et al.*, 2002). It is widely used in Germany, its country of origin. The test can also be used for fresh mixes of high and very high workability, where collapsed slumps are recorded. This capability has increased the use of the test for assessment of superplasticised and other special flowing fresh mixes.

The apparatus consists of a flat, square (700 ± 2 mm × 700 ± 2 mm) plywood top plate, which has its upper surface lined with a metal sheet at least 1.5 mm thick. Centrelines at 90° are engraved on the surface of the metal lining, together with a concentric circle of 200 mm diameter (**Figure 5**). The plate is hinged at one end to a base, while on the other end, clips allow the plate to be lifted a vertical distance of 40 mm. Etched into the metal sheet are two perpendicular lines that cross in the centre of the plate and a 200 mm circle concentric with the centre plate. The frustum of a cone used to mould the concrete is shorter than the slump cone, with a top diameter of 130 mm and a bottom and height of 200 mm. The cone is placed in the centre of the plate and filled in two layers, each of which is compacted with a tamping rod. The plate is lifted, using the attached handle, a distance of 40 mm and then dropped a total of 15 times. The horizontal spread of the concrete is measured.

The spread/flow table test results have been correlated with the results of the slump test. However, there have been disagreements about the reliability of such correlations or the extent of validity of particular correlations. The classification of the spread/flow table test according to the European Standard BS EN 206-1: 2000 is presented in **Table 6**.

Figure 5 Flow table test

Class	Flow diameter: mm
F1	⩽340
F2	350 to 410
F3	420 to 480
F4	490 to 550
F5	560 to 620
F6	⩾630

Table 6 Spread/flow table (adapted from BS EN 206-1: 2000)

For concrete of high workability, e.g. when superplasticisers are used, the flow table test is appropriate. It has also been suggested that a stronger correlation exists between the values of an ordinary slump and the values of the initial spread – measured before jolting of the top plate. Despite its simplicity, the flow table apparatus is large and must be placed on firm and level ground. The jolting of the concrete does not accurately simulate filed practices and can lead to segregation, and in concrete construction practice such handling is normally avoided.

Consistence of fresh concrete

According to BS EN 206-1: 2000, due to the lack of sensitivity of the test methods beyond certain values of consistence, it is recommended to use the indicated tests for:

$10 \leqslant \text{slump} \leqslant 210\,\text{mm}$;
$\text{Vebe} \geqslant 30\,\text{sec}$ and $5\,\text{sec}$;
$1.04 \leqslant \text{degree of compactability} < 1.46$;
$340 < \text{flow table} \leqslant 620\,\text{mm}$.

The tolerance of the target values of consistence are given by BS EN 206-1: 2000 (see **Table 7**).

	Slump		
Target value: mm	⩽40	50 to 90	⩾100
Tolerance: mm	±10	±20	±30
	Vebe time		
Target value: sec	⩾11	10 to 6	⩽5
Tolerance: sec	±3	±2	±1
	Degree of compactibility		
Target value: sec	⩾1.26	1.25 to 1.11	⩽1.10
Tolerance: sec	±0.10	±0.08	±0.05
	Flow table diameter		
Target value: sec	All values		
Tolerance: sec	±30		

Table 7 Tolerances for target values of consistence

Test methods to measure the workability of highly flowable concrete

For highly flowable concrete, such as self-compacting concrete (SCC) or self-levelling concrete, other test methods were used to assess the workability. As SCC is concrete that is able to flow and compact on its own weight, its yield value is very low. The workability of fresh SCC can be divided into three key properties.

1. Filling ability: this is the ability of the SCC to flow into and fill completely all spaces within the formwork under its own weight.
2. Passing ability: this is the ability of the SCC to flow through tight openings such as spaces between steel reinforcing bars and between bars and formwork without hindrance and under its own weight.
3. Resistance to segregation: the SCC must meet the required levels of properties (1) and (2) whilst its composition remains uniform throughout the process of transport and placing.

Slump flow and T_{500} of self-compacting concrete

The slump flow test is a common procedure used to determine the horizontal free-flow characteristics of SCC in the absence of obstructions. It was first developed in Japan to characterise fluid concrete mixtures for placement under water. The procedure is based on DraftprEN 12350-8 for determining the slump flow of SCC. A sample of freshly mixed concrete is placed in a mould shaped as the frustum of a cone as used for the EN 12350-2 slump test. The concrete is placed in one lift and not compacted by any means of mechanical or manual agitation. The mould is raised, and the concrete allowed to subside. The slump flow consists of measuring the mean base diameter of the concrete sample at the end of the slump test which is expressed to the nearest 10 mm (**Figure 6**). Because of the viscous nature of SCC, the readings of the slump flow measurement were determined after there was no more discernible movement of the concrete, approximately 60 sec after the removal of the slump cone. The result is an indication of the filling ability of SCC. With the same test, the T_{500} slump flow time was measured when the concrete was slumping until it reached 500 mm of slump flow and is given as the nearest 0.5 sec. The T_{500} time is also a measure of the speed of flow and hence the viscosity of the SCC. T_{500} time is a test to assess the flowability and the flow rate of SCC in the absence of obstructions. The test is easy to perform either at a concrete plant or on site.

The European guidelines for SCC have classified four classes of slump flow:

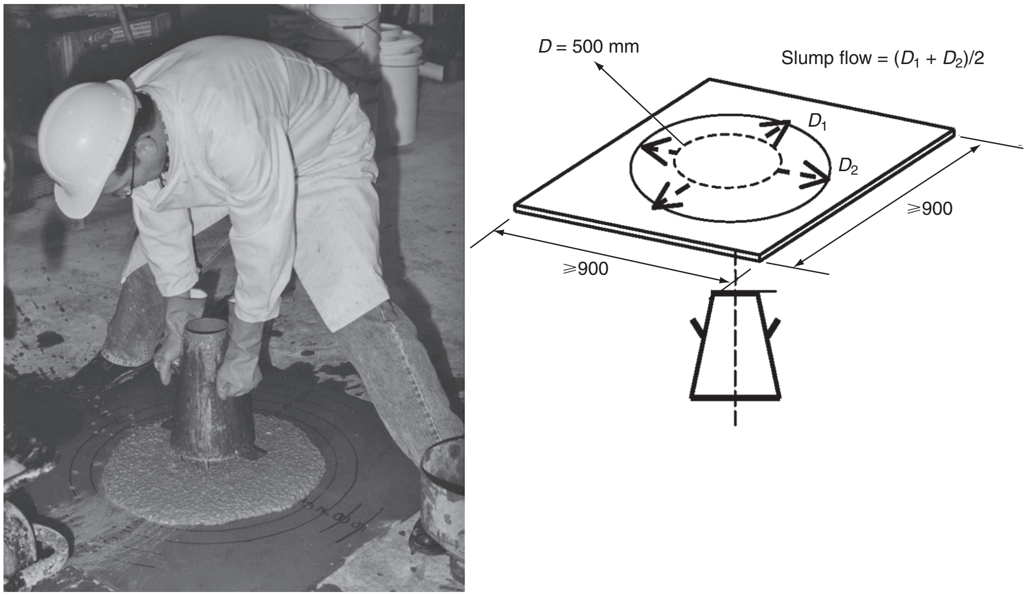


Figure 6 Slump flow test and T_{500} time

Class	Slump flow (mm)
SF1	550 to 650
SF2	660 to 750
SF3	760 to 850

V-funnel test

The V-funnel test is described by DraftPrEN 12350-9 and is not suitable when the maximum size of the aggregate exceeds 20 mm. The V-funnel time is the period of a defined volume of SCC passing through narrow-opening passing ability, which involves viscosity of freshly mixed SCC from observation of the flow speed of the sample through the specially designed funnel under self-weight. Dimensions of the V-funnel are given in **Figure 7**. This test also covers evaluation of the segregation resistance of freshly mixed SCC by the observation of the variation on the flow speed due to the difference of the sample's remaining period in the funnel.

According to the European guidelines for SCC, two viscosity classes with T_{500} and V-funnel time have been

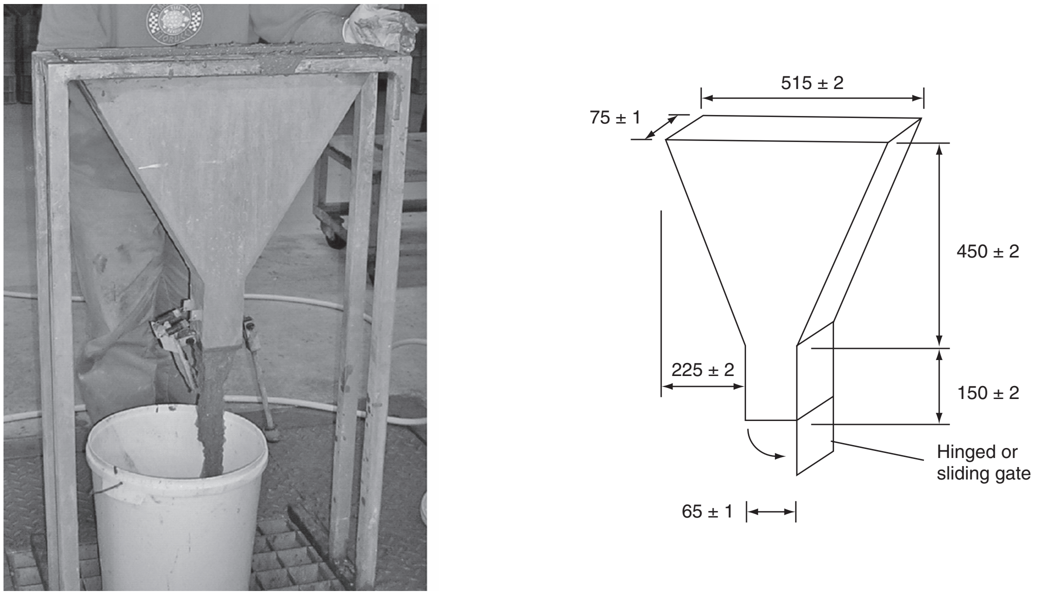


Figure 7 V-funnel test

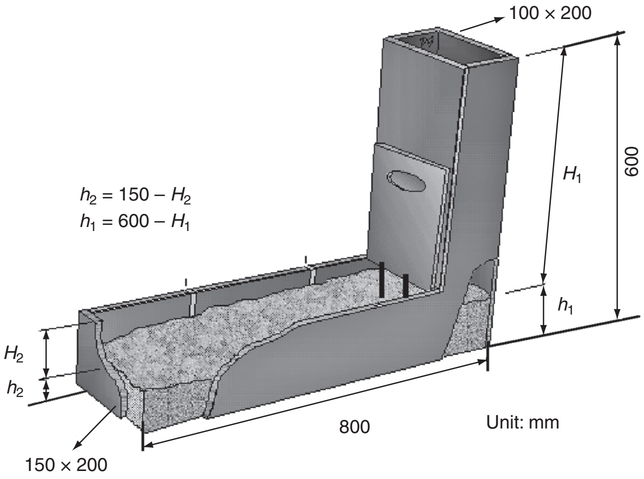


Figure 8 L-box test

established:

Class	T_{500} (s)	V-funnel time (s)
VS1/VF1	$\leqslant 2$	$\leqslant 8$
VS2/VF2	>2	9 to 25

L-box test

The L-box aims to investigate the passing ability of self-compacting concrete; its dimensions are shown in **Figure 8**. The L-box is described by DraftPrEN 12350-10. With the L-box it is possible to measure different properties such as blocking and segregation. The vertical part of the box is filled with 12.7 litres of concrete, which are left to rest for 1 min to allow any internal segregation to occur. After that, the gate is opened and the concrete flows out of the vertical part into the horizontal part through the reinforcement bars. The gap between the reinforcement bars was 41 ± 1 mm and 59 ± 1 mm for 10 mm and 20 mm coarse aggregate, respectively. Three smooth steel bars and two bars (12 mm) were used in the L-box for 10 and 20 mm coarse aggregate concrete, respectively. The height H_1 and H_2 of concrete were measured and used to determine the passing ability ratio h_2/h_1 to the nearest 0.05 (**Figure 8**).

The European guidelines for SCC have classified two classes according the passing ability:

Class	Passing ability
PA1	$\geqslant$0.80 with 2 rebars
PA2	$\geqslant$0.80 with 3 rebars

J-ring test

The J-ring test aims to investigate the passing ability of SCC to flow though tight openings including spaces between reinforcing bars and other obstructions without segregation or blocking (Sonebi *et al*., 1999). The J-ring test is described by (DraftPrEN 12350-12). The J-ring test is used to determine the blocking characteristics of SCC. The equipment consists of a ring placed on several rebars with adaptable gap widths, combined to the Abram's cone (**Figure 9**). The apparatus had a diameter of 300 mm with a fixed spacing of vertical bars. The apparatus has a ring of a

Figure 9. J-ring combined to slump cone

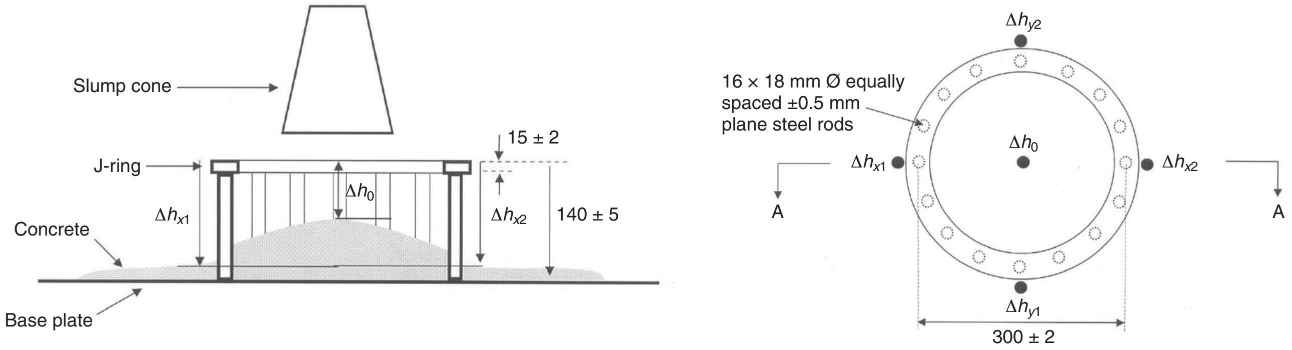


Figure 10 Section AA across and narrow gap of the J-ring test

rectangular cross-section in which there is a narrow gap (bar spacing approximately 41 mm) with dimensions as shown in **Figure 10**. The wide-gap J-ring had bar spacing of approximately 59 mm. It can be noted that other combinations of the J-ring's gap spacing may be used. Note that the slump cone may have the feet removed in order for it to fit inside the J-ring. T_{500J} time can be also measured when the concrete touches the circle of 500 mm diameter, if this has been requested.

The mean of two perpendicular diameters is calculated (d_m and d_r). The concrete surface at the central position Δh_0 and at the four positions outside the J-ring, two Δh_{x1}, Δh_{x2} in the x-direction and the other two Δh_{y1}, Δh_{y2} in the y-direction (perpendicular to x) are measured (**Figure 10**).

The J-ring flow spread SF_J is expressed to the nearest 10 mm given by the following equation:

$$SF = \frac{(d_m + d_r)}{2}$$

where:

SF_J	flow spread (mm)
d_m	largest diameter of flow spread (mm)
d_r	largest diameter at 90° to d_m (mm)

The J-ring blocking step B_J is calculated using the following equation:

$$B_J = \frac{(\Delta h_{x1} + \Delta h_{x2} + \Delta h_{y1} + \Delta h_{y2})}{4} - \Delta h_0$$

where:

B_J	blocking step
Δh	height

Sieving segregation test

The sieving segregation test is used to assess the resistance of SCC (Bartos *et al.*, 2002; Cussigh *et al.*, 2003). The test is described by Draft Pr EN 12350-11. This test is not applicable to concrete containing fibres or lightweight aggregate. The sieve segregation test is shown in **Figure 11**. The test aims to determine how likely an SCC mix is to segregate by allowing a 10 litre concrete sample to undergo static segregation for 15 min (in a bucket). Then the top layer of the sample (4.8 kg ± 0.2) is poured onto a 5 mm sieve and some mortar passes through the sieve. After 2 min the weight of material that has passed through the sieve is recorded. The segregation ratio is then calculated as the proportion of the sample and the material passing through the sieve. More mortar passing through the sieve indicates a greater liability to segregation. The segregation portion SR is calculated as follows:

$$SR = \frac{(m_{ps} - m_p) \times 100}{m_c} (\%)$$

where:

SR	segregated portion
m_{ps}	mass of sieve receiver plus passed material (g)
m_p	mass of sieve receiver (g)
m_c	initial mass of concrete place onto the sieve (g)

Figure 11 Sieving segregation test

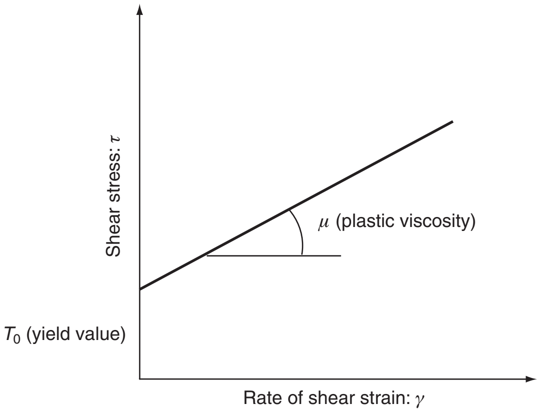


Figure 12 Variation of shear stress vs rate of shear strain

The European guidelines for SCC have classified two classes according to segregation resistance (sieve segregation) as follows:

Class	Segregation resistance (%)
SR1	$\leqslant 20$
SR2	$\leqslant 15$

Rheology test

Rheology is the science of the flow and deformation of materials, and it is concerned with the relationship between shear stress, shear strains and time. Its application to the flow properties of fresh concrete has been extensively documented (Tattersall *et al.*, 1983; Tattersall, 1991; Bartos *et al.*, 2002; Sonebi *et al.*, 2007). There is general agreement that concrete behaves as the Bingham model as follows:

$$\tau = \tau_0 + \mu\dot{\gamma}$$

Where τ_0 is the yield value which concrete can resist stresses not exceeding the yield stress τ_0 without flowing (overcome interparticle attractive effects), μ is the plastic viscosity and $\dot{\gamma}$ is the shear rate.

There is linear relationship between τ and μ. Thus, the two rheological parameters τ and μ characterise the behaviour of concrete (**Figure 12**). For example, a two-point workability test was used to determine the rheological parameters (**Figure 13**). Measurements of shear stress at two points, or at two shear rates, were therefore required to define its behaviour (Tattersall, 1991). Other rheometers such as coaxial rheometer BML, BTRheom and MKIII Tattersal can be used to determine the yield stress and the plastic viscosity (Ferraris *et al.*, 2003).

Knowledge of the variation of the two Bingham parameters with mix constituents and proportions provides a much greater understanding of the fresh concrete's behaviour than can be obtained with single point tests. For example, the nature of the changes in workability due to admixtures, particularly plasticisers and super-plasticisers and cement replacement materials (e.g. pulverised fuel ash (PFA)), can be distinguished (Sonebi *et al.*, 2003). Two particular uses are suggested:

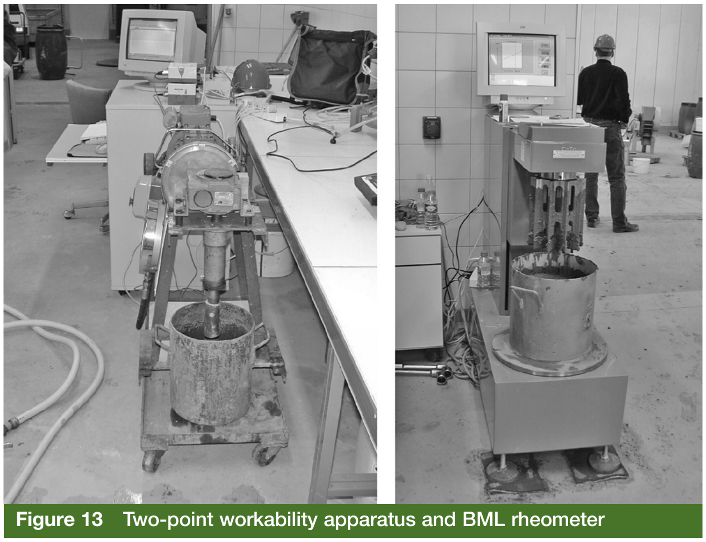


Figure 13 Two-point workability apparatus and BML rheometer

- Quality control of all types of fresh concrete, e.g. to determine the cause of an abnormally high workability.
- Development and assessment of all types of high-performance concrete, especially those where rheology is of prime importance, e.g. self-compacting concrete.

The yield stress is a measure of the behaviour of the concrete under self-weight, and has been shown in several studies to be reasonably well correlated to slump, which is a self-weight test. The plastic viscosity contributes to the handling characteristics under higher rates of shear strain, e.g. during pouring and compacting, and relationships between the two parameters and the performance of the concrete in practical processes have been established (BS EN 12350-2: 2000). During compaction by vibration, the vibration energy must be sufficient to overcome the yield stress, and the subsequent vibration response can be related to the plastic viscosity.

Air content

There are two standardised test methods for determining the air content in fresh concrete: the pressure method and water column method. The pressure method BS EN 12350-7-2000 can be used as the conventional method for measuring air in fresh concrete (**Figure 14**). This method gives only the air content without any information about the parameters that determine the quality of the air–void system. The size and number of voids and spacing between them can be measured on a polished sample of hardened concrete, according ASTM C457. Air entrained in concrete is produced by adding an air-entraining admixture during the mixing process. The air-entraining stabilises bubbles

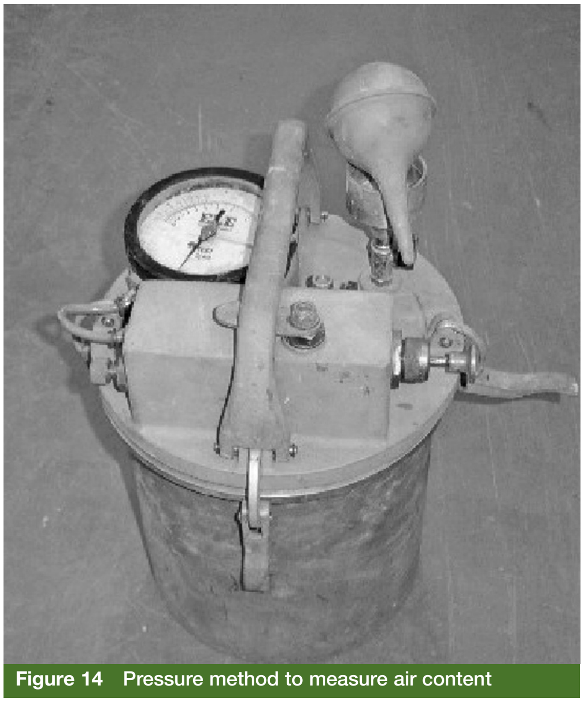


Figure 14 Pressure method to measure air content

formed during the mixing process on concrete, and therefore enhances the incorporation of bubbles of various sizes by lowering the surface tension of mixing water, which impedes bubble coalescence, and anchors bubbles to cement and aggregate particles.

Entrained air usually enhances the workability of concrete. It is particularly effective in low binder content mixes that otherwise might be harsh and difficult to work. The slump is increased around 25 mm for each 0.5–1% increase in air. The water demand of wet concrete for equal slump is decreased by approximately 3–6 kg/m^3 per percentage point increase in air. The resistance of hardened concrete subjected to freeze–thaw in a moist condition is significantly improved by using entrained air. The increase of air content in concrete by 1% led to a reduction of approximately 2–6%, and from 2–4% for flexure strength. The modulus of elasticity is decreased by increasing the air in concrete. Using air entrained also improves the bleeding and the sulfate resistance. The thermal conductivity is decreased by 1–3% per percentage point of increase in air. It was reported that the increase of air had little effect on the permeability, drying shrinkage, creep and abrasion.

Bleeding and segregation

Bleeding

Bleeding can be described as when water within the fresh mix separates from the mix and rises to the surface of the concrete, and, as water, is the constituent of lowest density in concrete, it will tend to rise to the surface of the fresh material (Neville, 1995). Bleeding can be related to Betancourt's first assumption that a solid denser than a liquid tends to sink into it. The bleed water becomes separated from the cement in the cement paste and, because it has a lower density than the solid, the bleed water rises upwards.

The bleed water is the clear water that can gradually accumulate at the top surface of freshly placed concrete, mortar, grout or cement paste. This bleeding is caused by sedimentation or settlement of solid particles (aggregates and cement) and the simultaneous upward migration of water. The bleed water can rise to the surface with other fines particles from the matrix which forms on the horizontal surfaces of concrete, called laitance. This laitance trapped between successive layers of concrete results in weak and porous interface. A small amount of bleeding is normal and expected on freshly placed concrete. It does not necessarily have an adverse effect on the quality of the plastic or hardened concrete. However, excessive bleeding can affect the performance in plastic or hardened concrete. Desirable bleeding can be achieved with proper mix proportions, mix ingredients, and proper construction practices and placing equipment. The bleeding is not necessarily a harmful property nor is excessive bleeding desirable. In some deep applications, placed lean concrete can lead to a large amount of bleed water on the top surface. This not only creates a placing problem but also reduces the strength and durability performance of the concrete near the surface. Excessive bleeding also delays finishing. On other hand, lack of bleed water on concrete flat work can sometimes lead to plastic shrinkage cracking or a dry surface that results in a finishing problem.

Increase of bleeding can be achieved simply by increasing the dosage of water in concrete as well as reducing the dosage of the binder and the fines in sand. Bleed water of concrete placed in dry, warm and windy conditions will evaporate quickly, and result in plastic shrinkage and potential severe cracking. In this case, bleeding may need to be increased to help prevent the plastic shrinkage cracking or improve the finishing of concrete.

Reduced bleeding can be achieved in different ways:

- Reduce the water dosage, water-to-cementitious materials ratio (w/cm), and the slump.
- Increase the dosage of cement which results in reducing w/cm.
- Increase the fineness of the binder.
- Increase of the fines in sand.
- Use supplementary materials such as ground granulated blast-furnace slag, PFA or silica fume.
- Use blended cement (binary or ternary).
- Use chemical admixtures that reduce w/cm.
- Use air-entrained in concrete.

The bleeding of fresh concrete can be determined by two methods described in ASTM C232. One method consists of compacting a sample of concrete by tamping without further disturbance, and the second method consists of compacting concrete by vibration. The bleed water at the surface is expressed as the volume of bleed water per unit area of exposed concrete, or as a percentage of the net mixing water in the sample.

Segregation

In fresh concrete, segregation refers to the separation of one or more of the constituent materials from the main body when in its fresh state. Segregation can be defined as the separation of constituents of a heterogeneous mixture making their distribution no longer uniform (Neville, 1995). Segregation creates a non-uniform material and a situation where there is a higher proportion of the separated material in one section of the hardened concrete and a lower proportion in another section. Non-uniform distribution of constituent materials can lead to a hardened concrete which has an uneven distribution of properties, and in severe cases this could result in structural failure, but is more likely to result in poor durability which brings with it a high maintenance requirement and the likelihood of a reduced lifespan.

Segregation is the tendency for coarse aggregate to separate from the sand–cement mortar. This results in part of the batch having too little coarse aggregate and the remainder having too much. The former is likely to shrink more, crack and have poor resistance to abrasion. The latter may be too harsh for full compaction and finishing and is a frequent cause of honeycombing. The method and equipment used to transport and handle the concrete must not result in segregation of the concrete materials. When concrete is placed, there is segregation when mortar tends to flow ahead of coarser material. To avoid segregation, concrete should not be moved horizontally over too long a distance as it is being placed in forms or slabs.

Betancourt (1988) considered segregation of fresh concrete as a phenomenon related to viscosity and density of the cement paste. His explanation of segregation was based on two simple assumptions:

1. A solid denser than a liquid tends to sink in it.
2. A viscous liquid flows with difficulty around a solid.

For a solid material of given density, dispersed in cement paste, Betancourt defined a stability curve (**Figure 15**).

Betancourt's stability curve suggests that if the density of the paste is less than the density of the aggregate, and if the paste does not have sufficient viscosity, then the aggregate will sink within the paste. However, if the density and viscosity of the paste are too great, then the aggregate will float on the paste. Segregation in fresh concrete can manifest itself in various ways – in traditional vibrated concrete the most common forms are bleeding and honeycombing.

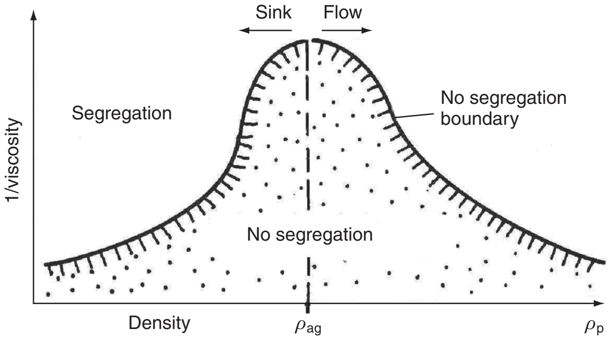


Figure 15 Betancourt's stability curve (adapted from Betancourt, 1988)

Chutes and drop-chutes are used to move concrete to lower elevations without segregation and spattering of mortar on reinforcement and forms. Slopes of chutes (vertical to horizontal) should range between 1 to 2 and 1 to 3. Factors that increase the risks of segregation are:

- Inadequate amount of fine aggregate (sand) in the mix.
- Inadequate dosage of cement.
- Big gap-grading of aggregate, particularly with maximum size of 40 mm or more.
- Higher water-to-cementitious materials ratio (w/cm). This results in a reduction to the viscosity and the cohesivity which leads to no-uniform distribution of aggregate in concrete.
- Low water-to-w/cm, resulting in dry mix with less paste to maintain the uniformity of the mix during handling.
- High amount of angular-shaped aggregate with crushed aggregates.
- Overdose of superplasticiser resulting in reducing the cohesivity of mix by decreasing mainly the yield stress.
- To enhance the segregation of the concrete mix, the following can be done:
  - Increases the amount of fines in the mix, such as the cement, fines in sand and supplementary materials.
  - Use a continuous grading of aggregate and smaller maximum size of aggregate.
  - Use air-entrained concrete.
  - Optimise water-to-binder ratio and amount of binder.

Sonebi *et al.* (2003, 2005, 2007) studied the effect of the variation of dosages of water and dosage of superplasticiser on segregation resistance of fresh SCC using three test methods: wet sieving test, settlement column segregation test and penetration test during the European project testing

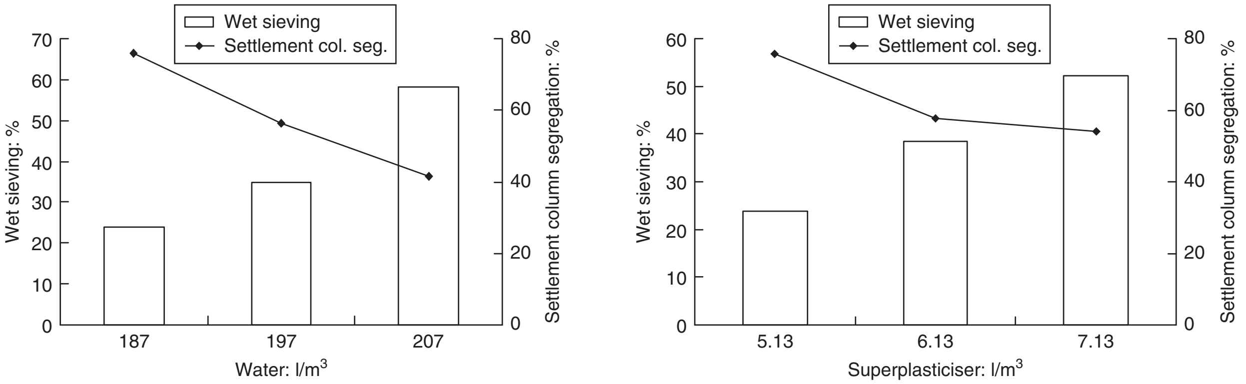


Figure 16 Effect of dosages of water and superplasticiser on wet sieving and settlement column test to assess segregation of SCC

SCC. **Figures 16** and **17** show that the increase of both dosages of water and super-plasticiser led to a reduction in segregation resistance and an increase of slump flow (Cussigh *et al*., 2003). The increase of water or superplasticiser dosage resulted in more laitance on the top of concrete measured by the wet sieving test, more penetration (**Figure 17**), and more coarse aggregates sink to the bottom of the settlement column test (ratio = total of coarse aggregate in top/total coarse aggregate in bottom) (**Figure 16**).

Figure 18 shows the effect of adding more dosage of superplasticiser on the segregation of fresh SCC. The SCC mix having more segregation exhibited more laitance at the top of the sample of concrete compared to stable SCC.

Influence of constituent materials

The biggest factor influencing the fresh concrete is water content. The workability of concrete is affected also by the aggregate properties, w/cm ratio, and the cementitious materials content. The elapse time when concrete mix and tested and the temperature of constituents of concrete affected the fresh properties. Some aggregate properties which affect the workability are nature of aggregate, maximum particle size, particle shape and surface texture, and the grading of aggregates. Chemical admixtures such as superplasticiser, retarder and viscosity-modifying agent significantly influenced the workability, the passing ability and segregation of fresh concrete (Khayat *et al*., 1996; Sonebi *et al.*, 2003; Sonebi *et al*., 2007).

Types of aggregate and properties

The particle size distribution of aggregate defines the basic surface area of all aggregates in concrete and determines the quantity of cement necessary to coat the aggregates with a layer sufficiently thick and workable in order to enhance the movement of all particles to achieve the required workability of fresh concrete.

The maximum sizes of aggregate mentioned in the design of normal concrete mixes (Teychenné *et al*., 1988) are 10 mm, 20 mm and 40 mm. **Table 8** shows the approximate free-water contents required to give various levels of workability where the workability is given in slump value (mm) and Vebe time (s) as a function of the type and maximum size of aggregate.

The grading of fine aggregate, particularly the percentage passing through 600 µm, is taking into consideration in the

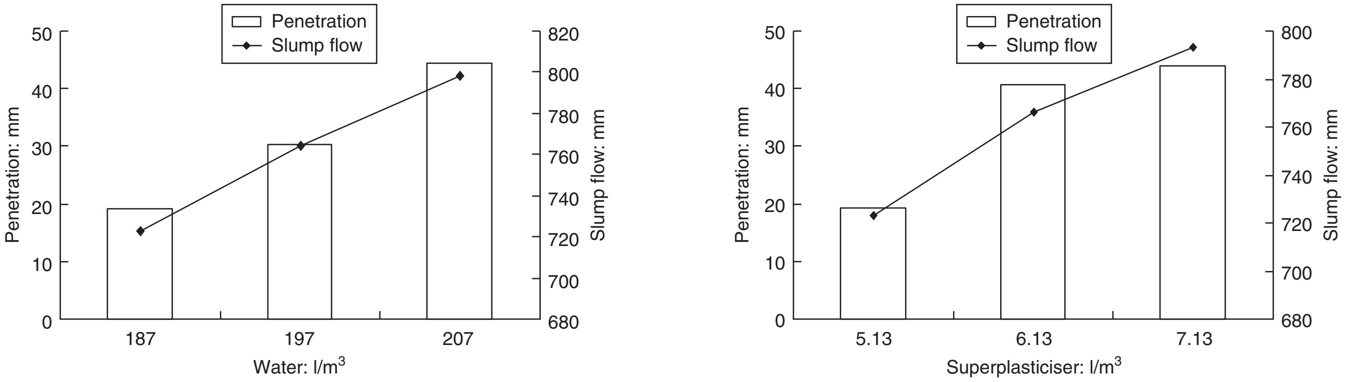


Figure 17 Effect of dosages of water and superplasticiser on wet sieving and settlement column test to assess segregation of SCC

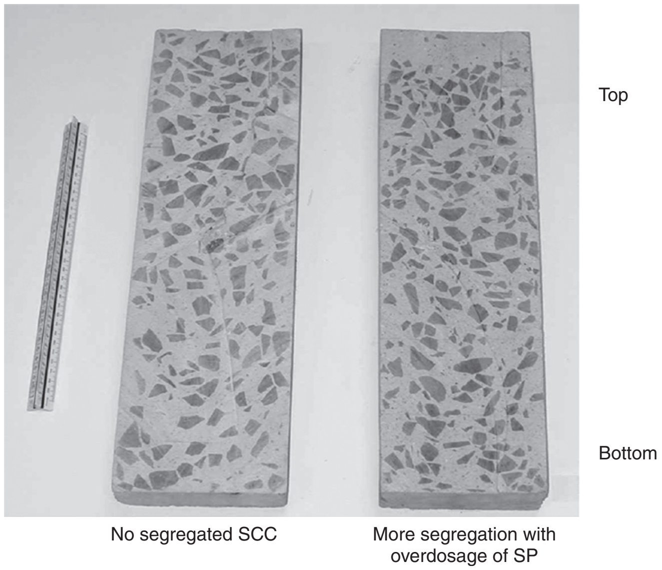


Figure 18 Effect of overdosage of superplasticiser on the segregation of SCC (Sonebi *et al*., 2007)

Slump: mm Vebe time: s		0–10 >12	10–30 6–12	30–60 3–6	60–180 0–3
Maximum size aggregate: mm	**Type of aggregate**				
10	Uncrushed	150	180	205	225
	Crushed	180	205	230	250
20	Uncrushed	135	160	180	195
	Crushed	170	190	210	225
40	Uncrushed	115	140	160	175
	Crushed	155	175	190	205

Note: When coarse aggregate and fine aggregates of different types are used, the free-water content is estimated by the expression:

$2/3W_f + 1/3W_c$

where

W_f free-water content appropriate to type of fine aggregate

W_c free-water content appropriate to type of coarse aggregate

Table 8 Approximate free-water contents required to give various levels of workability (adapted from Teychenné *et al*., 1988)

mix design of ordinary concrete (Teychenné *et al*., 1988). For a given maximum size of aggregate, workability and water/cement ratio, **Figure 19** presents the curves which allowed for determining the proportion of fine aggregate out of the total aggregate content which contains between 15% and 100% of particles passing through a 600 μm sieve.

The shape and texture of aggregates influence the workability of fresh concrete. The shape of coarse aggregate is determined in terms of the flakiness index from BS EN 933-3: 1997 in accordance with the relevant category for particular application or use. The shape index is also determined in accordance with BS EN 933-4: 2000 according to the particular application or use. Rounded or spherical with a smooth, glassy surface texture is recommended by the British Standard which in this case requires a minimum dosage of cement. The shape of the particles influences significantly the workability of fresh concrete. For example, the reduction of workability caused by the shape of coarse aggregate can be compensated by a combination of several modifications such as increase of dosage of cement, increase of fine particles, and use of chemical admixtures (superplasticiser, air-entrained).

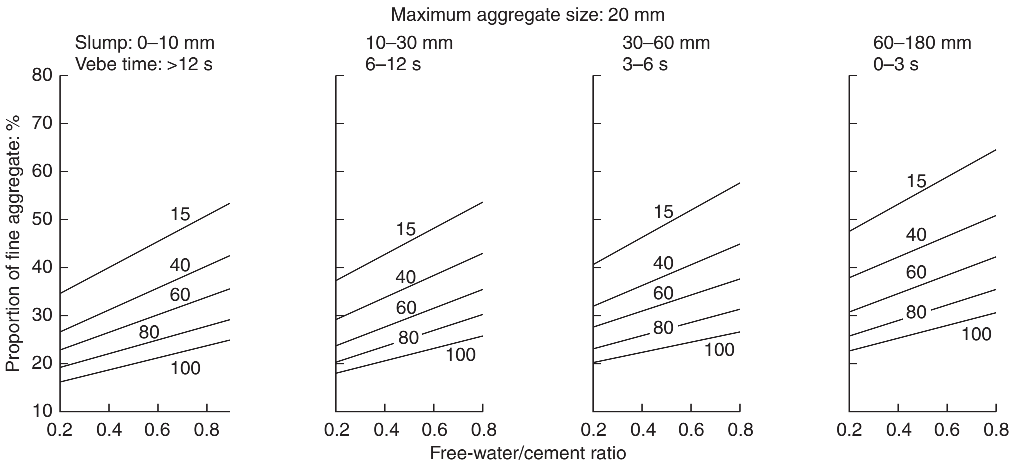


Figure 19 Recommended proportions of fine aggregate according to percentage passing a 600 μm sieve

Aggregate	Size	Percentage passing by mass					Category G^d
		$2D$	$1.4D^{a,b}$	D^c	d^b	$d/2^{a,b}$	
Coarse	$D/d \leqslant 2$ or $D \leqslant 11.2$ mm	100	98 to 100	85 to 99	0 to 20	0 to 5	$G_C85/20$
		100	98 to 100	88 to 99	0 to 20	0 to 5	$G_C80/20$
	$D/d > 2$ and $D > 112$ mm	100	98 to 100	90 to 99	0 to 15	0 to 5	$G_C90/15$
Fine	$D \leqslant 4$ mm and $d = 0$	100	95 to 100	85 to 99	–	–	G_F85
Natural graded 0/8	$D = 8$ mm and $d = 0$	100	98 to 100	90 to 99	–	–	$G_{NG}90$
All in	$D \leqslant 45$ mm and $d = 0$	100	98 to 100	90 to 99	–	–	G_A90
		100	98 to 100	85 to 99			G_A85

[a] Where the sieves calculated are not exact sieve numbers in the ISO 565:1990 R 20 series then the next nearest sieve size shall be adopted.

[b] For gap-graded concrete or other special uses, additional requirements may be specified.

[c] The percentage passing *D* may be greater than 99% by mass but in such cases the producer shall document and declare the typical grading including the sieves *D*, *d*, *d*/2 and sieves in the basic set plus set 1 or basic set plus set 2 intermediate between *d* and *D*. Sieves with a ratio less than 1.4 times the next lower sieve may be excluded.

[d] Other aggregate product standards have different requirements for categories.

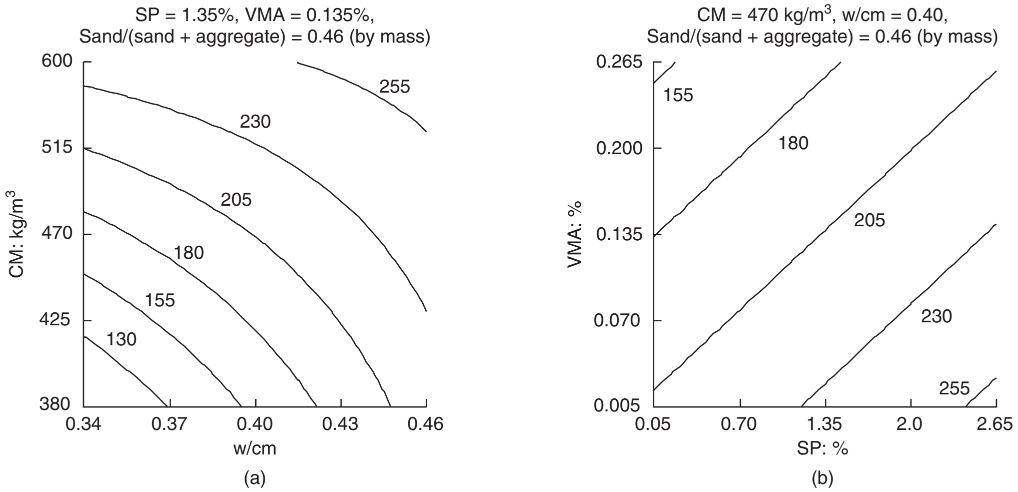
Table 9 General grading requirements (adapted from BS EN 12620: 2002)

The British Standard BS EN 12620: 2002 describes the aggregates for concrete, shown in **Table 9**, with their size design ratio *d*/*D* (lower/upper sieve sizes) and the selected category.

Effect of water content, w/cm ratio, volume of coarse aggregate and chemical admixtures on workability

The water demand of concrete is influenced by the type and the amount of the cementitious materials, the characteristics of aggregates, the moisture of aggregates and absorption, the required workability which is specified, and the ambient conditions in which the concrete will be cast.

Several researchers investigated the key parameters of the fresh properties of concrete using a factorial design approach. It was reported that slump is increased primarily by the increase in CM content and w/cm ratio (Khayat *et al.*, 1996). However, the increase of viscosity-modifying admixture (VMA) led to a reduction of workability. **Figure 20** presents the isocurves of slump with the variations of CM content and w/cm. The curves indicate that the increase of w/cm and the cementitious materials content of underwater concrete led to an increase of slump (Khayat *et al.*, 1996). However, the increase in VMA led to a reduction in workability, and conversely, the increase in dosage of superplasticiser (SP) resulted in an improvement of slump (**Figure 20**). **Figure 20** presents the isocurves of



(a)

(b)

Figure 20 Isocurves of slump (a) vs CM content and w/cm, and (b) vs SP and VMA (adapted from Khayat *et al.*, 1996)

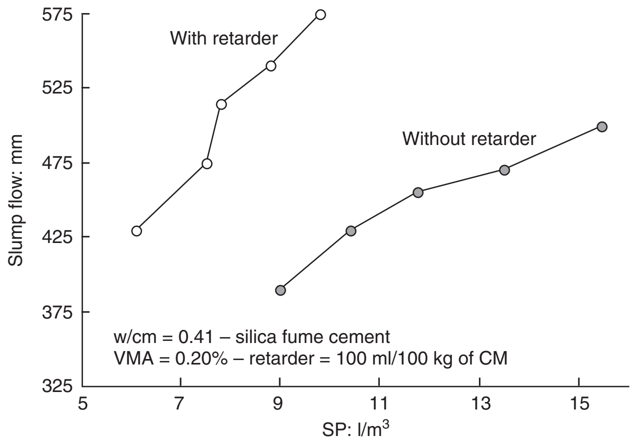


Figure 21 Effect of retarder agent on the workability and demand of super-plasticiser (adapted from Khayat *et al*., 1996)

slump with the variations of cementitious content and w/cm and the dosages of SP and VMA. The curves indicate that the increase of w/cm and the cementitious materials content of underwater concrete led to an increase of slump (**Figure 20(a)**) and the increase of SP resulted in an increase of the slump; conversely, the increase of VMA reduced the workability (**Figure 20(b)**) (Khayat *et al*., 1996).

The effect of a retarder agent on the slump flow of underwater concrete made with w/cm = 0.41, 100 ml/100 kg of CM of retarder agent, and 0.20% of VMA is shown in **Figure 21** (Khayat *et al*., 1996). For any given dosage of superplasticiciser, the concrete mixes containing retarder agent exhibited better workability (higher slump flow). To secure any given slump flow, the demand of SP is higher for concrete mix made without retarder agent.

For a given type of aggregate used for SCC, the slump flow is affected by the water content, the volume of coarse aggregate, and the dosage of SP (Sonebi *et al*., 2007). It has been reported that the dosage of water had the greatest effect on the slump flow and the increase of dosages of water and superplasticiser led to an increase of slump flow (**Figure 22**). The increase of coarse aggregate volume (CA) (from 220 to 360 litres/m^3) resulted in an increase of slump flow (Sonebi *et al*., 2007). The increase of the dosage of water on V-funnel test led to a reduction of V-funnel flow time and the increase of the volume of coarse aggregate indicated that V-funnel time increased (**Figure 23**). The increase of the dosages of water and SP led to an increase in the L-box blocking ratio. However, the effect of increase of the volume of CA on L-box test indicated that L-box blocking ratio reduced significantly (**Figure 23**). This is due to the increase of risk of blockage with collision of coarse aggregate particles behind reinforced bars of L-box when the volume of CA is high (Sonebi *et al*., 2007).

The loss of workability in time is affected by hydration associated with temperature rise, evaporation and absorption of water in the case of dry aggregates. **Figure 24** presents the isocurves of the effect of dosages of water and super-plasticiser on the slump flow at 5 min and 60 min of SCC made with volume of coarse aggregate of 290 litres/m^3 (Sonebi *et al*., 2007). It can be seen that for any given dosages of water and SP, the slump flow decreased after 60 min. For example, for concrete made with dosages of water and SP of 188 kg/m^3 and 3.8 l/m^3, respectively, the

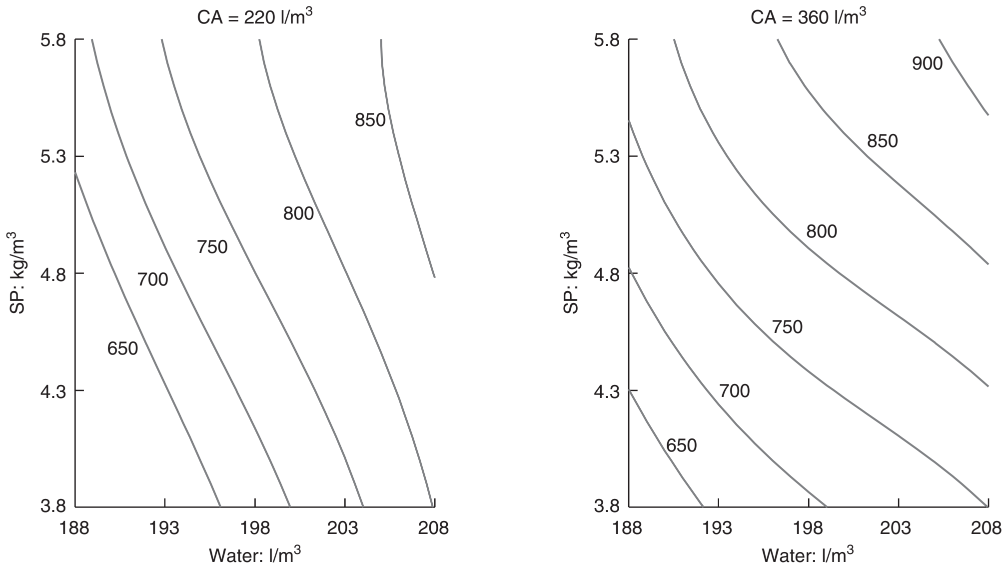


Figure 22 Isocurves of slump flow vs water content, dosage of SP for concrete made with 220 and 360 l/m^3 of coarse aggregate (Sonebi *et al*., 2007)

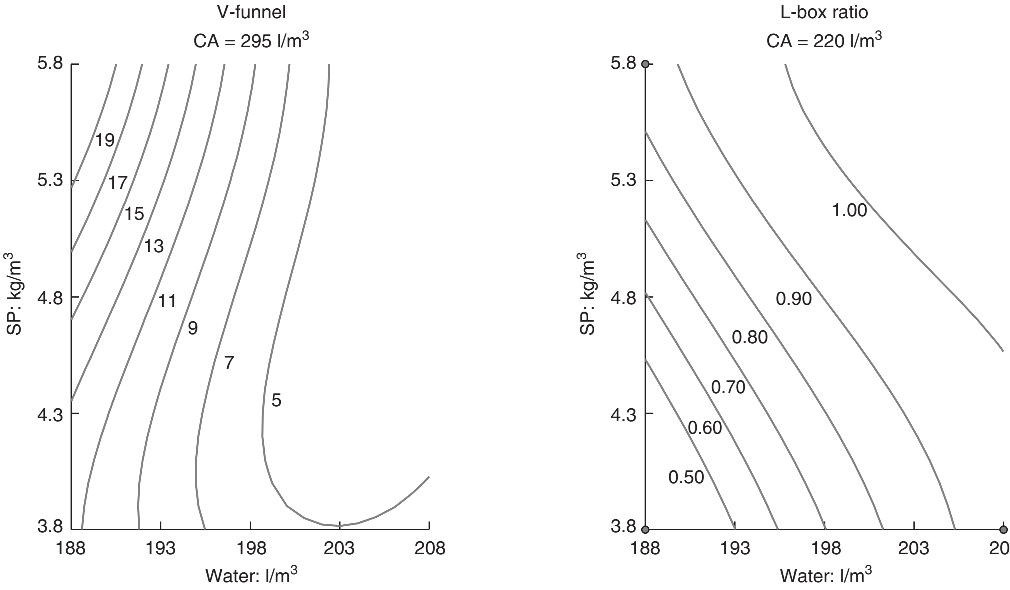


Figure 23 Isocurves for V-funnel time and L-box blocking ratio (Sonebi *et al*., 2007)

slump flow at 60 min was reduced from 600 mm to 350 mm (37% loss of slump flow).

The type and the dosage of supplementary materials influenced significantly the workability of concrete. It was reported that the increase of the dosage of PFA as replacement of cement in SCC led to an improvement of the slump flow (Sonebi *et al*., 2003). For example, **Figure 25** shows the isocurves of slump flow versus the variation of the dosage of PFA and w/cm for fixed content of cement and dosage of SP. It can be see that the increase of dosage of PFA led to an increase of slump flow. PFA improves the contact between the particles of cement by ball-bearing effect and reduces the friction forces, and therefore the workability is improved.

Conclusions

The workability test methods described in this chapter provide a basis for measuring the fresh properties of a wide range of concrete from traditional vibrated concrete

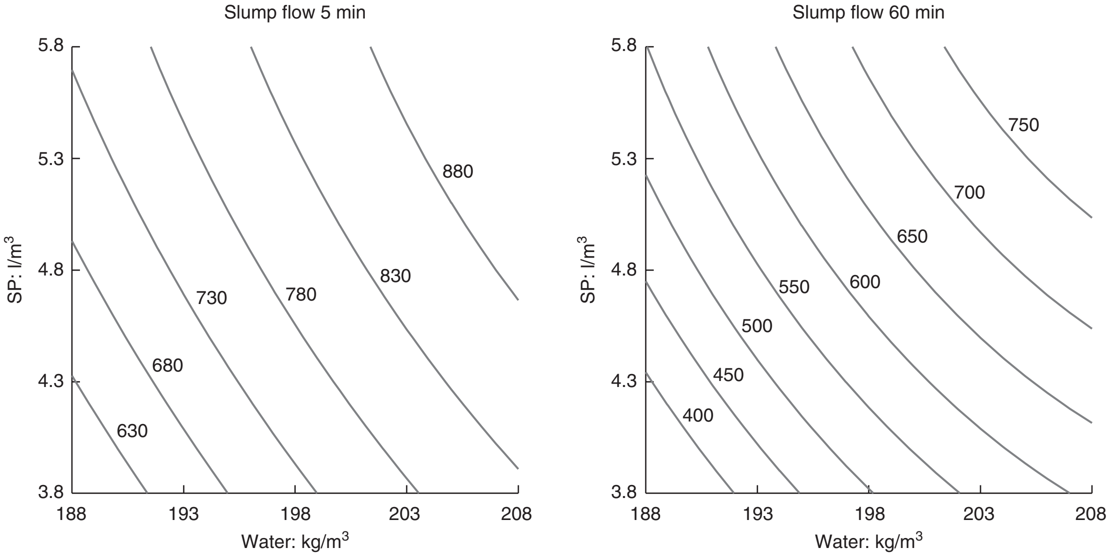


Figure 24 Isocurves of slump flow at 5 min and 60 min vs water and SP – CA = 290 litres/m^3 (Sonebi *et al*., 2007)

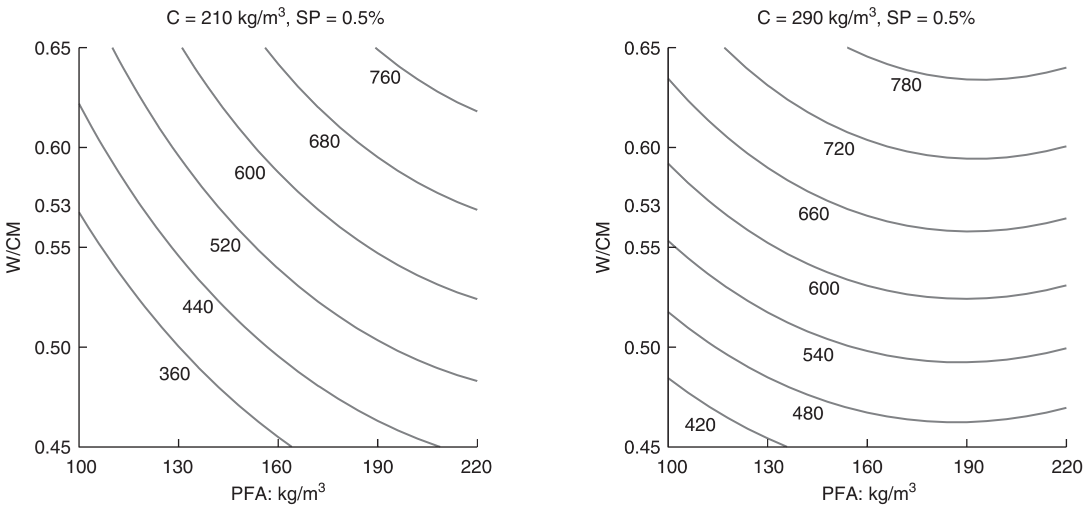


Figure 25 Isocurves of slump flow vs dosage of PFA and W/CM (Sonebi *et al*., 2003)

to highly flowable concrete. The air content, the bleeding and segregation are described with the influence of constituent materials on the fresh properties. The slump test has been used to assess the workability of fresh concrete for almost a century because of its simplicity and the accuracy of the test. The selection of test method to measure workability is left by the specifier to the decision of the contractor on the basis of his skill and experience of the type of construction and the equipment which he selects to use to place, compact and finish the concrete. For a large project where substantial funding is spent to produce and place concrete, the part of funding spent to measure workability is insignificant. The development of new high-performance concrete such as self-compacting concrete will render a need for more accurate field characterisation of concrete rheology which can be accepted and adopted by all diverse parties within the concrete industry.

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Chapter 13

Early-age properties of concrete

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This chapter describes the evolution of early-age properties of concrete. The reactions of hydration and the development of the hydrated structure are discussed first, followed by an understanding of the heat evolution during hydration. The alterations in the hydration process brought about by the use of chemical admixtures are then described. Subsequently, the process of setting and hardening is discussed. This includes the methods to monitor setting and hardening, as well as the development of strength and engineering properties. The last section of the chapter is dedicated to understanding the causes and significance of non-structural cracks, which are an offshoot of the early-age behaviour of concrete.

CONTENTS

Introduction

The early-age properties of concrete are governed by the physical and chemical processes taking place in the cementitious system. The primary factor controlling early-age behaviour is the hydration of cement. It is, therefore, relevant to once again touch upon this complex issue, which has numerous facets. Heat evolution during hydration has significant influence on the performance of concrete at early ages, with respect to the potential for non-structural cracking arising from thermal stresses. The setting of concrete and, subsequently, hardening (development of strength) are also directly related to the processes occurring during hydration.

Modern concrete is rarely produced without the use of admixtures, either chemicals, minerals or both. Early-age properties of concrete are significantly affected by the presence of these components. It is, therefore, imperative to gain an understanding of their influence on concrete properties, particularly with respect to their impact on the hydration process, and on the thermal and shrinkage behaviour at early ages.

Hydration

Concrete is a heterogeneous, two-phase particulate composite, where the cement paste forms the continuous medium and the aggregates are the particulates. The cement paste enables the concrete to flow and compact into the desired form, while the aggregates are primarily responsible for dimensional stability. In the hardened concrete, the hydrated cement paste binds the aggregate together to form a strong and stiff structure.

Hydration of cement involves a series of reactions between the constituent compounds of cement and water. These reactions lead to the setting and hardening of concrete. The kinetics of the hydration process has a significant influence on the early-age properties of concrete because the hydration process is responsible for the building up of the microstructure of the binding cement paste.

Sequence of reactions

The reactions in cement chemistry can be broadly classified into (i) silicate and (ii) aluminate reactions, which lead to the formation of the hydrated cement products (Neville, 1996; Taylor, 1997; Hewlett, 2004).

The silicate bearing compounds of cement, namely, C_3S and C_2S, react with water to produce the calcium silicate hydrate, or C–S–H, gel (the term 'gel' is used to describe the loosely amorphous hydration product; the exact stoichiometry of C–S–H cannot be accurately described, although the CaO/SiO_2 is of the order of 1.5–2). Hexagonal calcium hydroxide (CH) is also formed from the silicate reactions. Almost three times as much CH forms during C_3S hydration as compared to C_2S hydration. This aspect is critical to the long-term durability of the concrete, because CH is highly susceptible to chemical attack.

The aluminates (C_3A and C_4AF), in the absence of gypsum (as in the case of clinker), react rapidly with water to produce calcium aluminate hydrates (C–A–H), which leads to a massive heat output and 'flash' setting of the cement. Once this happens, the concrete is not workable, and the normal strength gain process does not occur. Gypsum is added to cement clinker to control the adverse aluminate reactions. In the normal course of events, the aluminates react with gypsum to produce ettringite (or the AF_t phase). When there is an excess of C_3A present (after the formation of ettringite – this is always the case for ordinary Portland cements), ettringite is transformed into monosulfate (or the AF_m phase). The C_3A still remaining after this reaction goes on to form C–A–H (such as C_4AH_{13}). C_4AF reacts in a manner similar to C_3A (although not as rapidly) and results in the formation of iron-substituted ettringite and monosulfate.

The conversion of ettringite to monosulfate, which always occurs for ordinary Portland cements, has important implications on the resistance of the concrete to sulfate attack. When external sulfates become available to the concrete, the monosulfate can reconvert to ettringite, which is known to be expansive. Therefore, sulfate-resistant cements are designed to have low C_3A contents, which causes all the C_3A to form ettringite in the fresh state itself.

The progress of cement hydration depends, in the initial stages, on the rate of dissolution of the involved phases and, in the later stages, on the rate of nucleation of hydration products. Once a hydrated structure is formed, the rate of hydration is controlled by the rate of diffusion of water through the hydration products to reach the unhydrated material. Thus, a number of external factors, such as temperature and relative humidity, can control the rate of hydration. Apart from these, the cement composition (specifically, the proportions of various phases, and the amount and type of gypsum), fineness, water/cement (w/c) ratio, and presence of chemical and mineral admixtures can affect the rates of reaction.

When mineral admixtures having reactive silica (pozzolans, such as fly ash and silica fume) are used as replacement for cement, an additional reaction between the reactive silica and the CH formed during cement hydration leads to the development of additional CSH. This reaction is beneficial in more ways than one: (i) the CH content is reduced, thus durability against chemical attack is improved, (ii) the additional CSH imparts better strength and fills up the pores in concrete, and (iii) the interfacial transition zone between paste and aggregate gets refined because of the additional CSH. Overall, the pozzolanic reaction is slower than the reaction of cement; however, when fine pozzolans with a high degree of reactive silica are used (such as silica fume and rice husk ash), the reactivity is high.

Development of hydrated structure

Microstructural details of concrete reveal that CSH forms an almost continuous phase. Two types of CSH are commonly observed – the 'inner' CSH just around the hydrating large cement grains, and the 'outer' CSH, which is visibly more porous than the inner CSH (Scrivener, 1989). Unlike CSH, deposits of CH are not uniform across the microstructure of concrete. There is a tendency for CH deposition in pore spaces and around aggregates. Indeed, research has shown that the interfacial zone between paste and aggregate contains a disproportionately high amount of CH. The calcium sulphoaluminates (ettringite and monosulfate) are not easily discernible in the microstructure, as they are generally intermixed with the CSH.

Hydrated cement paste is composed of capillary pores and the hydration product. The pores within the structure of the hydration product are termed 'gel' pores. This hydration product includes C–S–H, CH, AF_t, AF_m, etc. Approximately one-third of the total pore space is comprised of gel pores (Powers and Brownyard, 1948), and the rest are capillary pores. Water within cement paste is held in different forms, namely:

1. Capillary water: present in voids larger than 50 Å. Further classified into: (a) free water, the removal of which does not cause any shrinkage strains, and (b) water held by capillary tension in small pores, which causes shrinkage strains on drying.
2. Adsorbed water: water adsorbed on the surface of hydration products, primarily C–S–H. Water can be physically adsorbed in many layers, the drying of which is responsible for a significant part of the shrinkage. However, the drying of less accessible layers (around fine pores) occurs only at low relative humidity, even as low as 30% (Hansen, 1986).
3. Interlayer water: water held in between layers of C–S–H. The drying of this water leads to considerable shrinkage due to the collapse of the C–S–H structure (2 and 3 are together called 'gel' water).
4. Bound water: this is chemically bound to the hydration product, and can only be removed on ignition. Also called 'non-evaporable' water.

It must be stated here that the water in the pores is more aptly called 'pore solution' because it comprises several ionic species (Andersson *et al.*, 1989). After about a week of hydration, the only ions remaining in the pore solution in significant quantities are Na^+, K^+ and OH^-; these ionic species (along with CH, which is formed as a result of hydration) are responsible for the high pH of the cement paste (12–13). The ionic strength of the pore solution also depends on the presence of chemical and mineral admixtures; as a result, the conductivity of the pore solution is affected (this assumes importance in electrochemical reactions, such as corrosion (Bentur *et al.*, 1997).

Among the solid hydration products, calcium hydroxide occurs in a hexagonal form while ettringite and monosulfate possess distinct crystalline structures. However, CSH has a very loosely crystalline structure, which leads to its being referred to as a 'gel'. The structure of CSH is a subject of much study but most researchers (e.g. Feldman and Sereda, 1970; Jennings, 2000) agree that it possesses a layered structure with polymeric calcium silicate sheets and the different forms of water (described earlier).

Heat evolution during hydration

The overall reaction between cement and water is exothermic, and the heat evolved during the process is termed 'heat of hydration'. The amount of heat evolved depends on the compound composition of the cement, as each compound possesses different heats of hydration

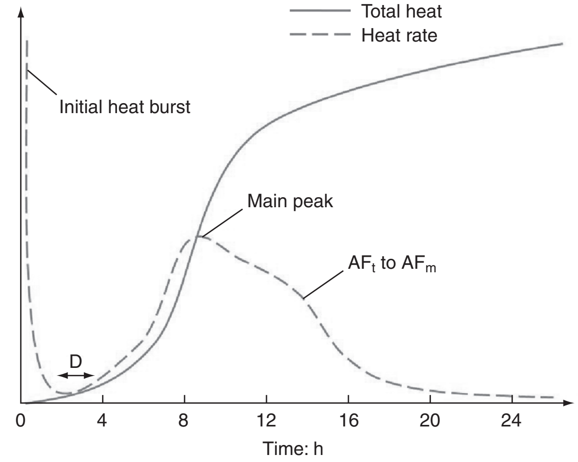


Figure 1 Heat evolution pattern upon hydration for an ordinary Portland cement

(approximate values according to Neville (1996): C_3S – 502 J/g, C_2S – 260 J/g, C_3A – 867 J/g, C_4AF – 419 J/g).

There are two significant aspects of heat evolution during hydration – one is the total heat evolved and the second is the rate of heat evolution. While two cements could potentially end up evolving the same total amount of heat over a long time period, the rates of heat evolution, particularly in the very early stages of hydration, could be the cause of a major distinction in performance. In simple terms, a quicker rate of heat evolution means a faster reaction, quicker setting and early strength. However, on the flip side, higher rates of heat evolution also increase the risk of cracking associated with thermal stresses. These aspects will be explored at a later stage.

The heat evolution pattern for a typical cement (Taylor, 1997) is provided in **Figure 1**. An immediate burst of heat occurs due to the heat of wetting (caused by creation of the solid–liquid interface), and some early C_3S and C_3A hydration. The early hydration products form a barrier over the unhydrated particles, and the reaction slows down, leading to the so-called 'dormant period' ('D' in **Figure 1**). During this period, the rate of heat evolution is small; however, the reaction does not stop. The barrier around the cement grains progressively weakens, and the reaction proceeds at a faster rate, leading to the evolution of the main heat peak – associated with rapid dissolution of C_3S and formation of CSH (also, formation of ettringite from C_3A). As the hydration proceeds, more hydration products are formed; as a result, the water has to move towards the surface of the cement grains by diffusion across the layers of hydration products. This causes a slowdown of the rate of heat evolution (in most calorimetric studies, a third peak corresponding to the conversion of ettringite to monosulfate can also be observed).

As expected, the heat evolution patterns would be different for the different types of cement. Compared to ordinary Portland cement, a low heat cement would generally tend to show a longer dormant period and a smaller main peak (as a result of the reduced C_3S and C_3A), while a rapid hardening cement would show the opposite trend. It must be noted, however, that the total heat evolved over a long period of time may be similar for different cements.

The presence of chemical and mineral admixtures can significantly affect the heat evolution pattern of the cement, particularly with respect to the length of the dormant period and the time of occurrence of the main peak. When set-controlling admixtures, superplasticisers and glycol-based shrinkage-reducing admixtures are used, the dormant period is usually affected. While an accelerator shortens the dormant period, a retarder lengthens it. When mineral admixtures such as fly ash and slag are used, a retardation of the reaction occurs, specifically resulting in an increase of the length of the dormant period. Moreover, in the presence of these admixtures, the rates of heat evolution are also lower. As a result, pozzolanic substitution of cement is highly desired for mass concrete members where temperature rise due to heat of hydration could be severe.

As a result of the heat of hydration, the temperature of the concrete increases. The temperature rise of a particular member depends on the size of the member (if the member is massive, the temperature increase in the interior can be large), as well as the ambient temperature (the higher the ambient temperature, the higher the rise in temperature).

Influence of chemical admixtures

Set-controlling and water-reducing admixtures have a marked influence on the kinetics of cement hydration and the structure of the hydration products. Typically, these chemical admixtures affect cement hydration during the early stages, namely during the processes of dissolution cement compounds and nucleation of hydration products. According to Joisel (1973), only the dissolution is affected by these admixtures. If the hydrating PC is considered as a mixture of cations (Ca) and anions (silicate and aluminate), then the following scenarios can occur:

1. An accelerator should promote the dissolution of both cations and anions. Since several anions are present, the accelerator should promote the dissolution of that anion which has the lowest dissolving rate, i.e. silicate.
2. A retarder impedes the dissolution of Ca ions and aluminates.
3. The presence of monovalent cations – K^+ and Na^+ – reduces the solubility of Ca but promotes the dissolution of silicates and aluminates. At small concentrations, the former effect is predominant, and at high concentrations, the latter effect is predominant.
4. Monovalent anions – Cl^-, NO_3^-, etc. – reduce the solubility of silicates and aluminates, and promote the

dissolution of Ca. At small concentrations, the former effect is predominant, and at high concentrations, the latter effect is predominant.

5 In the case of salts of weak bases and strong acids (e.g. $CaCl_2$) or strong bases and weak acids (e.g. K_2CO_3), at low concentration, the dominant effect is the retardation of Ca and aluminate dissolution; at high concentration, acceleration of the reaction occurs. Calcium chloride (at 1–3% by weight of cement) is the most effective accelerator.

The morphology of the hardened cement paste could be affected by the alteration of the cement hydration reactions by the retarding effect of the superplasticisers as well as the intercalations with the early hydration products (Jolicoeur and Simard, 1998; Roncero *et al*., 2000, 2002). Polysulphonate-based superplasticisers are reported to combine with the interstitial phases of cement ($C_3A + C_4AF$) to form organomineral composites resembling ettringite (Flatt and Houst, 2001; Banfill, 2003). They could also modify the morphology of ettringite produced during hydration, i.e. ettringite could crystallise in thick rod-like crystals rather than conventional needle-shaped crystals (Roncero, 2000; Roncero *et al*., 2000; Prince *et al*., 2002). Similarly, the incorporation of a glycol-based shrinkage-reducing admixture also affects the morphology of the hydrates formed, as well as their growth rates (Roncero *et al*., 2003).

In general, the change in morphology due to the incorporation of superplasticiser improves the rheological properties by yielding smaller hydrate particles and preventing hydration products from bridging neighbouring cement particles. This results in substantial reduction in the yield stress of paste and increase in fluidity. There is also a difference in porosity and pore size distribution of superplasticised concrete compared with normal concrete. Generally, higher numbers of smaller pores are produced in superplasticised mixtures; this is attributed to better silicate polymerisation leading to the refinement of pores and consequent reduction in porosity (Roncero, 2000).

Setting of concrete

The hydration of cement is responsible for the setting and strength gain of concrete. After mixing, placing and consolidation, concrete undergoes a series of changes in its structure, transforming from a viscous semi-solid into a relatively stiff viscoelastic solid.

Setting of concrete is the process of attainment of its shape or form. Two different phenomena occur within setting, and are classified as 'initial' and 'final' set. The initial set happens when the concrete is at a state where it cannot be worked upon any more. In other words, this is the time practically available for concreting operations (including mixing, placing, consolidation and initial finishing). In terms of the hydration time of cement, the initial set generally happens after the dormant period (see **Figure 1**). The hydration process intensifies after the initial set, and the concrete soon attains its final form. The time taken for this is termed the final set. At this stage, the redundant (non-load-bearing) formwork may be removed from the concrete member.

In the laboratory, the initial and final setting times of cement are measured using the Vicat apparatus (ASTM C 91, 2008). The test involves determination of the penetration of a 1 mm diameter needle (with an attached weight) into a cement paste of normal consistency (ASTM C 187, 2008). The initial set is defined as the time when the needle penetrates a depth 35 mm from the top (total depth of mould is 40 mm), while the final set is defined as the time when the needle just makes a mark on the top surface and cannot penetrate the paste. The setting time of concrete is determined by the penetrometer test (ASTM C 403, 2008), which is similar in concept to the Vicat test. Here, the penetration of plungers of different cross-sectional areas into a mortar (prepared by sieving the wet concrete through a 4.75 mm sieve) is assessed, and the load imparted for the penetration is measured. The penetration pressure is then calculated, and initial set is defined as the time when the penetration pressure is $3.4\ N/mm^2$, while final set is measured at a penetration pressure of $27.6\ N/mm^2$.

Sometimes the cement composition can affect the way concrete sets (Mehta and Monteiro, 2006). If the cement contains a very small or negligible quantity of gypsum, then the C_3A present in the cement can react with water rapidly, producing a high amount of heat, and cause a condition known as 'flash set'. Cement thus set cannot be broken up and remixed. It results in poor strength gain. Another condition may also occur during mixing, known as 'false set'. This refers to the gelling of the mixture that occurs while mixing. This gel can be broken up by further agitation. It does not adversely affect the concrete properties. False set may occur when old cement is used, or when the cement contains calcium sulfate in the form of hemihydrate or anhydrite (which gets converted back to gypsum when water is added, and causes a gel to form).

Apart from cement composition, other factors that affect the setting times include (i) temperature and humidity (ambient conditions), (ii) use of additives (mineral admixtures such as fly ash and slag increase the time of setting; chemical admixtures – accelerators and retarders – are specially formulated to alter the setting behaviour), (iii) fineness of cement (the finer the cement, faster the hydration and thus the setting), and (iv) concrete mixture composition, particular, cement and water content.

Strength development

The typical pattern of strength development for an ordinary Portland cement concrete (assuming proper compaction

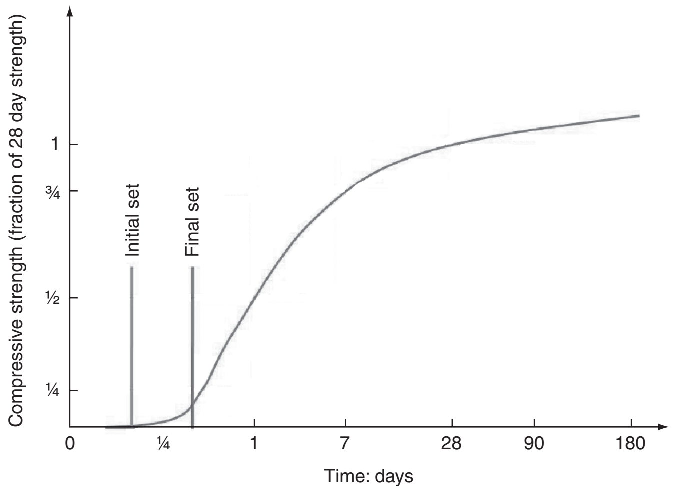


Figure 2 Typical strength development pattern for an ordinary Portland cement concrete

and curing) is shown in **Figure 2**. After the initial set, the concrete possesses some 'green' strength, i.e. it can support its shape; however, the true strength gain process starts only after final set. This 'green' strength is important in specific applications, such as in slipforming, shotcreting, concrete pavers and extrusion. Within 3 days, most modern cements are able to attain close to 50% of the 28-day strength, while by 7 days, the level of strength attained is nearly 75%. After 28 days, the compressive strength continues to increase, albeit at a slower rate. For normal concretes, only up to 10–15% strength increase may be noticed after 28 days. However, when pozzolanic materials, such as fly ash and slag are used, the strength gain process continues beyond 90 days (in the case of high-volume fly ash replacements, beyond 180 days). It must be noted here that in the case of fly ash and slag concretes, the curing process in the field should be continued for a longer duration compared to Portland cement concrete, because of the slowed reactivity (Mehta and Monteiro, 2006).

The increase in strength with age can be attributed to the increased hydration, which causes a gradual filling up of the pores and a consequent reduction in the porosity. Evidence from literature indicates that compressive strength is inversely related to the cube of porosity (Neville, 1996). Either a reduction in total porosity or a pore refinement without much change in the total porosity occurs when supplementary cementing materials are used. The enhancement of strength is also related to a gradual strengthening of the interfacial transition zone between the paste and aggregate. In normal concretes, cracks due to loading tend to concentrate in this region. As a result, the structure and properties of this zone are responsible for the response to loading. It has been extensively reported that the interfacial zone possesses greater porosity than the bulk paste, in addition to microcracks that are formed due to non-load effects such as shrinkage, or due to stress concentrations caused by the irregularities in the coarse aggregate. With increase in hydration, this zone gradually densifies because of the deposition of hydration products. The use of pozzolanic materials leads to further refinement of the interface, both due to the formation of additional CSH as well as due to fine particulate pozzolanic materials filling the pores (Mehta and Monteiro, 2006).

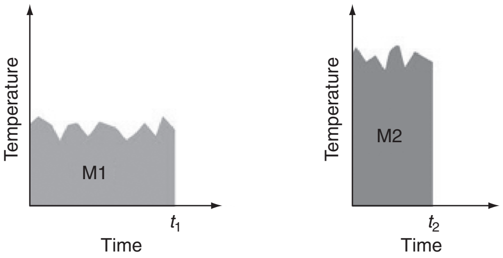


Figure 3 Concept of equal maturity (indicating two cases where the same strength would be reached at different ages if the maturity is the same, i.e. M1 = M2, due to different temperature histories)

The area under the temperature–time curve is called 'maturity', and is often used as an indicator of the strength level of the concrete (Malhotra and Carino, 2004; ASTM C 1074, 2008). The primary assumption is that concretes of equal maturity possess equivalent compressive strengths. The schematic diagram in **Figure 3** illustrates this concept. The maturity index can be calculated using the Nurse Saul function (Nurse, 1949; Saul, 1951), as:

$$M = \Sigma(T - T_0)\Delta t \tag{1}$$

In the above equation, T_0 is the datum temperature – below which the hydration reactions cease to happen. Empirical relations (example given in **Figure 4**) may then be established between the compressive strength and maturity index from laboratory data. These relations can then be used for prediction of strengths for field concrete.

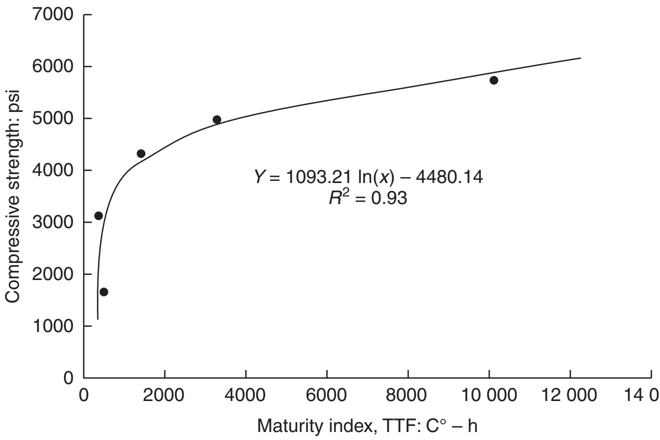


Figure 4 Example of an empirical relation between strength and maturity

Indeed, maturity-based measurements can be employed widely to get estimates of the time setting and for removal of formwork.

The tensile and bond strengths of concrete also evolve in a manner similar to compressive strength. Apart from the water to cement ratio, other factors that affect the development of tensile and bond strengths (Neville, 1996) include aggregate interlocking (angular aggregates provide better interlocking), aggregate texture (rough surface causes better bond with the cement paste, leading to improved tensile performance), and level of compaction of the concrete.

Although the tensile strength of concrete is not important from the load-bearing point of view, it is significant for the cracking resistance of concrete in various situations. Generally, the tensile strength is only one-tenth to one-seventh of the compressive strength. Three methods are commonly used to determine the tensile properties: direct tension, split tension, and flexure. The direct tension method (O'Cleary and Byrne, 1960) is the most appropriate for measuring the tensile response of concrete. However, this experiment is very difficult to perform because of the difficulty in gripping the ends of the specimen for the test. Crushing of concrete at the grips may occur before the specimen actually fails in tension (prismatic or cylindrical specimens may be used – for best effects, the smallest diameter should be used; however, this is restricted by the maximum size of aggregate).

The split tension test (ASTM C 496, 2008) is conducted by loading a cylindrical concrete specimen along its length. This results in the development of tensile stresses along the central diameter in the lateral direction (except for compression very close to the loading points). When these stresses exceed the tensile capacity of the concrete, the specimen simply splits into two halves. The resultant tensile strength is calculated as:

$$\text{Split tensile strength} = 2P/(\pi LD) \tag{2}$$

where P is the peak load, L and D are the length and diameter of cylinder respectively.

The ideal method for determination of tensile strength of concrete is the flexural test, since it matches the real-life situation in RCC flexural members. Here, a simply supported prismatic specimen of concrete is loaded either at the mid-point or at one-third of the span from each end. Failure of the specimen occurs in bending when the tensile stress at the bottom of the beam exceeds the tensile capacity of the concrete, which is called the flexural strength or modulus of rupture. The tensile strength can be calculated as:

$$\text{Maximum tensile stress} = Mc/I \tag{3}$$

where M = moment at peak load, c = distance from neutral axis, and I = moment of inertia of the cross-section.

The advantage of the third-point loading arrangement (ASTM C 78, 2008) is that in the zone between the two loading points, there is a constant bending moment (and the shear is zero). As opposed to the direct tension test where the strain distribution across the cross-section is more or less uniform until cracking, here there is a strain gradient across the depth of the beam – being representative of real situations.

A number of relationships have been suggested between compressive and flexural strength of concrete; most of these are of the form:

$$f_t = k(f_c)^n \tag{4}$$

where n varies between 0.5 and 0.75.

One aspect of primary importance is the zone at the interface of the paste and aggregate – called the interfacial transition zone (ITZ). It has been observed that the porosity of the paste as well as the proportion of calcium hydroxide in this zone is considerably higher than in the bulk paste. Furthermore, the ITZ sees a shift from a component of high stiffness (aggregate) to a component of low stiffness (paste). This mismatch can cause crack formation in the ITZ even at low load levels or as a result of indirect effects such as shrinkage or thermal stresses. Thus, this zone forms the weak link in the concrete, both from a strength and durability point of view.

In a normal-strength concrete, microcracks initiate in the interfacial zone at low load levels (as a result of the load being concentrated around the areas of high stiffness, i.e. the aggregates). These microcracks propagate further into the mortar, localise and lead to the ultimate failure of the concrete. This behaviour is depicted in the schematic diagram shown in **Figure 5** (based on Mehta and Monteiro, 2006). As seen in this figure, when the load is below 30–40%, the cracks are limited to microcracks in the ITZ. When the load level increases to about 50%, the microcracks become larger. At higher loads (~75%), these cracks start propagating into the mortar, and then localise to cause the ultimate failure.

In the case of high-strength concrete, the use of low w/c and mineral admixtures strengthens or densifies the ITZ. As a result, the cracks take the smallest path – through the aggregates. Thus, the aggregate strength in this case has a direct bearing on the concrete strength. Another important consequence of the stronger ITZ and the rupture of the aggregates is that the failure in higher-strength and more mature concrete is more brittle than in normal concretes (Gettu and Shah, 1994; Gettu *et al.*, 1998).

Stiffness and Poisson's ratio

The elastic properties of concrete (modulus of elasticity and Poisson's ratio) also evolve in a manner similar to compressive strength. Due to the non-linear stress–strain response

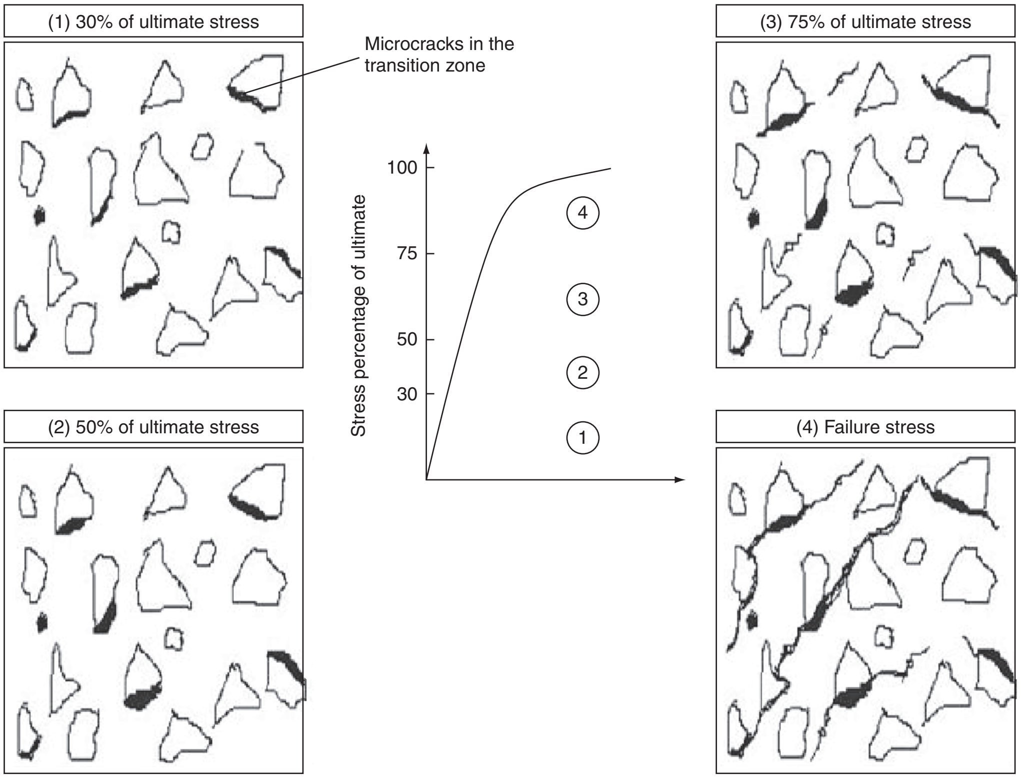


Figure 5 Development of cracking in concrete with progressive loading (adapted from Mehta and Monteiro, 2006)

of concrete, and the absence of a well-defined yield point, various different definitions of modulus of elasticity are used. As shown in **Figure 6**, the initial tangent modulus is the slope of the initial portion of the curve (passing through the origin). A tangent modulus may also be determined at any point on the curve, but this is of little practical importance since it applies to very small changes in load above or below the point where it is taken. The secant modulus is the slope of a line drawn between the origin and any point on the curve. Since this value decreases with an increase in stress, the stress at which the modulus has been determined should be explicitly stated. The maximum stress applied in this static test is usually prefixed (ASTM C 469, 2008 – 40% of ultimate stress; BS 1881 Part 121: 1983 – 33%). To achieve proper seating of the specimen and the deformation gauges, at least two cycles of preloading to the maximum stress are required.

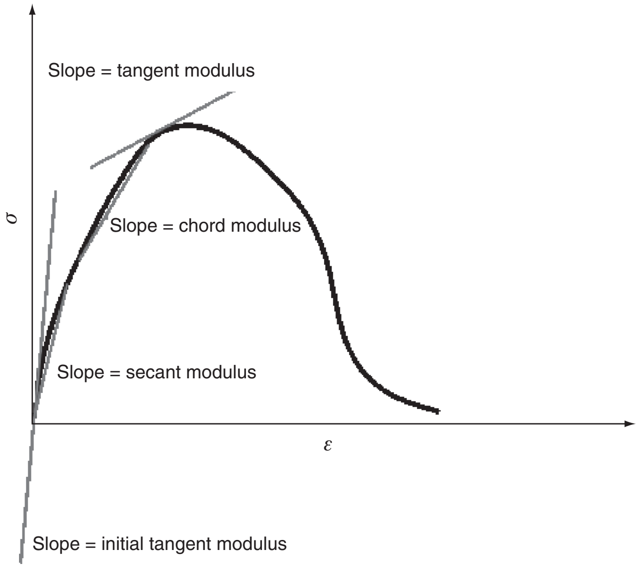


Figure 6 Calculation of different moduli of elasticity from the stress–strain curve

According to ACI 318-95 (2008), the modulus of elasticity of concrete (as determined in the static test) for normal-weight concrete is given by:

$$E = 4.73(f_c')^{0.5} \tag{7}$$

where E is in GPa and f_c' is in MPa.

Sometimes, the dynamic response of concrete is evaluated either by subjecting it to a vibration or by sending ultrasonic pulses through concrete (Malhotra and Carino, 2004). In the first case, E varies directly as the square of the fundamental frequency of vibration. In the pulse velocity test, E is proportional to the square of the velocity multiplied by the density. Since the levels of stress generated by these tests in concrete are extremely low, the E values obtained from this test (also called dynamic moduli) are equal to the initial tangent modulus obtained in the static tests.

The Poisson's ratio for concrete varies between 0.15 and 0.22 (Neville, 1996). It is generally found to be similar in tension and compression. Tests on saturated mortar have shown that Poisson's ratio is higher at higher rates of strain (Harsh *et al.*, 1990). Experimental determination of Poisson's ratio (μ) can be done by calculating the moduli of elasticity (E) and rigidity (G) using dynamic response measurements using the following relationship:

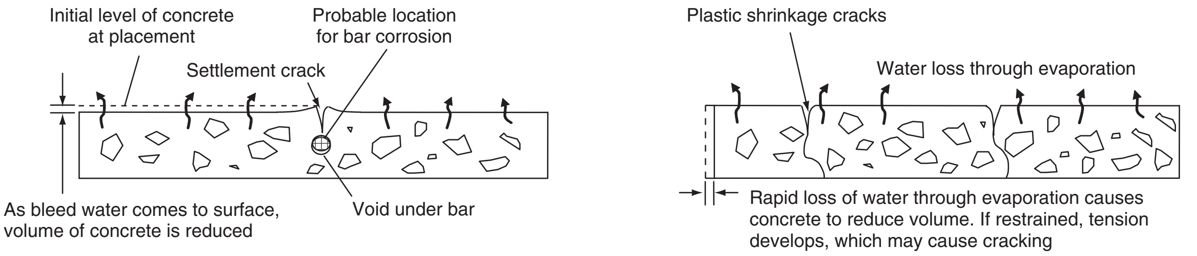
$$\text{Poisson's ratio } \mu = E/2G - 1 \tag{8}$$

Non-structural cracks

Non-structural cracks (i.e. cracks not induced by the applied loads) occur primarily due to volume changes. These volume changes occur as a response to shrinkage, creep or change in temperature. The presence of any restraint, when concrete undergoes volume changes, results in cracking of the concrete. The susceptibility to cracking is particularly high at the early stages of concrete, when there is plenty of water available and the material has not gained sufficient strength to resist the effect of the restraint.

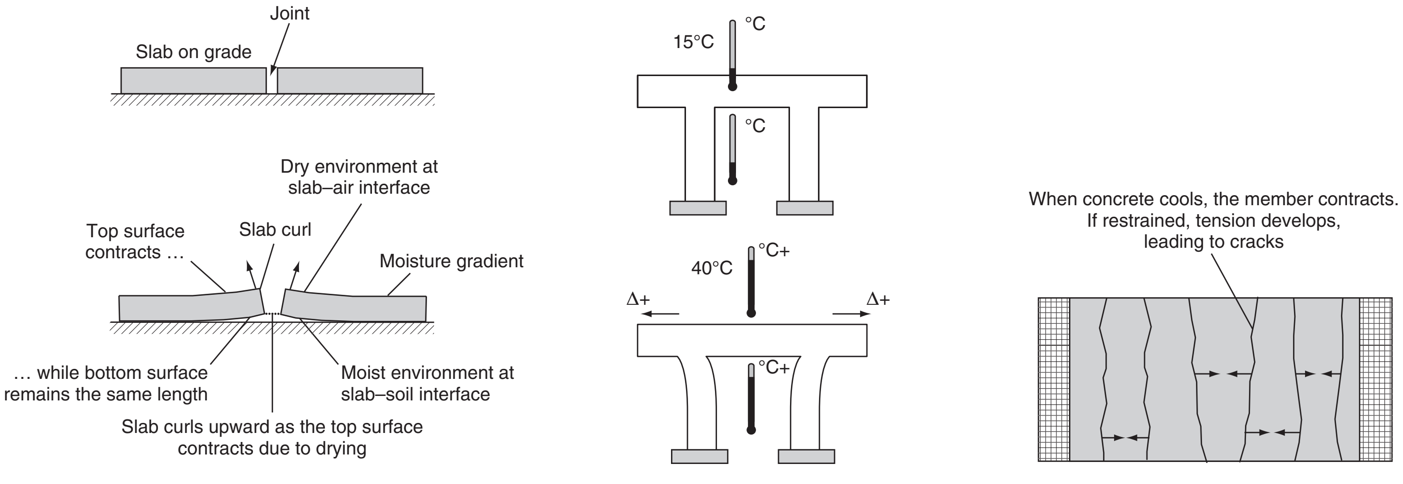
As discussed above in 'Setting of concrete', concrete undergoes a gradual transformation from a viscous semi-solid to a viscoelastic solid. As aging occurs, there is a gradual change in the amount and distribution of water and hydration products. At early ages, the presence of a large quantity of water, accompanied by the lower stiffness of concrete, causes the concrete to undergo large volume changes due to shrinkage or creep. As the strength (and thus the stiffness) continues to increase, along with a decrease in the amount of free water available, the tendency to shrink or creep also reduces.

Cracking can develop in horizontal concrete members (especially those with a large surface area) soon after placement, even before attaining a final set. The primary mechanisms driving such cracking are plastic shrinkage and plastic settlement (Lerch, 1957). Plastic settlement cracks occur over the top reinforcing bars as a result of the subsidence of concrete, as shown in **Figure 7(a)**. Concrete mixtures prone to high bleeding can develop such cracks. On the other hand, when the availability of bleed water on the concrete surface is less, drying of the top surface of concrete occurs (in other words, the rate of evaporation from the surface is higher than the rate at which bleed water is replenished at the surface). The



(a) Plastic settlement cracking due to volume changes

(b) Plastic shrinkage cracking due to drying of top surface after placement



(c) Curling of slabs due to shrinkage

(d) Cracking in a frame due to thermal movement

(e) Cracking in concrete upon cooling from high temperature (plan view)

Figure 7 (a) to (e) Processes of non-structural cracking due to volume changes (based on Emmons, 1993) resultant volumetric contraction of the surface concrete is resisted by the interior concrete, which results in the formation of cracks (**Figure 7(b)**). Concrete mixtures with fine fillers and mineral admixtures (especially silica fume) can experience severe plastic shrinkage cracking (Cohen *et al*., 1990). Exposed surfaces subjected to a combination of high temperature, wind speed and low relative humidity are especially prone to such cracking. Plastic shrinkage cracking can be mitigated to a large extent through the incorporation of fibres and glycol-based shrinkage-reducing admixtures (Mora *et al*., 2000, 2009).

Creation of moisture gradients in a slab can also lead to the development of shrinkage-induced cracks. As depicted in the schematic diagram in **Figure 7(c)**, when the slab is subjected to moisture gradients as a result of a dry surface and a wet soil, there can be an upward curl of the contracting surface, which can cause cracks at the bottom of the slab (Emmons, 1993).

Differential heating of a member could also be a cause for cracking. **Figure 7(d)** shows an example of a concrete frame that experiences differential heating of the top and bottom. The resultant expansion of the top creates cracking in the vertical members.

Another major reason for non-structural cracks is thermal effects. Concretes (especially of high strength and low w/c ratio) that possess high cement contents can develop high heat of hydration (Neville, 1996). As a result, the concrete members, especially ones that are massive, can experience temperature increases of the order of 30–40°C. When such members cool to the ambient temperature, cracks may develop as a consequence of the movement during contraction (see **Figure 7(e)**). In mass concrete, the situation is particularly sensitive, since the temperatures in the interior may far exceed the temperatures in the exterior of the member. Thus, a thermal gradient can get created within a member, leading to a high potential for cracking.

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Chapter 14

Dimensional stability and deformation

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Being a brittle material, concrete is subject to cracking at relatively low tensile strains. Such strains can develop due to a number of different factors, all of which must be considered when designing concrete elements or structures, since cracking can significantly degrade both the mechanical behaviour and durability of concrete components. The main factors contributing to strain development in concrete include drying shrinkage, swelling, stress-induced deformation (nominal elastic and compressive creep), temperature effects and constituent material properties. Any of these factors can lead to volume change and cracking in a concrete element.

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CONTENTS

Introduction

There is an old saying when it comes to concrete: it gets hard and it cracks. Cracking of concrete occurs when the applied tensile strains exceed the allowable strains of the concrete. The easiest way to prevent cracking of concrete and obtain a stable element is to characterise the strain that will develop internally. Equation (1) can be used to calculate the total amount of strain that can be applied to an uncracked concrete element (Hansen and Young, 1991).

$$\Delta\varepsilon_d = \varepsilon_s + \varepsilon_i + \varepsilon_c + \varepsilon_t \qquad (1)$$

where

ε_d = deformation strain
ε_s = drying shrinkage strain since t_0
ε_i = initial elastic strain at time of application of load (t_0)
ε_c = total time-dependent deformation strain due to creep
ε_t = total deformation due to thermal strain conditions.

In reality, all of these different types of strains can be applied to a concrete element simultaneously and cause potential deformations. To understand what forces will affect a specific element, in a specific environment or under a certain set of conditions, each of these strains must be considered separately. Each of the different types of strain will be discussed individually in order to illustrate how each is developed and how each can be characterised.

Drying shrinkage

Water in a cement matrix can be divided into three different types: chemically bound water, physically absorbed water and free water. Water which has reacted with the Portland cement has become an integral part of the solid cement paste matrix. Water occupying the gel pores, and soaking the gel particles is called physically absorbed water. The remainder of the water occupying the capillary pores is the free water. The summation of the physically absorbed water and free water is referred to as evaporable water. Evaporable water can be removed from the cementitious paste by drying the sample at 105°C. The amount of evaporable water in a cementitious paste matrix depends on the water/cement ratio, the degree of hydration, and on whether the cement paste has been exposed to additional water from the environment (Hansen, 1987).

Drying shrinkage is the time-dependent reduction in volume due to the loss of water when the element is at a constant relative humidity and temperature. Drying shrinkage strain in concrete is measured on a load-free specimen. Drying shrinkage does not include length changes due to temperature, loading, or chemical reactions, but depends solely on the humidity of the surrounding environment and loss of water over time. The ultimate drying shrinkage of a specimen is independent of its size and shape, but more related to the drying shrinkage per unit of weight loss. The development of a drying shrinkage history for a specimen can be predicted with good accuracy if the shrinkage of laboratory samples and the element's volume to surface area ratio is known (Almudaiheem and Hansen, 1987).

Measuring drying shrinkage strains over time in cement paste or concrete is complicated for several reasons. Early-age cementitious-based materials dry non-uniformly, which results in a shrinkage gradient throughout a specimen during drying. This shrinkage gradient leads to a differential shrinkage pattern which results in the formation of inhomogeneous stresses. The outer skin of the concrete specimen experiences tensile stresses while the specimen core experiences compressive stresses due to the development of moisture gradients. To avoid cracking, tensile stresses created by drying shrinkage must remain lower than the surface tensile strength of the specimen (Bisschop, 2002). Standardised laboratory tests often used to determine the drying shrinkage of concrete, wherein concrete specimens are exposed to constant temperature and humidity in order to reduce some of the variables, are as follows:

- ASTM C 159
- AASHTO M210
- COE CRD-C 25-94
- BSI BS 1881-5
- JIS A 1129-1.

Swelling

The apparent volume of a concrete element is that which is bounded by its external surfaces, regardless of its internal structure or porosity. The solid volume of a concrete element is that fraction of the apparent volume which is occupied by solid matter. Early-age concrete possesses the potential for a decrease in the solid volume, accompanied by an increase in its apparent volume. This phenomenon occurs when concrete or cementitious paste is permanently submerged in water, which penetrates into the capillary pores to create the swelling effect (the opposite of drying shrinkage). However, this expansion is small and rapidly stabilises. Strains associated with drying shrinkage are typically much higher than those associated with swelling. A typical value of strain induced by swelling in this manner is 150×10^{-6}, although this value is much higher in cementitious pastes (Neville, 2003). Two methods typically used to minimise swelling are restricting the ingress of water into the concrete by creating a dense matrix, and creating an effectively distributed air void system.

Stress-induced deformation

When concrete is subjected to a sustained stress it experiences an immediate initial deformation (nominal elastic) followed by a gradual increase in deformation over time (creep). Drying shrinkage behaviour can appear very similar to that observed under creep conditions. Most of the variables that affect drying shrinkage also affect the creep of concrete. This section provides an overview of the nominal elastic and creep related strains experienced by concrete.

Nominal elastic strain

The initial strain induced at the moment of loading is short term in nature and known as the nominal elastic strain. This strain is dependent upon the time needed to apply the load and the duration needed to reach the initial strain reading. This initial component of stress-induced deformation can be determined using standard test procedures for the experimental determination of static elastic modulus (CEB, 1993; RILEM 107-CSP, 1998; Bazant and Baweja, 2000; Carreira and Burg, 2000).

A stress–strain diagram can be used to describe the instantaneous axial deformation of the load being applied to the concrete specimen. Internal cracking will develop initially between the cement paste and the aggregate but eventually progressive cracking will develop in the cementitious paste matrix. The characteristics of progressive internal cracking will vary from one mix to the next due to the aggregate type, supplementary cementitious materials, and air void connectivity. Progressive internal cracking will influence the shape of the stress–strain diagram. As load-induced internal cracking increases, the concrete will eventually reach a point where the stress–strain relationship becomes non-linear. This deviation from linearity portion is typically deemed to occur at approximately 30% of the concrete's ultimate load (Popovics, 1979). Any load applied to the specimen below this threshold will result in the concrete returning to its original shape when unloaded. The following standardised laboratory tests are used to determine the stress–strain relationship of concrete where the initial strain corresponds to a load duration of 0.01 day (approximately 15 min) (Carreira and Burg, 2000):

- ASTM C 469
- COE CRD-C 19-94
- BSI BS 1881-5/121
- ISO 6784
- JIS A 1149.

One of the factors that could influence initial strain is the different material types that make up the concrete matrix. A list of some of these material types and their affects on initial strain are listed in the Section on Polymers. Acceptable values for initial strain will depend on exposure conditions and how resistant the concrete is to cracking.

Compressive creep

Creep of concrete is defined as a time-dependent deformation due to a sustained load (Neville, 2003). The creep of concrete consists of three basic components that make up the total amount of deformation that is measured: basic creep, drying creep and non-recoverable creep. Long slender concrete elements are at a higher risk of suffering the effects of creep than other elements. Prestressed concrete beams used for bridge/building construction exhibiting large amounts of creep and the associated deformation must be taken into consideration when designing and placing the elements.

The hydration of Portland cement in concrete produces a jelly-like gel around each particle. This gel eventually reaches a crystalline state over time, primarily composed of calcium silica hydrates (C–S–H). The amount of time needed to fully hydrate a Portland cement grain is dependent upon particle size, amount of available water and the proportion of minor mineral phases. Until the cement has fully hydrated, which typically takes years (if not decades), this gel is compressible under loads applied to the element. Concrete that is cured under ambient air conditions shows that about 25% of the total creep will take place in the first 14 days, about 50% during the first three months and about 75% over the first year (Large, 1957). Concretes that are moist cured will exhibit significantly less creep than air-cured samples.

There are several factors that can reduce the amount of creep deformation that a concrete mixture will undergo such as increasing the aggregate content, increasing the available moisture in the surrounding air (particularly during the curing process), reducing the temperature of the element, and increasing the theoretical thickness of the element. Decreasing the water-to-cementitious materials ratio can reduce the amount of creep to the point where there is not enough water to fully hydrate all of the cement particles (Lopez *et al.*, 2007). To minimise the effect of creep, the best measure is to allow the element to be fully cured for as long as possible; the higher the degree of hydration, the less creep deformation that will occur (Persson, 1997).

The time-dependent increase in strain under constant loading during which moisture changes are prevented is termed basic creep. Basic creep is independent of specimen size and shape since the conditions that are established to prevent the movement of moisture through the element are not affected by size or shape. When the humidity conditions allow dehydration of the concrete element to take place, the resultant effect is called drying creep. The magnitude of drying creep is dependent on specimen size and shape because absorption and permeability characteristics dictate the rate of drying creep (ACI Committee 209, 2005). Non-recoverable creep appears when a constant load is removed from a concrete specimen. At this time, there will be a certain amount of elastic strain recovered, as well as the basic creep strain, which will reverse some of the creep-induced deformation. This difference between the loaded and unloaded size of the element, after removal of the sustained load, is the non-recoverable creep strain. In the laboratory, ASTM C 512 is often used to determine the creep of concrete. Since the critical issue associated with creep of concrete in structures is the stresses induced in adjacent elements, the maximum allowable creep for a given concrete is more a function of stress and deformation tolerances in the structure's design.

Temperature effects

Concrete is similar to most elastic materials in that it will expand and contract with changes in temperature. However, unlike many other materials, concrete is a composite material made of varied constituents exhibiting different thermal properties which depend on age, moisture content and porosity (Mindess *et al.*, 2003). The study of temperature effects on concrete has traditionally focused primarily on mature concrete. However, recent trends in the construction industry have enabled concrete practitioners to construct relatively large structures and structural elements. Appropriate design provisions must be made for the placement and early-age control of temperature in massive concrete structures. As a result, the thermal properties of concrete have become important factors used in the design of concrete structures.

Thermal expansion/contraction

Thermal expansion is of considerable importance for concretes subjected to vast temperature differentials throughout their lifecycles, such as that used in bridge superstructures, pavements, building facades, and early-age mass concrete elements. The coefficient of thermal expansion of concrete is typically determined as the change in length with respect to a known change in temperature, as illustrated in Equation (2) (AASHTO, 2007).

$$\alpha_{\mathrm{T}} = \frac{\left(\dfrac{\Delta L}{L_{\mathrm{O}}}\right)}{\Delta T} \tag{2}$$

where

ΔL = change in concrete specimen length (m)
L_{O} = original length of concrete specimen (m)
ΔT = change in temperature (°C).

The standard test methods that have been developed to measure the coefficient of thermal expansion of concrete and mortar are as follows; AASHTO TP-60 (AASHTO, 2007), BS EN 1770, BS 6319, ASTM C531, CRD-C 39 (1981). In circumstances where materials or structural engineers decide that the value of the coefficient of thermal expansion is necessary for design purposes, the appropriate standardised test should be performed on a specimen from a representative concrete mixture to obtain this design value. As coefficient of thermal expansion testing has become standardised, physical testing for quality assurance has become more common in specification documents.

Like strength and other physical properties, the coefficient of thermal expansion is dependent upon mixture design. The proportions of hydrated cement paste, coarse aggregate and fine aggregate comprise the vast majority of concrete's volume and thus exert the greatest influences on the expansive properties of the resultant concrete. **Table 1** provides typical values for the coefficient of thermal expansion of concrete with respect to coarse aggregate mineralogy (Gajda, 2007).

It should be noted that while **Table 1** provides typical coefficient of thermal expansion values, standardised testing should be performed in order to obtain the actual value for a given concrete. The typical value for coefficient

Aggregate type	$\alpha_T \times 10^{-6}$: (m/m)/°C
Quartz	11.9
Sandstone	11.7
Granite	9.5
Basalt	8.6
Limestone	6.8

Table 1 Typical concrete thermal expansion coefficients by aggregate type

of thermal expansion of hydrated cement paste is known to be 18 – 20 × 10^{-6} (m/m)/°C or as much as three times the coefficient of thermal expansion of concrete. Designers should thus be aware that elements cast from a concrete mixture with a high volume of paste will tend to have a higher coefficient of thermal expansion. There have been case studies involving concrete claddings in high-rise buildings that have lost functionality due to excessive thermal expansion resulting from a high paste content in the concrete (Neville, 2003). Engineers and designers should be diligent in considering such issues when designing structures or structural components.

Volume changes due to temperature effects are a particularly important design parameter for concrete pavements. Pavements are essentially a longitudinally continuous structure and, when concrete is used for construction, thermal expansion properties become essential in design considerations. Thermal properties have been linked to the cracking, significant movement, and eventual failure of pavements (Sargious, 1978; Kim and Won, 2004).

Mass concrete

Strains induced in mass concrete elements develop as a result of the thermal gradients that arise within massive concrete elements at early ages. The hydration of Portland cement paste is an exothermic reaction resulting in the production of large quantities of heat. In smaller structures, this heat production does not adversely affect the concrete, as the heat dissipates relatively quickly. However, in mass concrete elements, heat production can result in a significant temperature rise which often results in high internal temperatures that drop quickly near the outer edge due to the dissipation of heat from the exposed surface. As a result, thermal gradients and temperature differentials are produced and internal stresses and strains develop.

Definition of mass concrete

Mass concrete has historically been associated with large gravity structures such as hydraulic dams and similar structures (Bamforth, 1984). However, concrete practitioners and materials engineers have more recently reported that problems normally associated with such massive structures are not strictly limited to extremely large volume placements. The American Concrete Institute defines mass concrete as 'any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change, to minimise cracking' (ACI Committee 207, 2005). The definition of mass concrete has evolved as the understanding of concrete material science has evolved. In the 1980s Bamforth (1984) suggested that mass concrete be defined as any element with a minimum dimension of 50 cm. Many of the construction specifications prescribed by state regulating agencies throughout the US define mass concrete as any structure with a minimum dimension of 122–138 cm. Thus, the concrete industry has become more conservative regarding the consideration of thermal effects in placing concrete in what has historically been considered relatively small structures.

Cracking of mass concrete

The hydration reaction of Portland cement is exothermic in nature, therefore all hydrating concrete experiences some degree of temperature rise at early ages. Since concrete is a relatively good insulating material, the majority of the heat produced in a massive concrete element tends to accumulate within the concrete, resulting in large temperature rises. However, the near surface of mass concrete tends to be cooler due to ambient temperature and heat dissipation (Bamforth, 1984). As discussed, an increase or decrease in the overall temperature of concrete will result in an expansion or contraction of the concrete proportional to its coefficient of thermal expansion. However, the inner portion of the concrete will reach greater temperatures than the outer portion of the concrete, resulting in a temperature differential. Therefore, in mass concrete the inner, hotter portion of the concrete tends to expand at a greater rate than the outer cooler portion of the concrete, as depicted in **Figure 1**.

Since, concrete is elastic and brittle in nature, it will tend to develop stresses as a result of the differential expansion

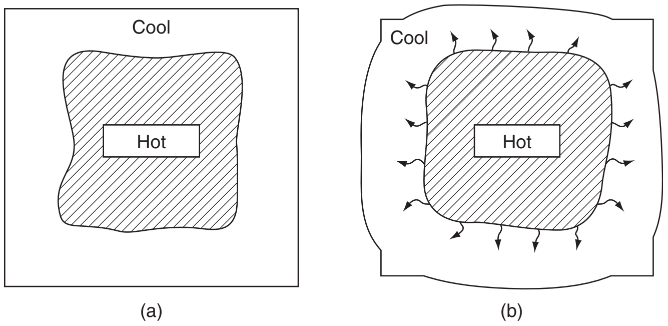

Figure 1 Typical temperature development in mass concrete

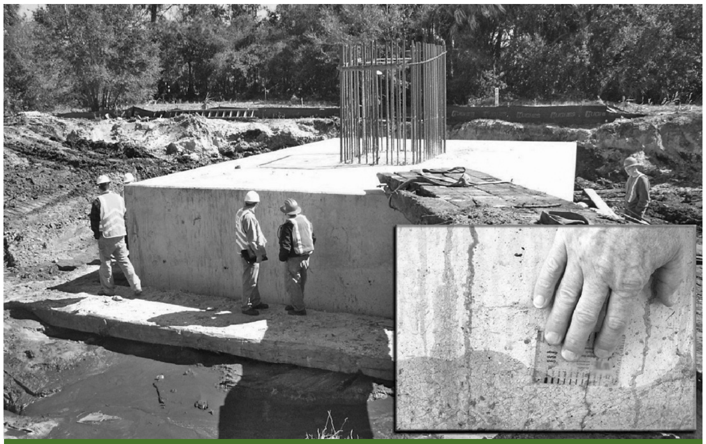

Figure 2 Cracked concrete foundation and close-up view of thermal cracking

rather than deform to relieve stress as shown in **Figure 1(b)**. When the temperature differential between the outer strata and inner core becomes too great, the difference in expansion will cause tensile stresses which can result in cracking. In the event proper controls are not applied to mass concrete at early ages, thermal cracks can occur, resulting in substantial repair, rehabilitation, or replacement costs for concrete elements. The pile cap/footing shown in **Figure 2** is approximately 10 m long × 6 m wide × 2 m high, and experienced thermal cracking throughout due to large temperature differences. The repair and contingency plan was substantial due to the fact that over 30 cracks propagated through the entire height of the footing.

Heat of hydration and temperature rise of mass concrete

The temperature rise of mass concrete is dependent upon the concrete mixture ingredients, especially the amount and type of cementitious materials. Heat generation characteristics of the hydrated cement paste are dependent upon the exothermic hydration of Portland cement, which is controlled by cement chemistry, and the incorporation of supplementary cementitious materials. The temperature rise in concrete is dependent on the heat of hydration of the hydrated cement paste and the thermal characteristics of the aggregates.

There are several test methods for determining the heat of hydration of cementitious materials, with calorimetry being the most direct method for measuring heat evolution. The development of applicable calorimeters was necessary to measure and predict the factors controlling the rate of temperature rise in concrete, temperature gradients, and temperature differentials (Morabito, 1998). There are four types of calorimeters used for the measurement of heat of hydration of concrete: adiabatic (no heat loss or gain), semi-adiabatic (some heat loss), solution calorimeters (some heat loss) and isothermal (constant temperature) (Poole, 2007). Adiabatic and semi-adiabatic calorimeters are typically used for concrete, whereas solution and isothermal calorimeters are typically used for cement. There have been standard test methods created for the measurement of heat of hydration of each, most of which use either semi-adiabatic calorimeters or isothermal calorimeters. A list of standard test methods which use solution calorimeters is as follows:

- ASTM C 186
- BS 4550
- CEN EN 196-8
- JIS R 5203.

A list of standard test methods which specify semi-adiabatic calorimeters includes:

- BS EN 196-9
- CEN EN 196-9
- DIN EN 196-0.

The US Army Corps of Engineers CRD C38-73 is a standardised test method which requires that additional heat be added to the specimen to create an adiabatic, zero heat loss environment. This type of set-up can be somewhat difficult to implement because it is possible to add sufficient additional heat to artificially create a higher heat of hydration than that actually produced by the cement.

The measurement of heat of hydration of cement has been used to create predictive relationships between cementitious properties, specimen geometry, and temperature rise (Alexander, 1996). As the age of computing has evolved, the ability to create finite-difference and finite-element models has made it more practical for designers and end-users of mass concrete. As the accuracy of these computer models improves, and the models themselves are verified, it is anticipated that thermal cracking of concrete structures can be more effectively avoided in the future.

Factors influencing dimensional stability and deformation

There is a myriad of different materials added to concrete that can have a drastic effect on drying shrinkage, elastic strain, creep and cracking potential. The most common component materials are the aggregates, cementitious materials and admixtures. Changing any of these materials can have drastic effects on the properties of the concrete and its long-term service life. **Table 2** presents a summary of the potential influence concrete components can have on dimensional stability.

Aggregates, in particular, can have a profound and important influence on the hardened properties of concrete, primarily due to the proportion of the concrete's volume for

Material type	Deformation characteristics	Degree of influence	Methods of control
Aggregate	Elastic strain/ creep, thermal expansion	High	Quality of aggregate
Admixture	Drying shrinkage, swelling	Moderate	Use properly depending on application
Supplementary cementitious materials	Drying shrinkage, swelling, creep, heat of hydration	Moderate to high	Use properly depending on application

Table 2 Factors influencing dimensional stability

which they account. The deformation properties of concrete are affected by aggregates through a combination of effects including water demand, aggregate stiffness, volume concentration and paste–aggregate interaction. Results from varying aggregate types have shown that concrete modulus of elasticity, shrinkage and creep can each vary by as much as 100%, depending on the type of aggregate selected. A correlation starts to emerge to indicate that the aggregate elastic modulus can represent the overall concrete stiffness (Alexander, 1996).

Admixtures may or may not have a drastic influence on the hardened properties of concrete. Most admixtures are designed to improve the properties of the hardened concrete if used properly. One newly developed class of admixtures that will help reduce drying shrinkage and reduce the potential of cracking is shrinkage reducing admixtures (SRAs). SRAs are designed to reduce the potential for short-term and long-term cracking in concrete (Folliard and Berke, 1997). Air-entraining admixtures typically do not affect the dimensional stability properties of concrete as long as the air content is not increased so much that desiccation occurs (Tazawa and Miyazawa, 1995; Persson, 1997).

Most modern high-performance concretes will contain a supplementary cementitious material such as fly ash, ground granulated blastfurnace slag, silica fume, metakaolin or rice hull ash. Each material is different and can reduce the potential for cracking at different ages. Fly ash, when used in modest amounts (15–25%) can produce a modest decrease in creep at later ages, but does not show much reduction in shrinkage at any age. Ground granulated blastfurnace slag appears to reduce creep in concrete when higher dosages are added to the mix, but can provide higher shrinkage at early ages. Silica fume has a tendency to reduce the drying shrinkage and creep of concrete at later ages, but can lead to early-age shrinkage (Khatri and Sirivivatnanon, 1995). High-performance concrete can exhibit high levels of drying creep even when exposed to water, due to its low permeability (Persson, 1997). Most early-age cracking caused by the addition of supplementary cementitious materials can be reduced by providing sufficient water at early ages for proper curing of the concrete.

Summary

List of deformation characteristics:

- drying shrinkage
- swelling
- stress-induced deformation
- nominal elastic strain
- compressive creep
- thermal expansion/contraction
- cracking of mass concrete
- heat of hydration and temperature rise of mass concrete.

There is an abundance of factors that can influence the behaviour of concrete that should be considered in practical concrete design. This chapter discusses several parameters which are important for the incorporation of dimensional stability and deformation characteristics of concrete and their consideration for application and practical usage.

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Chapter 15

Properties of hardened concrete

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This chapter deals with properties of hardened concrete, mainly dependent on composition and materials used, but also on production processes. A historical note is then presented from its simplest forms over 7000 years, its intense development in the nineteenth century with the discovery of Portland cement and the use of reinforced concrete, up to today, with concrete as the second most used material worldwide. Strength is then presented including test description. Failure mechanisms are then described with focus on cracking in compression, creep and shrinkage. The following sections deal with pore structure and transport mechanisms within the material – that is, permeability, diffusion and capillary absorption – making clear the importance of water/cement ratio and curing on concrete quality. Concluding the chapter, the influence of constituent materials – that is, cement, aggregate and admixtures – on hardened concrete properties is addressed. In terms of cement, the influence of alterations in compound composition of clinker is addressed, first referring to sulfate-resisting cement, low-heat cement, rapid-hardening cement and white cement. Second, the influence on concrete properties of additions such as slag and pozzolans – that is, metakaolin, rice husk ash, fly ash and silica fume – is addressed. Finally, as many of these materials are by-products or even waste materials, it is emphasised that their use as cement replacement materials reduces carbon dioxide emissions and enhances durability, therefore leading to greater economy and advantages in terms of environment and sustainable construction.

CONTENTS

Introduction

Concrete is a brittle material which results from an adequately proportioned mixture mainly of cement or binder, aggregates and water. It may also contain admixtures and additions.

Hardened properties of concrete are fundamental. Concrete is different from other structural materials which are delivered ready to use. In fact properties of hardened concrete depend on batching, mixing, handling, placing, compaction and curing as well as composition and characteristics of materials used – binder (cement), coarse and fine aggregates, water, and eventually admixtures and additions. Moreover, the properties of concrete continue to change over time, even after hardening, sometimes even for concrete that is 20 or 30 years old.

Historical note

Concrete is not just a modern material and simple forms of it have been used for thousands of years.

The most ancient type of concrete appears to be that found in the ground floor of a home in Lepenski Vir, on the banks of the Danube, which was dated around 5600 BC. In the pyramids in Egypt a mixture of aggregates with a binder, generally lime or gypsum, was most probably used (Coutinho and Gonçalves, 1994).

The Greeks and Romans used calcined limestone but lime does not harden under water and the Romans used ground lime and volcanic ash or finely ground burnt clay tiles for construction under water. The active silica and alumina in the ash and the tiles combined with the lime produced what became known as pozzolanic cement, from the name of the village of Pozzuoli, near Vesuvius, where the volcanic ash was first found (Neville, 1995). Structures built by the Romans such as aqueducts, bridges, piers and great buildings such as the Pantheon in Rome and the Coliseum (**Figure 1**) were made of masonry and bonded by mortar, and they still exist today. During the Middle Ages and the Renaissance, although the use of cement slowed down, it was still used in ground pavings and foundations. It was in the nineteenth century that the use of concrete developed. In fact John Smeaton found that hydraulic lime could be obtained by burning a mixture of lime and clay, which finally led to the discovery of Portland cement by other scientists, where the temperature had to be high enough for clinkering. In 1824 Joseph Aspdin patented Portland cement.

The name Portland cement, given originally due to the resemblance of the colour and quality of the hardened cement to Portland stone, a limestone from Dorset, UK, has been used since to describe a cement obtained by intimately grinding calcareous and argillaceous or other

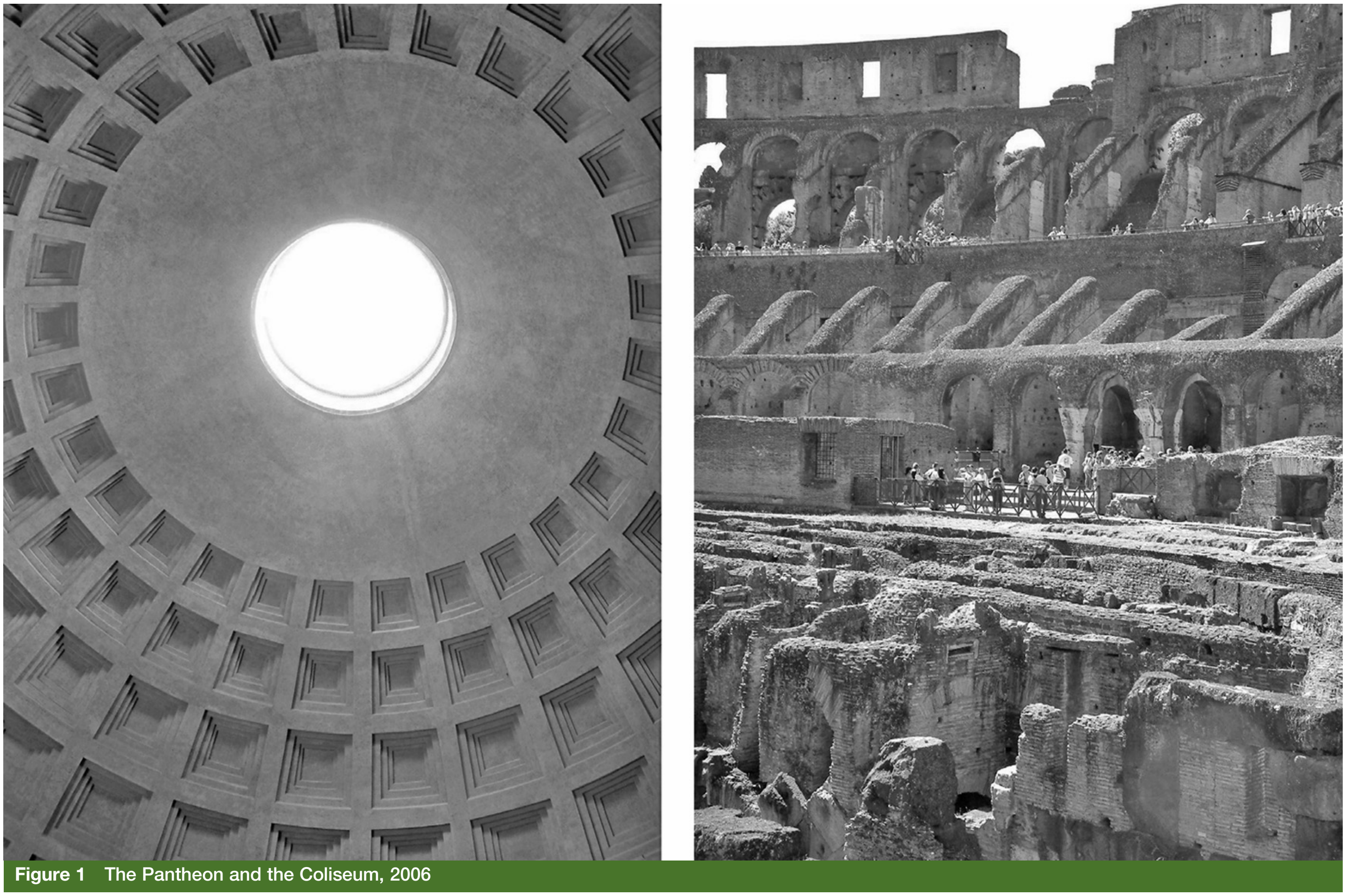


Figure 1 The Pantheon and the Coliseum, 2006

silica-, alumina-, and iron-oxide-bearing materials, burning them at a clinkering temperature and later grinding the resulting clinker with gypsum (Neville, 1995).

Reinforced concrete has been used since 1848, with a boat being built by Lambot which was exhibited at the Universal Exhibition in Paris, 1855.

In the early times of the use of reinforced concrete, strength was generally considered the only important property and concrete strength was relatively low. Elements were produced with large cross-sections, cement content was high, concrete cover was generous and great care was taken in handling, placing and compaction, although compaction techniques were fairly primary. Reinforcement used was of mild steel and therefore concrete hardly showed any cracking. All these factors delayed corrosion of reinforcement and reinforced concrete seemed to be everlasting. In fact, as a consequence of the way reinforced concrete was being produced, durability had not yet become a concern with strength being the only acceptance criterion.

As knowledge and research was developing, especially in terms of mix design, binder properties and other constituents, concrete strength increased steadily (**Figure 2**). Structural design also developed along with knowledge and properties of steel, leading to the reduction of cross-sections used in concrete structures, which gradually became grander. At the same time reinforcement was becoming denser, making it harder for placing and passing of fresh concrete. This often led to overdosage of water and fine aggregate as well as a reduction of concrete cover.

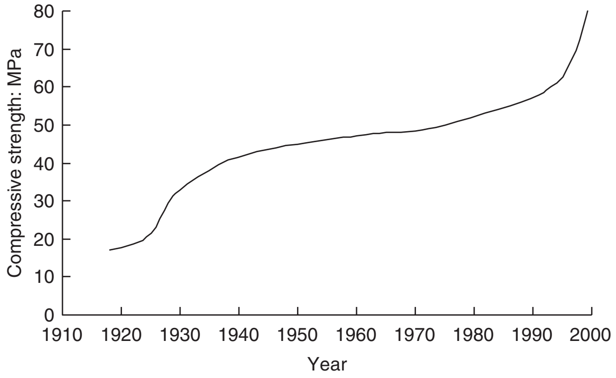


Figure 2 Development of strength of concrete with time (adapted and updated in Sousa-Coutinho, 2005 from Pomeroy, 1987 and Neville, 1995)

As the number and dimensions of concrete structures increased, some engineers and technicians responsible for concrete production felt overconfident and seem to have relaxed on fulfilling requirements and care in execution, especially in more densely reinforced areas. Prestressed concrete was being implemented (since 1928), with much smaller element cross-sections and stronger steel subject to high tension values, making it prone to corrosion. Some cases of admixtures that later proved to have adverse effects were also reported.

All these conditions, aggravated by higher levels of aggressiveness due to pollution, led to reduction in the efficiency of natural protection of steel by concrete, and durability became, as it still is today, an important issue to be considered together with strength.

Strength

Strength of concrete is almost invariably a vital element of structural design and is specified for compliance purposes. It usually gives an overall picture of the quality of concrete because strength is directly related to the structure of the hydrated cement paste (Neville, 1995). Nevertheless, strength may not be fundamental in terms of behaviour of a structure. In fact, in many cases, there are other properties that may be more important such as permeability or resistance to aggressive agents, which are more difficult to measure. For example, a low-permeability, low-porosity concrete is more durable and this also means it has high strength. In structural situations concrete will be subject to one of a variety of types of loading, resulting in different modes of failure. Knowledge of the relevant strength is therefore important: for example, in columns or reinforced concrete beams, compressive strength is required; for cracking of a concrete slab tensile strength is important. Other situations may require torsional strength, fatigue or impact strength or strength under multiaxial loading. Most strength testing involves the use of a few, relatively simple tests, generally not related to a particular structural situation. Procedures enabling data from simple tests to be used in design have been obtained from empirical test programs at an engineering scale on large specimens (Domone, 1998).

Concrete strength (compressive and tensile) is largely determined by the strength of the cement paste which depends on the porosity which, in turn, mainly results from the water/cement (W/C) ratio. Concrete strength also depends on the strength of the paste–aggregate bond and on properties of the aggregate such as strength and concentration in the paste – that is, aggregate content in concrete. Strength of the paste–aggregate bond is a result of the roughness of the aggregate surface and, mainly, of the strength of the paste.

Concrete is mainly used to withstand compression in a structure, but tensile strength is also important to avoid serious cracking and therefore maintaining continuity and also protecting reinforcement against corrosion in the case of reinforced concrete structures.

Therefore, the most common strength tests are the compression test and two tests that enable determining indirectly the tensile strength – the flexural test and the splitting test. Direct determination of tensile strength is not practical, mainly because of difficulties in gripping and aligning the specimens.

Compressive strength

A compressive strength test may be carried out on a concrete cube or on a cylinder, as stated in the European Standard EN 206-1: 2000. The procedure must be in accordance with EN 12390-3: 2001. The specimens must be cast in robust moulds and conform within narrow tolerances to shape, dimensions and planeness as specified in EN 12390-1: 2001. Making and curing specimens for strength tests must follow procedures described in EN 12390-2: 2001. After demoulding when set, specimens are generally cured under water or in a fog room at constant temperature until testing. Specimens are then loaded to failure in a compression testing machine conforming to EN 12390-4: 2001 (**Figure 3**).

When load distribution on the specimen is strictly uniform the failure pattern is a double pyramid, truncated for shorter specimens such as cubes (**Figure 4**). In fact, during the test, stress is far from uniaxial because of lateral restraint forces in the concrete near the platen, caused by the differences between the Poisson's ratios of the two materials being deformed transversally. Therefore, this concrete is in a triaxial stress state, inducing higher failure strength compared with unrestrained concrete. This fact is the main objection to using cubes, however the alternative cylinders are more difficult to handle and prepare. In fact

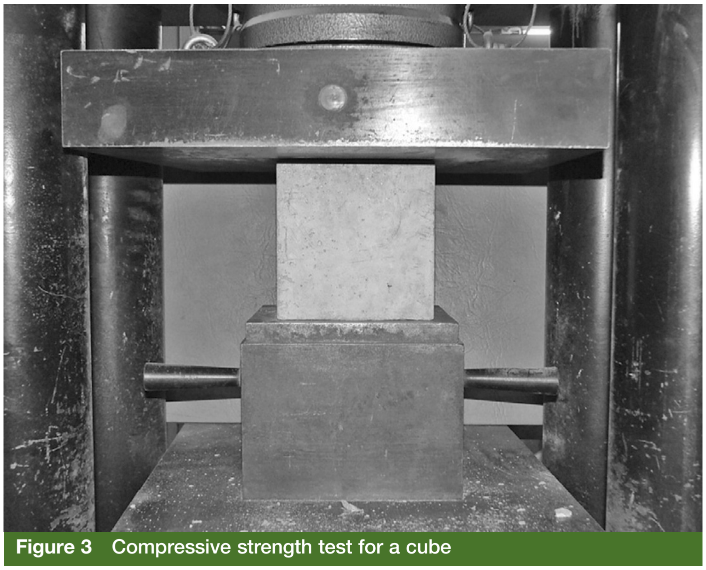


Figure 3 Compressive strength test for a cube

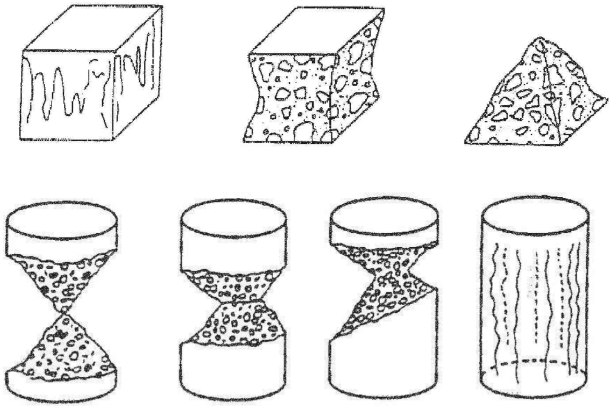


Figure 4 Satisfactory failure of cube and cylinder specimens (adapted from EN 12390-3: 2001)

the top surface must be smoothed or levelled out for testing and this is, of course, more time-consuming. The cylinder strength is generally assumed to be about 80% of the cube strength.

Sometimes it is necessary to determine compressive strength on parts of a structure, and then test-cores in accordance with EN 12504-1: 2000 must be drilled out from specific places, to be tested for compressive strength. In any case the maximum load sustained by the specimen is recorded and the compressive strength of the concrete is calculated.

Tensile strength

Information on concrete tensile strength is of interest because of estimating load under which cracking can develop. Cracking is of utmost importance, not only in terms of structural continuity but also in terms of durability.

Testing concrete directly for uniaxial tensile strength presents some practical difficulties compared with testing other materials such as steel. In fact it is difficult to align and grip relatively large cross-section concrete specimens. Complex gripping systems have been developed, and more recently, simpler methods based on bonding concrete to steel with special bonding products. In any case direct tension is usually only used in research and for regular situations; indirect tensile testing is carried out, either with a flexural test or with a splitting test.

Flexural strength

A flexural strength test is usually carried out on a concrete prismatic beam. In Europe the procedure must be in accordance with EN 12390-5: 2000. The specimens must be produced and cured in accordance with standard procedures or sawn specimens can be used. Usually a two-point loading test (**Figure 5**) is used and the theoretical maximum tensile stress (modulus of rupture) is reached in the bottom fibre of the beam, causing a crack that propagates upwards. Analysis based on simple beam-bending theory and linear elastic stress distribution up to failure leads to flexural strength, given by equation (1), if fracture occurs within the central one-third of the beam. Flexural strength thus calculated is about three-quarters greater (Neville, 1995) than direct tensile strength because of the inaccurate assumptions considered.

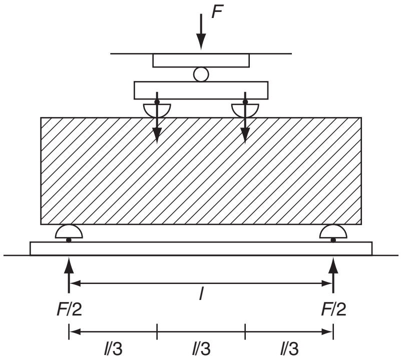


Figure 5 Determination of flexural strength (modulus of rupture)

$$f_{\mathrm{cf}} = \frac{Fl}{d_1 d_2^2} \tag{1}$$

f_{cf} = flexural strength, in MPa
F = maximum load in N
l = distance between the supporting rollers, in mm
d_1 and d_2 = lateral dimensions of the specimen, in mm

Splitting strength

Usually a splitting test is carried out on a cylinder placed on its side and loaded across its vertical diameter, inserting hardboard or plywood strips between the platens and the cylinder so that load is applied over the full length (**Figure 6**). The European standard EN 12390-6: 2001 describes the procedure. Values obtained are also greater than uniaxial tensile strength.

$$f_{\mathrm{ct}} = \frac{2 \times F}{\pi \times L \times d} \tag{2}$$

f_{ct} = tensile splitting strength, in MPa
F = maximum load, in N
L = length of the line of contact of the specimen, in mm
d = diameter of the cross-section, in mm

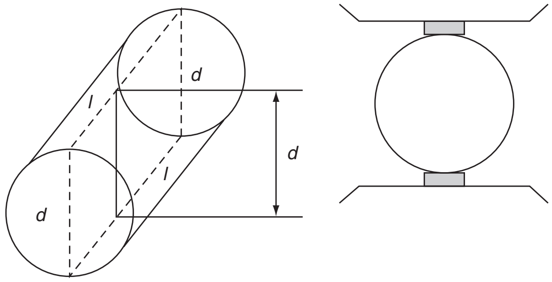


Figure 6 Determination of splitting strength

Failure mechanisms

Failure of concrete is the consequence of cracking which develops from microcracking. Microcracks (usually considered under 0.1 mm) initiate at the interface between coarse aggregate and cement paste, even prior to application of load on concrete. They are probably due to the inevitable differences in mechanical properties between the coarse aggregate and the hydrated cement paste, coupled with shrinkage or thermal movement (Neville, 1995).

Cracking in compression

Concrete may be considered composed of a hydrated cement paste matrix with aggregate particles dispersed in it. The stress–strain behaviour of both the hydrated cement paste and aggregate, when subjected to load individually, presents a substantially linear stress–strain relation (**Figure 7**). However, for the composite material – concrete – it is clearly non-linear. The non-linear behaviour of concrete is mainly due to the interfacial zone between the aggregate and cement paste, a relatively weak zone, where microcracks develop even before loading. This microcracking is due to thermal or drying shrinkage and bleeding.

As load increases, these microcracks remain stable and are considered irrelevant, up to about 30% of the ultimate load and the stress–strain curve is approximately linear (Stage 1). Upon further increase in load, microcracks begin to increase in length, width and number and non-linearity becomes apparent. Cracks propagate slowly, in a stable way and stay confined to the transition zone (Stage 2).

Above 50% of the ultimate strength cracks spread into the mortar (Stage 3) (cement paste and fine aggregate) and non-linearity becomes more evident. It is in this phase that cracking sounds start to be heard.

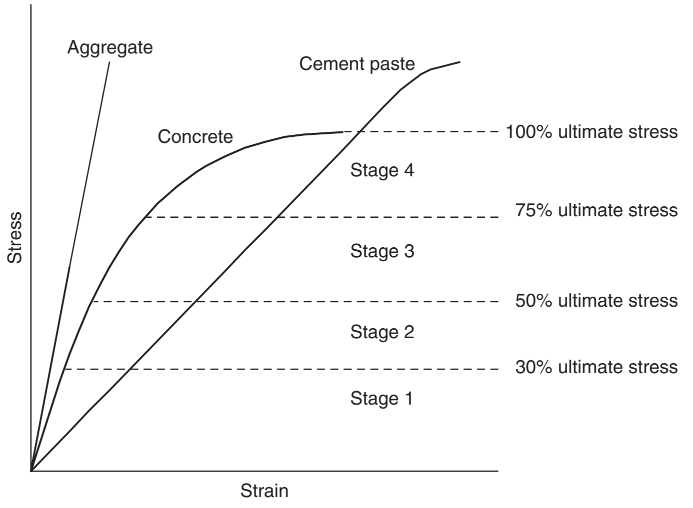


Figure 7 Typical stress–strain behaviour of aggregate, hardened cement paste and concrete under compressive loading. Development of cracking in concrete (adapted from Neville, 1995; Coutinho and Gonçalves, 1994; Domone, 1998)

Beyond this stage, around 75% of the ultimate strength, cracks propagate quickly through the mortar, bridging other transition zone cracks so that a continuous crack pattern is formed leading to very high strains and overall increase in volume, finally leading to complete breakdown and failure.

So, in a compressive strength test, the stress–strain curve obtained may be divided into four stages:

Stage 1	Pre-existent microcracks in the transition zone, remain stable; linear behaviour.
Stage 2	Cracks develop within the transition zone, still stable but cause non-linearity.
Stage 3	Cracks develop further within the transition zone but spread into the mortar, slowly causing further deviation from linearity.
Stage 4	Quick crack propagation bridging other growing cracks leading to very high strains, increase in volume and then failure.

Therefore, non-linearity behaviour in concrete is largely due to increasing strain caused by micro-cracking development as load level increases.

Creep and shrinkage

Moisture movement and heat, as well as applied stress, result in short- and long-term deformation of concrete.

Creep is the gradual increase in strain with time, under sustained load.

The magnitude of the creep strain is as great or greater than the elastic strains on loading, and it therefore often has a highly significant influence on structural behaviour. Creep is substantially increased when the concrete is simultaneously drying – that is, creep and shrinkage are interdependent (Domone, 1998).

'Basic creep' is the creep occurring in concrete at hygral equilibrium with ambient relative humidity. Simultaneous drying (i.e. shrinkage) increases creep, and the difference between the latter and basic creep is known as 'drying creep'. In practice, however, no such distinction is made and the term 'creep' is used indiscriminately whether or not drying is taking place.

A few mechanisms have been suggested to explain creep, and most of them attribute creep to movement of water inside the cement paste. Creep increases with the increase in the intensity of drying conditions – that is, with an increase in temperature and wind velocity and a decrease in relative humidity. Creep is also increased with stress-to-strength ratio and with an increase in moisture content. High-calcium fly ash and granulated blastfurnace slag tend to increase creep. On the other hand, creep decreases with an increase of aggregate concentration and rigidity (Soroka, 1993).

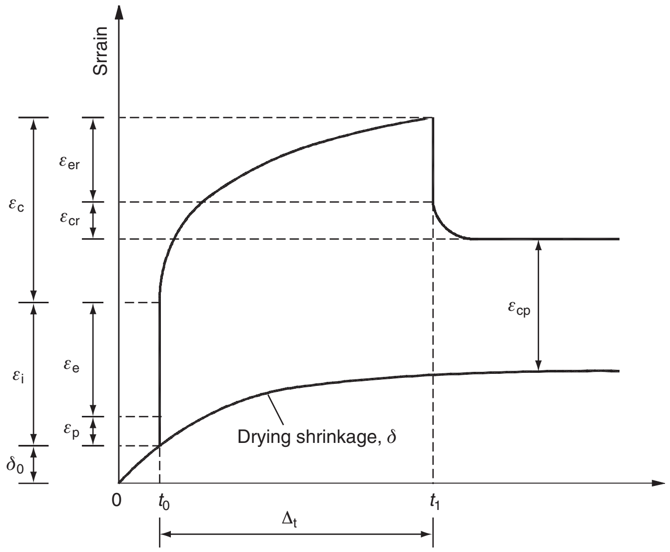


Figure 8 Short- and long-term deformation in concrete (Coutinho and Gonçalves, 1994. Reproduced by kind permission of Laboratório Nacional de Engenharia Civil (LNEC), Lisbon, Portugal)

Figure 8 gives an idea of the behaviour of concrete in a drying environment, if a uniaxial compressive stress is applied at age t_0, when the linear shrinkage strain is already δ_0. Compressive stress is maintained constant for the length of time Δ_t and removed at age t_1.

Until t_0, there is a net contraction in volume of the concrete resulting from drying, expressed as a linear strain (shrinkage). On loading there is an immediate strain response ε_i, proportional to the stress applied for low load-levels, and a creep strain ε_c, which increases with time at a decreasing rate. The immediate strain response is composed of a reversible or elastic strain, ε_e and an irreversible or plastic strain ε_p. This last component is relevant only in the first few loading cycles of concrete.

When at age t_1 concrete is unloaded, there is an immediate or elastic recovery strain, ε_{er} followed by a slow creep recovery with time, ε_{cr}, (Coutinho and Gonçalves, 1994).

Creep recovery is important in practice, to predict deformation of concrete when stress changes with time as the plastic, permanent or irreversible creep strain ε_{cp} is relevant.

Irreversible creep makes up the greater proportion of total creep, amounting to 80–90% of the total (Lawrence, 1998).

Permanent creep strain can reach values three times greater than elastic deformation and three times more important than drying shrinkage, for example in current cases of prestressed concrete.

Creep strain ε_c, as stated before, increases with time at a decreasing rate, but this is the case when the load is within certain limits. If load applied is very high after a first period of a decreasing rate with time (primary creep), it has been observed experimentally that creep strain increases at a constant rate for a period of a few hours (secondary creep). Finally, rate increases at an accelerated rate in a few minutes or seconds (tertiary creep) until failure.

Hardened cement paste consists of a porous structure with a minimum pore volume of about 28%. This minimum volume exists in the C–S–H gel completely filling the capillary pores. In other, more common situations porosity can attain values of over 50%, affecting the moisture movement which depends on the surrounding environment. A decrease in moisture content involves decrease in volume, i.e. drying shrinkage or simply shrinkage. Gain in moisture content involves swelling, i.e. volume increase.

Those parts of the microstructure of cement pastes that are involved in the mechanism of drying shrinkage suffer a progressive and irreversible collapse during first drying; the degree of collapse depends on the length of time during which specimens are dried and on the humidity to which they are dried (Lawrence, 1998).

Shrinkage is related to water loss and, accordingly, all factors that affect drying, such as relative humidity, temperature and air movement, affect shrinkage as well.

Shrinkage increases with an increase in the water and the cement contents and decreases with the rigidity of the aggregate and its content. The use of mineral admixtures or blended cements may be associated with a greater shrinkage.

In practice, shrinkage is restrained, and this restraint produces tensile stresses in the concrete. Consequently, concrete may crack if, and when, induced stresses exceed the tensile strength of the concrete (Soroka, 1993).

Failure and microcracking

Concrete microcracking in reinforced or prestressed concrete can develop into cracking that will lead to degradation and eventually to failure, through several processes. Mechanical processes such as caused by overload or creep, may cause serious cracking and eventually failure, but other processes related to environment will most certainly interact, for example chloride ingress or carbonation causing reinforcement corrosion, leading to accelerated cracking. Therefore, in a particular reinforced or prestressed concrete structure, degradation will occur due to a specific combination of a number of these processes which will often act synergistically. Naturally these processes can be related more with mechanical behaviour (overload, creep, etc.) or with durability (carbonation, corrosion, alkali–silica reaction, etc.).

It is also important to note that maintenance of a reinforced or prestressed concrete structure should always be considered and, therefore, although bad cases of degradation may occur, failure is not usual, except for inappropriate design (if not considering misusage or accident).

Degradation of reinforced or prestressed concrete is typically considered separating processes involving concrete and steel reinforcement, this is:

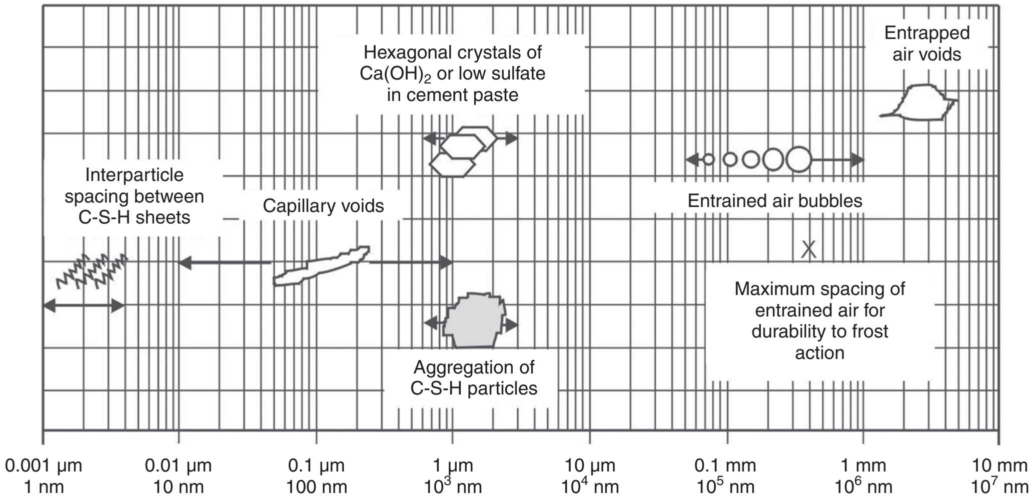


Figure 9 Dimensional range of solids and pores in hydrated cement paste (according to Mehta and Monteiro in Bertolini *et al*., 2004)

- concrete deterioration; and
- steel corrosion.

Concrete deterioration and steel corrosion processes are usually classified into the following groups:

Concrete deterioration	Mechanical	Ex.: plastic shrinkage, plastic settlement, overload, cyclic loading, thermal gradients, differential foundation movements, drying shrinkage, creep
	Physical processes	Ex.: freeze–thawing cycles, erosion, abrasion, cavitation
	Chemical processes	Ex.: acid attack, attack by pure water, sulfate attack, alkali–aggregate attack
	Biological processes	Ex.: biogenic attack, plant growth

Steel corrosion
- Carbonation
- Chloride ion penetration
- Stray currents
- Stress corrosion cracking
- Hydrogen embrittlement

Pore structure

Hardened paste is characterised by a highly porous structure which results from hydration reactions and contains interconnected pores of different sizes (**Figure 9**).

Hardened cement paste consists of:

- a residue of unhydrated cement, at the centre of the original grains;
- the hydrates, chiefly calcium silicates (C–S–H) but also some calcium aluminates, sulfoaluminates and ferrites;
- crystals of calcium hydroxide which occupy 20 to 25% of the volume of solids. The hexagonal crystals have dimensions of a few microns (ηm) (1 ηm $= 10^{-6}$ m);
- the unfilled residues of the spaces between the cement grains, called capillary pores, with dimensions varying from a few nanometers (nm) to about 5×10^3 nm (1 nm $= 10^{-9}$ m).

The calcium silicate hydrates (C–S–H) form a rigid gel and govern the mechanical properties. C–S–H occupies the largest proportion of the volume, approximately 50 to 60% of the completely hydrated cement paste.

Their structure varies from poorly crystalline fibres to a reticular or crumpled sheet-like network. The most important feature of this is its colloidal scale (gel particles are very small) and extremely high surface area estimated to be between 100 and 700 m^2/gm, i.e. about a thousand times greater than the cement particles from which it was formed (Domone, 1998). The cohesion forces are surface properties and, as such, increase with the decrease in the particle size or, alternatively, with the increase in their specific area. Accordingly, the mechanical strength of the set cement is attributable, partly at least, to the very great specific surface area of the cement gel (Soroka, 1993).

The spaces between the particles, known as **gel pores**, correspond to a volume of about 28% of the gel with diameter varying from fractions of a nanometer to several nanometers (**Figure 9**).

Capillary pores, as already mentioned, are the remains of the original water-filled spaces that have not become filled with hydration products and have dimensions varying from a few nm to about 5×10^3 nm. That is, gel pores are on average about two orders of magnitude smaller than capillary pores.

The volume of capillary pores varies and depends, in the first instance, on the original W/C ratio and subsequently on the degree of hydration.

If the cement paste is well hydrated and produced using low W/C ratio, these pores have dimensions of 10 to 50 nm but can reach up to 3–5 μm if the concrete is made using high W/C ratios or is not well hydrated (Bertolini *et al.*, 2004).

The largest pores, called **macropores** or entrapped air voids, which can be as wide as a few mm, result from entrapped air during the mixing process, which was not removed during compaction. Macropores can correspond to a volume of 1 to 5% of the total pore volume in concrete and even more in mortars. Macropores can be reduced if compaction is efficient and therefore workability must be adequate.

Sources of porosity include not only the types of pores mentioned but also entrained air bubbles, porosity in aggregate, porosity associated with paste–aggregate interfacial zones and microcracks and discontinuities associated with dimensional instabilities that occur during curing (Brown and Dex, 1991).

Entrained air bubbles, which are used in certain applications such as resistance to freeze–thaw attack (frost action), should be discrete, nearly spherical, so that no channels for the flow of water are formed and the permeability of concrete is not increased. They never become filled with hydration products because gel only forms in water.

These air bubbles are introduced intentionally in fresh concrete by air-entrainment admixtures and the diameters vary from about 0.05 to 0.2 mm (Bertolini *et al.*, 2004). Aggregate will present a porosity of 1–2%, but usually aggregate pores will not interfere with the transport mechanisms because they are discontinuous.

Reducing porosity in terms of macropores and capillary pores plays an important role in increasing mechanical strength. These pores are also relevant to durability of concrete because they affect the transport mechanisms. The influence of these pores on transport mechanisms cannot be explained simply by pore volume (Bertolini *et al.*, 2004) but the concept of connectivity, i.e. the degree of continuity of the pore system, must be considered, i.e. the so-called ‘open porosity’.

Open porosity consists of pores which are interconnected and enable transport of liquids or gases, or exchange of dissolved substances. It corresponds to the maximum reversible water content and, for cement paste, lies in the region of 20 to 30% (CEB, 1992). The pore size distribution particularly influences the rate of transport.

Porosity, which is of the order of 50% (Soroka, 1993), and pore size distribution are functions of W/C ratio and degree of hydration and are strongly influenced by curing conditions and cement composition.

Permeability

Most concrete properties depend much more on the physical other than the chemical characteristics of the porous structure.

Permeability of hardened cement paste is practically nil for capillary porosities of less than 10%, which corresponds to overall porosity in the region of 35%. This threshold is generally interpreted as a transitional stage between a system of interconnected pores and a system of isolated pores. With equal overall porosity, permeability depends on pore size distribution, pore continuity and isotropy. Concrete permeability depends largely on pores whose diameter exceeds 100 nm; an essential role is also played by their shape and connectivity. Mention is also made of the influence of the specific area of hydration products; cement pastes with a large specific surface area can trap a large amount of water, thus slowing down the migration of humidity in pastes with a high capillary porosity (Ollivier *et al.*, 1995).

Conclusions regarding concrete permeability are frequently based on extrapolation of results obtained for cement pastes. However, it may be reasonable to subdivide the porosity in concrete into two classes:

1 that in the paste matrix;
2 that associated with the aggregate and paste interface.

The principal source of the matrix porosity contributing to permeability is that associated with residual space between cement grains which was originally filled with water. The contribution of this source of porosity towards permeability may be amenable to assessment by investigations of cement paste. However, the contribution of the porosity associated with the interfacial zones between paste and aggregate and the microcracks that develop in the interfacial region, which extend to the paste, must be assessed by determinations carried out directly on aggregate containing materials (Brown and Dex, 1991). Nevertheless the W/C ratio and the degree of hydration continue to be the main factors that determine capillary porosity.

As we have seen, hardened cement paste and concrete contain pores of varying types and sizes, and therefore the transport of substances through concrete can be considered as a particular case of the more general phenomenon of flow through a porous medium.

There are three fluids principally relevant to durability which can enter concrete: water, pure or carrying aggressive ions; carbon dioxide and oxygen. Durability of concrete largely depends on the ease with which fluids, both liquid and gases, can enter into, and move through, the concrete; this is commonly referred to as permeability of concrete. Strictly speaking, permeability refers to flow through a porous medium. Now the movement of the various fluids through concrete takes place not only by flow through the porous system but also by diffusion and sorption, so that our concern is really with **penetrability** of concrete (Neville, 1995). Nevertheless the commonly accepted term is permeability to describe this general property.

Penetrability of concrete – this is, the ease of movement of fluids through concrete – involves three mechanisms already stated: permeability, diffusion and capillary absorption.

1 Permeability corresponds to flow caused by a pressure differential.
2 Diffusion is the flow resulting from a differential in concentration.
3 Capillary absorption, only possible in partially dry concrete, results from capillary movement in the pores which are open to the ambient medium.

Permeability – flow caused by a pressure differential

Flow caused by a pressure differential is governed by Darcy's law for laminar flow:

$$V = ki \tag{3}$$

where:

V = flow velocity (m/s)
$i = \frac{H}{L}$ – hydraulic gradient (dimensionless)
k = coefficient of permeability (dimensions of a flow velocity) (m/s)

As the flow velocity corresponds to the ratio between flow of water Q (m^3/s) through a cross-sectional area of A (m^2) and the hydraulic gradient is the ratio between the drop in hydraulic head through the sample H (m) and the length of the sample L (m) along which the water is flowing, then:

$$V = \frac{Q}{A} = k\frac{H}{L} \tag{4}$$

This enables determination of the coefficient of permeability for steady-state flow of water at room temperature. These assumptions correspond to a simplification of reality as the hydraulic gradient is not constant and the pressure variation inside concrete is unknown (Coutinho and Gonçalves, 1994).

Diffusion

Diffusion, flow resulting from a concentration gradient, is an important mechanism in concrete. In terms of diffusion of gases, carbon dioxide, leading to carbonation of concrete and diffusion of oxygen feeding corrosion of embedded steel, is of primary importance. Gases diffuse four to five times orders of magnitude quicker through air than in water. Diffusion of aggressive ions such as chlorides and sulfates takes place only in the pore water and it is more intensive in saturated pores. Carbon dioxide, oxygen, chlorides and sulfates when diffusing move through pores from the surface of concrete where they are present in higher concentrations, to the interior, where concentrations are lower.

Diffusion is governed by Fick's first law, under conditions of **stationary** (unidirectional and constant) mass transfer (Crank, 1975):

$$F = -D\frac{\partial C}{\partial x} \tag{5}$$

where, for x-direction:

F = transfer rate of the substance per unit area normal to the x-direction (kg/m^{2}s)
$\frac{\partial C}{\partial x}$ = concentration gradient (kg/m^4)
C = concentration of the diffusing substance at distance x from the surface
D = diffusion coefficient (m^2/s)

D is the effective diffusion coefficient as F and D refer to the cross-section of the concrete sample but effectively flow takes place only through the pores.

Laboratory tests (LNEC E383; Page *et al.*, 1981) based on Fick's first law, have been developed to compare diffusion coefficients of different types of concrete in terms of chloride diffusion.

In most cases diffusion depends on time t, (**non-stationary** conditions) and is governed by Fick's second law (Crank, 1975):

$$\frac{\partial C}{\partial t} = D\frac{\partial^2 C}{\partial x^2} \tag{6}$$

If the coefficient of diffusion D is considered invariable in time, if concrete is considered homogeneous so that D is constant through the thickness of the concrete, if the concentration of the diffusing ion, taken at the concrete surface, C_s, is constant in time and if C_0 is the initial ion concentration in the concrete, then the solution for Fick's second law is (Crank, 1975):

$$C(x,t) = C_s - (C_s - C_0)\,\mathrm{erf}\,\frac{x}{2\sqrt{Dt}} \tag{7}$$

where $\mathrm{erf}\,(z)$ is the error function.

$$\mathrm{erf}\,(z) = \frac{2}{\sqrt{\Pi}}\int_0^z e^{-t^2}\,dt \tag{8}$$

The values of the error function have been tabulated or can be calculated (for further reading see Crank, 1975).

Equation (7) is often utilised to describe mathematically the experimental profiles of chloride concentration found in non-steady-state laboratory tests or from structures exposed in the field. The values of C_s and the effective D are obtained by optimising the fit of equation (7) to the experimental chloride profile. Values of the effective D may vary from 10^{-13} to 10^{-11}m^2/s for various binders and

S: mg/mm^2 × min.$^{0.5}$	Concrete quality	Examples		
		S	W/C	Strength: MPa
$S > 0.2$	Low			
$0.2 < S < 0.1$	Medium	0.170	0.6	
$S < 0.1$	High	0.090	0.4	
		0.073	0.45	
		0.039	0.5	43
		0.012	0.4	67
		0.005	0.3	90

Table 1 Sorptivity values for concrete (adapted from Browne, 1991; Neville, 1995 and Bertolini *et al*., 2004)

W/C ratios. The diffusion coefficient D can be used as the main parameter that describes the rate of chloride penetration and together with the fitted C_s and using equation (7) the chloride penetration can be extrapolated to longer times with limits. A non steady-state test has been standardised in the Nordic countries (Bertolini *et al*., 2004).

Capillary absorption

The absorption of water into concrete under capillary action is dependent on the square-root of time and may be modeled (Hall, 1989) by the following equation:

$$A = a_0 + St^{0.5} \tag{9}$$

where A (mg/mm^2) is the water absorption by unit area of concrete surface since the moment the specimen was dipped in water, S is the sorptivity of the material (mg/(mm^2 × min$^{0.5}$)), t is the elapsed time (min) and a_0 (mg/mm^2) is the water absorbed initially by pores in contact with water.

In practice, pre-dried specimens (40°C) are rested on supports in a shallow water bath of a few mm and the increase in mass of water absorbed with the square root of time is recorded. Usually results for the first four–five hours provide a very good linear fit and enable the determination of the sorptivity of the material. Examples of sorptivity are given in **Table 1**.

Permeability, porosity, W/C ratio and curing

Figure 10 is a simplified schematic representation (for example, the calcium hydroxide crystals and the unhydrated cores of cement grains were omitted) showing the physical effect of curing for different ranges of W/C ratios.

In fact, permeability of cement paste varies with the progress of hydration. In a fresh paste, the flow of water is controlled by the size, shape and concentration of the original cement particles. With the progress of hydration, the permeability decreases rapidly because the gross volume of gel, including the gel pores, is approximately 2.1 times that of the unhydrated cement, so that the gel gradually fills.

The steep increase of the water permeability above approximately 25% porosity (corresponding to a W/C ratio of 0.45 with a degree of hydration of 75%, **Figure 11(a)**) is the background of specified values in the codes of practice for high-quality concrete. For instance, **Table 2** shows the relationship between W/C ratio and degree of hydration in order to achieve segmentation of the capillary pores over 50 nm, in a Portland cement paste (Bertolini *et al*., 2004).

Influence of constituent materials

Cement

Most of what was discussed in the previous sections refers to Portland cement, which results from grinding clinker and a certain quantity of calcium sulfate, usually gypsum. Clinker is obtained by intimately mixing together calcareous and argillaceous and iron oxide-bearing materials, burning them at clinkering temperature, about 1450°C where about 20% of the material melts.

Cement is the material that most influences properties of concrete, both in terms of strength and durability. (We shall solely deal with CEM cement, i.e. in accordance with EN 197-1: 2000, which requires the presence of the Portland cement component and excludes, for example, high-alumina cement.)

One of the main advantages of using concrete is that it can be modified by the engineer so as to obtain different concrete properties to suit each specific application. The most important modifications concern Portland cement which may be achieved in two different ways:

1. A first way consists of altering the clinker compound composition (C_3S; C_2S, C_3A and C_4AF) by changing mainly the type and quantities of raw materials, in the cement manufacture.
2. A second way consists of using partial cement replacement materials, such as ground granulated blastfurnace slag, pulverised fuel ash, microsilica, metakaolin and rice husk ash, and additions such as limestone powder (**Figure 12**).

Global effect of use of Cement replacement materials in concrete

It has been well established that sustainable development of the cement and concrete industries can be achieved by complete utilisation of cement replacement materials, especially pozzolanic by-products (Papadakis, 2000). Substantial energy and cost savings can result when industrial by-products are used as a partial replacement for the energy-intensive Portland cement. Each ton of Portland

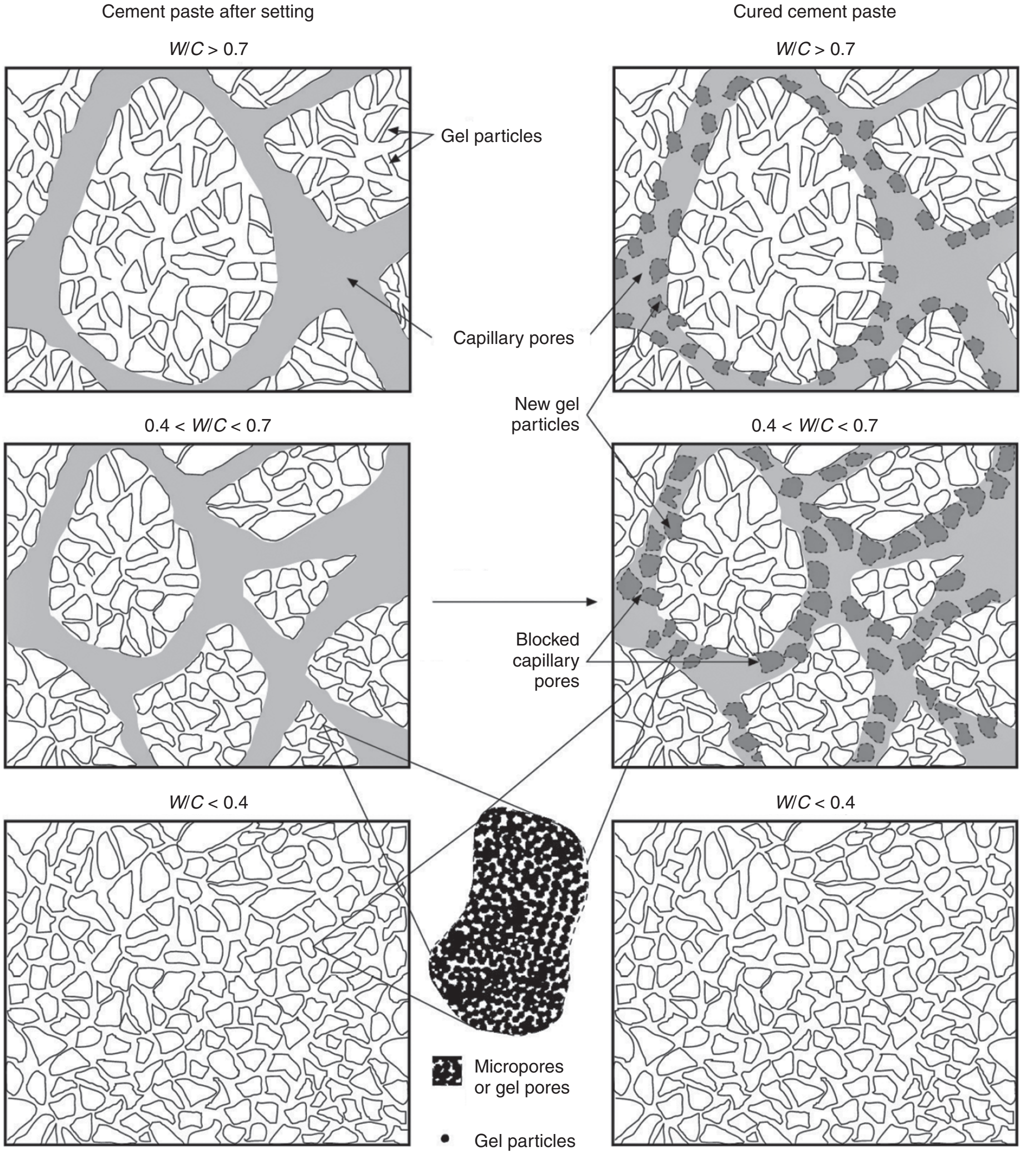


Figure 10 Simplified schematic representation showing the effect of curing on segmentation of capillary pores for different ranges of W/C ratios (Sousa-Coutinho, 2005)

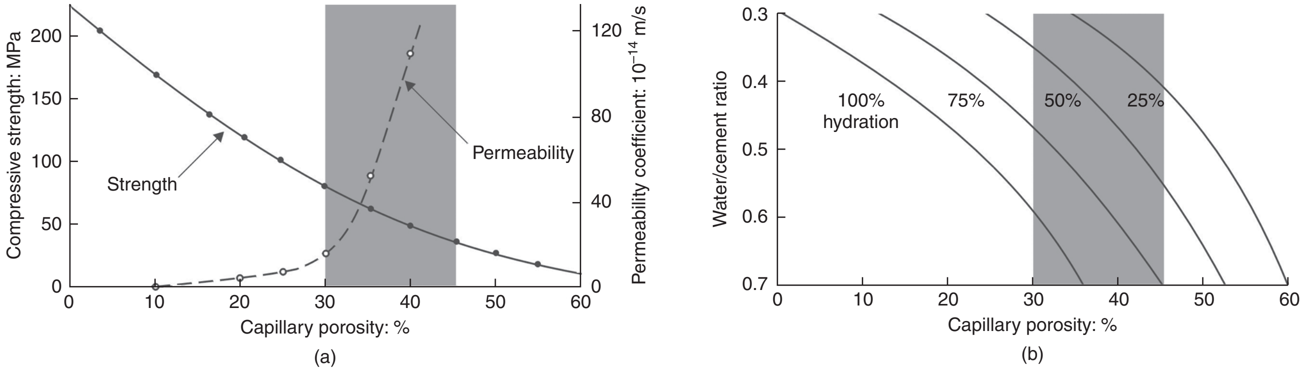


Figure 11 (a) Influence of capillary porosity on strength and permeability of cement paste. (b) Influence of water to cement ratio (W/C) and degree of hydration on capillary porosity for cement paste (adapted from Powers, 1958; Bertolini *et al*., 2004)

W/C	Degree of hydration: %	Curing time
0.4	50%	3 days
0.45	60%	7 days
0.5	70%	14 days
0.6	92%	6 months
0.7	100%	1 year
>0.7	100%	Impossible

Table 2 Curing time necessary to achieve a degree of hydration capable of segmenting capillary pores over 50 nm in a Portland cement paste (adapted from Bertolini *et al*., 2004)

cement clinker production is associated with a similar amount of CO_2 emission, one of the green-house effect gases, responsible for global warming, reduction of the ozone layer and acid rain (Mehta, 1994). The cement industry contributes over 6% to global CO_2 emissions, making the cement industry an important sector for CO_2 emission mitigation strategies such as increased use of additions (Worrell *et al*., 2001; Naik, 2005).

CO_2 emissions represent currently one of the most important environmental factors. The Kyoto Protocol adopted at the International Conference for the Prevention of Global Warming in 1997 required Japan, the USA and the EU to reduce by 2010 their emissions of greenhouse gas by 6.7 and 8%, respectively, compared to 1990 levels. As a reflection of the Kyoto Protocol some countries have already started to implement legal steps to support achievement of this goal. For example, the EU initiated and supports several international projects such as IPPC (Integrated Pollution Prevention Control), etc. However, the general agreement of the Kyoto Protocol has not yet been achieved and the amount of CO_2 emissions is still increasing (fib bulletin, 2004).

The presence of cement replacement materials in concrete is known to impart significant improvements in durability and often in workability. The use of by-products is an environmental-friendly method of disposal of large quantities of materials that would otherwise pollute land, water and air (Mehta, 1994). The cement production rate of the world, which was approximately 1.5×10^9 t/year (Glass, 2001), is expected to grow to about 2×10^9 t/year by 2010 (Mehta, 2001) and therefore further increase in cement demand is expected to be met by the use of these cement replacement materials.

Aggregate

As highlighted earlier, the strength of concrete depends mainly on the strength of cement paste although most of the volume of concrete corresponds to aggregate (70–80%). Concrete strength also depends on the strength of bond between the hydrated paste and aggregate, the so-called ‘transitional zone’, and less on other properties of the aggregate. Normal-weight aggregates are usually much stronger than hydrated cement paste and consequently concrete strength hardly depends on strength of the aggregate. On the contrary, strength of lightweight aggregates affects lightweight concrete which can be observed in the mode of concrete failure. For normal-weight concrete fracture occurs mainly in the paste aggregate interface but in lightweight concrete fracture extends throughout the aggregate.

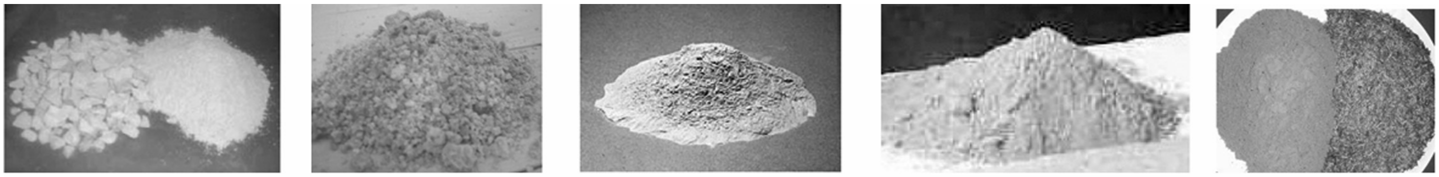
The strength of the bond between the hydrated paste and aggregate depends mainly on the strength of the paste, which is a result of the W/C ratio. Increase in roughness of aggregate surface will increase bond. In fact, it has been shown that concrete made with crushed aggregate is stronger than the same concrete made with gravel. This effect on compressive strength is also valid for flexural strength.

Aggregate content in concrete, grading, shape, stiffness of aggregate particles and maximum size of aggregate also influence concrete strength but in a secondary way.

In general, durability of concrete is affected by the type of bond between the hydrated paste and aggregate; this is the transitional zone. In fact this zone is weaker and more porous thus contributing to the connected pore system of concrete. Therefore the transitional zones contribute to permeability thus affecting durability.

In particular, other situations such as aggregate containing unsound particles, reactive silica (alkali–silica reaction) or some type of dolomitic limestone aggregate (alkali–carbonate reaction) may affect durability of concrete.

Unsound aggregate can expand on freezing or even on exposure to water and cause disruption ranging from popouts to extensive surface cracking and to disintegration over a considerable depth and can thus vary from no more



Lime filler Slag Fly ash Silica fume Rice husk ash

Figure 12 Cement replacement materials than impaired appearance to a structurally dangerous situation. Some porous flints and cherts, some shales and limestones can exhibit unsoundness (Neville, 1995). Aggregates to use in concrete must be tested for this property.

Amorphous silica in aggregate can react with alkalis present in hydrated Portland cement paste forming a gel that absorbs water and swells so as to cause cracking and disruption of concrete. This reaction is known as alkali–silica reaction or ASR. Active silica occurs as minerals opal, chalcedony, crystobalite and trydimite and as volcanic glass. These can be found in some flints, limestones, cherts and tuffs. Only a small portion of reactive silica in the aggregate (as low as 0.5%) can cause disruption of the concrete. The process is very slow and the cracking can take years to develop but once started it is almost impossible to eliminate and therefore extensive rebuilding or strengthening of the concrete structure may be required (Domone, 1994). Tests can detect reactive silica in aggregate.

Some dolomitic limestone aggregates can react with alkalis in cement and cause expansion in concrete. No simple test to identify these aggregates has been identified but these aggregates are not frequent.

Admixtures

Admixtures are chemicals added to concrete in small dosages (under 5% cement by weight), typically 1 to 2% during mixing which impart a specific modification or modifications compared to the normal properties of concrete. Admixtures are not an essential component of the concrete mix but their popularity and use have increased considerably in recent years.

In 1989 it was estimated that about 40% of all concrete produced in the UK contained an admixture and in North America, Australia and Japan, the proportion was even higher (Domone, 1998).

Admixtures are capable of imparting considerable physical and economic benefits with respect to concrete. These benefits include the use of concrete under circumstances where previously there existed considerable, or even insuperable difficulties. They also make possible the use of a wider range of ingredients in the mix (Neville, 1995) and enable production of self-compacting concrete (SCC). Self-compacting concrete, initially developed in Japan, is an innovative type of concrete whose main characteristic is to fill the forms and consolidate without the need of vibration. In recent years, there has been a growing interest in SCC technology among constructors and the construction industry in several countries. The principal reasons for this interest concern the ease of placing this type of concrete in heavily reinforced areas difficult to access, the reduced effort in accomplishing some of the casting tasks and the significant reduction of the construction period. Along with these advantages, in terms of environment, this technology will enable a considerable reduction of the acoustic noise levels and the use of cement replacement materials (Nunes *et al.*, 2006).

Admixtures may be organic or inorganic and they are usually classified by their function in concrete:

- water-reducing/plasticising admixtures;
- high-range water-reducing/superplasticising admixtures;
- water-retaining admixtures;
- air-entraining admixtures;
- set-accelerating admixtures;
- hardening-accelerating admixtures;
- set-retarding admixtures;
- water-resisting admixtures;
- set-retarding/water-reducing/plasticising admixtures;
- set-retarding/high-range water-reducing/super-plasticising admixtures;
- set-accelerating/water-reducing/plasticising admixtures.

Concluding remarks

- Properties of hardened concrete depend on composition and characteristics of materials used but also on production processes as it is not delivered ready to use like other structural materials.
- Concrete has been used in its simplest forms for over seven thousand years but its development intensified in the nineteenth century with the discovery of Portland cement and the use of reinforced concrete. Today concrete is the second most used material worldwide.
- In the early times of the use of reinforced concrete, cement content was high, concrete cover was generous and great care was taken in the production phases, delaying corrosion of reinforcement and making reinforced concrete seem everlasting. Durability had not yet become a concern, with strength being the only acceptance criterion.
- Concrete is mainly used to withstand compression in a structure, but tensile strength is also important to avoid serious cracking and therefore maintaining continuity and also protecting reinforcement against corrosion in the case of reinforced concrete structures.
- Strength mainly results from the W/C (water/cement) ratio. Strength is assessed through a compression test and two tests that enable determining indirectly the tensile strength – the flexural test and the splitting test.
- Failure of concrete results from cracking which develops from microcracking. Non-linear behaviour in concrete (above 30% ultimate load) is largely due to increasing strain caused by microcracking development as load level increases.
- Moisture movement and heat, as well as applied stress, result in short- and long-term deformation of concrete, such as creep. Creep is the gradual increase in strain with time, under

sustained load. Permanent creep strain can reach values three times greater than elastic deformation and three times more important than drying shrinkage, for example in current cases of prestressed concrete.

- Drying shrinkage or simply shrinkage is a decrease in volume caused by reduction in moisture content in hardened cement paste. In practice, shrinkage is restrained, and this restraint produces tensile stresses in the concrete. Consequently, concrete may crack if, and when, induced stresses exceed tensile strength.
- Concrete microcracking in reinforced or prestressed concrete can develop into cracking which will lead to degradation and eventually to failure, through mechanical processes (overload, creep, etc.) and/or other processes related to environment/durability (carbonation, corrosion, alkali–silica reaction, etc.) causing reinforcement corrosion, leading to accelerated cracking. Degradation will occur due to a specific combination of a number of these processes which will often act synergistically.
- Maintenance of a reinforced or prestressed concrete structures should always be considered and, therefore, although bad cases of degradation may occur, failure is not usual, except for inappropriate design (if not considering misusage or accident).
- Porosity, which is of the order of 50%, and pore size distribution are functions of W/C ratio and degree of hydration and are strongly influenced by curing conditions and cement composition.
- The ease of movement of fluids through concrete involves three mechanisms: permeability, diffusion and capillary absorption.
- Permeability of cement paste decreases rapidly with the progress of hydration. In a mature paste, the permeability depends on size, shape and concentration of gel particles and on whether or not the capillaries have become segmented, showing the importance of curing.
- Cement is the material which most influences properties of concrete, both in terms of strength and durability. Portland clinker compound composition (C_3S; C_2S, C_3A and C_4AF) can be engineered by changing mainly the type and quantities of raw materials in the cement manufacture.
- Modification of clinker compound composition can lead to sulfate-resisting cement, low-heat cement, rapid hardening cement and white cement. These alterations can be performed up to certain limits which may not modify concrete properties as necessary, for certain applications. Further and more versatile alterations can be achieved if partial cement replacement materials, i.e. additions, are used.
- Cement replacement materials or additions are finely ground inorganic materials, usually finer than cement, which can improve workability, durability and sometimes strength.
- Additions are commonly divided into three groups: low-active (lime filler), materials with latent hydraulic properties (slag) and pozzolanic materials (metakaolin, rice husk ash, fly ash, silica fume etc.).
- Using additions in cement generates very fine products of hydration that lead to refinement of pores, reducing permeability and therefore increasing durability. However, hydration reactions of these materials generally take longer than Portland cement and so refinement of pores will only occur if curing of concrete is long enough.
- Many of these cement replacement materials are by-products or even waste materials, usually imparting enhanced resistance to aggressive agents and therefore contributing to durability of concrete structures and sustainable construction.
- Each ton of Portland cement clinker production is associated with a similar amount of CO_2 emissions. The cement industry contributes over 6% to global CO_2 emissions making the cement industry an important sector for CO_2 emission mitigation strategies such as increased use of additives.
- Future increase in cement demand is expected to be met by the use of these cement replacement materials.
- Concrete strength depends mainly on the strength of cement paste although most of the volume of concrete corresponds to aggregate (70–80%). Strength also depends on the bond between the hydrated paste and aggregate which increases with roughness of aggregate surface.
- Durability of concrete is affected by the bond between the hydrated paste and aggregate where it is weaker and more porous thus contributing to the connected pore system of concrete and therefore to permeability.
- Amorphous silica in aggregate can react with alkalis present in hydrated Portland cement paste forming a gel that absorbs water and swells so as to cause cracking and disruption of concrete. This reaction is known as alkali–silica reaction or ASR.
- Admixtures are chemicals added to concrete in small dosages (under 5% cement by weight), typically 1 to 2%, during mixing which impart a specific modification or modifications compared to the normal properties of concrete. Admixtures are not an essential component of the concrete mix but their popularity and use have increased considerably in recent years.

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Chapter 16

Durability of concrete

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Concrete is one of the most extensively used materials in the world, due to its reputation as a strong and durable material with flexibility to form any difficult shape. Unfortunately this reputation has been tarnished due to many reported cases of premature deterioration of concrete structures. In this chapter, the mechanisms of deterioration of concrete structures are summarised and factors influencing them are identified. It is not the intention to elaborate on any of the deterioration mechanisms in detail, but to highlight factors which need to be given attention while designing durable concrete structures. In this context, methods of classifying exposure environment as per the current European standard are summarised. The importance of transport properties to durability is also demonstrated.

CONTENTS

Causes of deterioration of concrete

The technical term for the damage of concrete is deterioration, in contrast to corrosion in the case of damage to metals. Although it is difficult to classify the causes of concrete deterioration due to the interacting nature of various factors, efforts have been made in the past to group the various types as physical and chemical. Based on the durability problems reported in various technical literatures (Mehta and Gerwick, 1982; ACI Committee 201; Bjegovic *et al.*, 1987; Building Research Establishment, 1982; MacCraith, 1986), the causes of deterioration of concrete are as follows:

- abrasion
- alkali–aggregate reaction
- chemical attack
    - acid attack
    - alkali attack
    - carbonation
    - chloride attack
    - leaching
    - salt attack
    - sulfate attack
    - delayed ettringite formation
    - thaumasite attack
- corrosion of reinforcement
- cracks
- freeze–thaw and frost damage.

Among the various causes listed above, carbonation, chloride ingress and leaching can all lead to the corrosion of reinforcement, whereas their chemical effect on the integrity of the concrete is less important. Therefore, they are identified as factors influencing the corrosion (Basheer *et al.*, 1994). Similarly, the action of salt is not solely due to the chemical reaction with the concrete but is also due to the crystallisation pressure in capillary pores in concrete (Neville, 1996). Hence, the various topics are discussed in the order given in **Table 1**.

Corrosion of reinforcement in concrete

Causes of corrosion

Steel in concrete is naturally protected from corrosion because of the high alkalinity of the Portland cement matrix (Bjegovic *et al.*, 1987; Newman, 1986; Neville, 1983). This is because a thin film of iron oxide is formed on the steel, making it passive at high pH of the hydrated cement paste (HCP). Further protection is provided by the concrete physically by minimising the *penetration of water and air* into the cover zone. The period for which

Causes of deterioration	Principal component of reinforced concrete affected
Corrosion	Steel
Carbonation	Hydrated cement paste and steel
Chloride attack	Hydrated cement paste and steel
Leaching	Hydrated cement paste and steel
Freeze–thaw and frost damage	Hydrated cement paste and aggregate
Salt attack and salt scaling	Hydrated cement paste and steel
Chemical attack	Hydrated cement paste and aggregate
Sulfate attack	Hydrated cement paste
Delayed ettringite formation	Hydrated cement paste
Thaumasite attack	Hydrated cement paste
Acid attack	Hydrated cement paste and aggregate
Alkali attack	Hydrated cement paste
Alkali–aggregate reaction	Aggregate and hydrated cement paste
Cracking	
Prehardening state	Hydrated cement paste
Hardened state	Hydrated cement paste
Abrasion and erosion	Hydrated cement paste and aggregate

Table 1 Mechanisms of deterioration of reinforced concrete structures

this protection remains effective depends on a number of factors including the physical integrity of the cover. If the passivating film is destroyed due to one or more of the following reasons, steel becomes active and prone to corrosion.

1. Leaching of the alkaline material with water, and a reduction in pH.
2. Partial neutralisation (specifically, a reduction in pH below 9) by chemical reaction between the products of hydration of cement and carbon dioxide which *diffuses* in from the atmosphere.
3. Cracks in the concrete which allow carbon dioxide to *permeate* easily to the reinforcement zone.
4. The presence of chloride ions in the reinforcement zone. Even though the presence of chlorides near steel destroys the passivating film on steel, the actual mechanism of corrosion in the presence of chlorides is still a subject of contention (Bjegovic *et al*., 1987; Gonzalez *et al*., 1990).

For corrosion to occur, it is necessary not only that the passivating film be destroyed but also that there exists a difference in electrochemical potential within the steel–concrete system. This difference in potential may be due to any one of the following causes:

1. Differences in the steel itself, in that so-called active sites are present.
2. Differences in the chemical environment in the concrete, i.e. in alkalinity, due to either an uneven distribution of alkalis within the cement or a *non-uniform carbonation*.
3. Local *variations in the permeability* of concrete which may produce local dissimilarities in the chemical environment of steel (e.g. chloride ion concentration).
4. Differences in the exposure conditions of concrete and the thickness of cover which would allow *permeation of aggressive gases and liquids in different rates* resulting in different concentrations.

Therefore, both the depassivation of steel in concrete and the existence of an electrochemical potential difference result in the corrosion of steel. It may be observed that most of the factors listed against the initiation of corrosion can be related to the permeation characteristics of concrete.

Mechanism of electrochemical corrosion of steel in concrete

In electrochemical corrosion of steel in concrete, electric cells, called galvanic cells (**Figure 1**), are formed in various ways. However, in all circumstances, there are four prerequisites viz. an anode site, a cathode site, electrical conductivity between the cathodic and anodic sites, and a pathway (electrolyte) that provides ionic continuity between the anodic site and cathodic site. In the context of corrosion of steel, these four prerequisites can be defined as follows:

Anode: where the ions go into solution and electrons are released.

Cathode: where the electrons are consumed in the presence of water and oxygen.

Conductor: steel itself, which permits the transfer of electrons from the anode to the cathode.

Electrolyte: the moist concrete which permits the movement of ions between the cathode and the anode.

At the anode (the first prerequisite), the steel is oxidised. Therefore, the anodic reaction is:

$$2Fe \rightarrow 2Fe^{++} + 4e^{-} \quad \text{(anodic half-cell)}$$

The positively charged ferrous ions (Fe^{++}) pass into the solution and the free electrons (e^{-}) flow to the cathode (the second prerequisite) along the steel (the third prerequisite). The free electrons are absorbed at the cathode and combine with water and oxygen to form hydroxyl ions

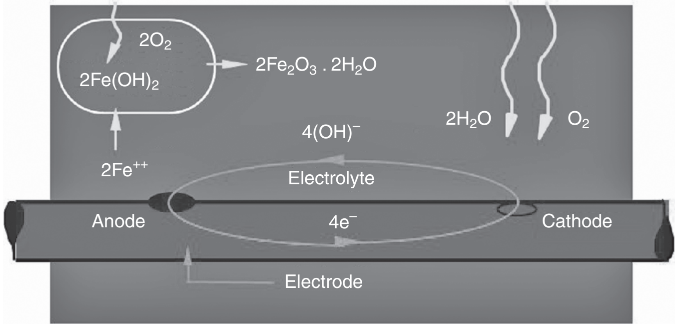

Figure 1 Simplified model illustrating galvanic corrosion of steel in concrete

(OH^-). Therefore, the reaction at the cathode is:

$$4e^- + O_2 + 2H_2O \rightarrow 4(OH)^- \quad \text{(cathodic half-cell)}$$

This reaction ensures electrical neutrality to be maintained between the two half-cell regions. The last prerequisite is that there must be a medium that allows ionic migration between the half-cells. This is provided by the moist concrete. Therefore, any electron current which may exist will flow from the anode to the cathode through the electrical interconnection between the two (steel), then through the electrolyte (moist concrete) as charges of ions back to the anode to complete the electrical circuit. The positive ion flow or the corrosion current flow in the electrolyte is opposite to the direction of electron flow.

The hydroxyl ions produced due to the cathodic reaction combine with the ferrous ions at the anode to form ferrous hydroxide and the oxidation converts it into insoluble hydrous ferric oxide or rust.

$$2Fe^{++} + 4(OH)^- \rightarrow 2Fe(OH)_2$$

$$2Fe(OH)_2 + 2O_2 \rightarrow 2Fe_2O_3 . 2H_2O$$

The transformation of metallic iron to rust is accompanied by an increase in volume. Generally in the case of corrosion of steel, the rust occupies more than twice the volume of the iron which has been oxidised (Gonzalez *et al*., 1990; Mehta and Monteiro, 2005) and in the process of its formation can exert a pressure greatly exceeding the ability of even the strongest concrete to resist. As a result, *the concrete cracks* and, if the corrosion continues, it may spall off.

If for any reason, any of these four prerequisites are absent, the corrosion process cannot occur. It may be observed that there is no corrosion if there is no water and oxygen at the cathode. Therefore, there is no corrosion in dry concrete.

Mechanism of corrosion in presence of chlorides

It has been reported (Gonzalez *et al*., 1990) that the ingress of chlorides into concrete may result in at least two types of corrosion: a local black variety and a general red variety, depending on the distribution of chlorides in concrete and the availability of free oxygen at the reinforcement. The black corrosion occurs when there is not much free oxygen available at the reinforcement and is characterised by a local pitting of the attacked steel leading to a soft black corrosion deposit. With cover concrete of a reasonable strength and depth, the soft corrosion products will not expand with sufficient force to break away this concrete. However, if oxygen is sufficiently available, the corrosion process results in a general red surface rust on the reinforcement. Inevitably the concrete cover is forced off by this harder and more expansive corrosion product, allowing more oxygen and moisture penetration along the bar, giving the familiar progressive corrosion.

Several researchers (Newman, 1986; Ann and Song, 2007; Fraczek, 1987; Hime and Erlin, 1987) have attempted to explain the specific effect of chloride ions on the corrosion of steel. Most researchers believe that the chloride ions initiate the corrosion reaction by depassivating the natural oxide film on the steel surface, allowing the iron to decompose into the solution. However, the depassivation does not necessarily occur through a direct reduction of alkalinity in the electrolyte by the chloride-induced reaction. It is also reported that (Fraczek, 1987) some researchers believe in the migration of chloride ions through the oxide film and attack on the steel directly. However, a simple theory is that a change in electrochemical potential due to a non-uniform penetration of chlorides into the concrete to the level of reinforcement leads to the formation of corrosion cells. Accordingly, the iron is oxidised faster in the presence of chloride ions and a complex ferric chloride is formed. This is unstable and hence reacts with the available hydroxyl ions to form ferric hydroxide. This releases the chloride ions back into the solution and consumes hydroxyl ions, as shown in the following chemical reactions:

$$2Fe + 6Cl^- \rightarrow 2FeCl_3 + 4e^-$$

$$FeCl_3 + 2OH^- \rightarrow Fe(OH)_2 + 3Cl^-$$

The electrons released in the oxidation reaction of iron flow through the steel to the cathode surface. This process would result in a concentration of chloride ions and a reduction of the pH at the points of corrosion initiation, probably accounting for pitting corrosion. The lowered pH at these sites contributes to the continual breakdown of the passive oxide film (Fraczek, 1987). However, it is important to emphasise that in the presence of chlorides, just as in their absence, electrochemical corrosion proceeds only when both water molecules and air are available.

Factors influencing corrosion

It is widely accepted (Beeby, 1979; Cookson, 1978; Rasheeduzzafar *et al*., 1989) that the process of corrosion consists of two phases, an initiation phase and a propagation phase. The initiation phase lasts from the construction of a member until certain conditions such as chloride level, carbonation, etc. are reached. The propagation or the active phase is the one in which corrosion takes place at a significant rate. Many factors affect these two phases, which could be explained with a corrosion–permeability interaction model, which is presented elsewhere. However, the effects of carbonation and chloride ingress are discussed in this section.

Effect of carbonation

Carbon dioxide forms about 0.03% of the atmosphere (Chemical Rubber Company, 1975) and can react with almost all hydrates in the cement paste matrix of concrete at this normal concentration. When carbon dioxide diffuses into concrete, it combines with water, forming carbonic acid which then reacts with the alkali hydrates, forming carbonates. First, calcium carbonate is formed by the combination of carbon dioxide with calcium hydroxide and calcium silicate hydrate:

$$\mathrm{Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O}$$

$$\mathrm{(C_3S_2H_3) + 3CO_2 \rightarrow 3CaCO_3 + SiO_2 + 3H_2O}$$

The second stage takes place as moisture containing aggressive carbon dioxide causes the calcium carbonate to dissolve to calcium bicarbonate:

$$\mathrm{CaCO_3 + CO_2 + H_2O \rightarrow Ca\ (HCO_3)_2}$$

Some of the hydrogen carbonate is washed out but some of the solution penetrates to combine with calcium hydroxide to give a final stage of the reaction where calcium carbonate is formed within the body of the material:

$$\mathrm{Ca\ (HCO_3)_2 + Ca(OH)_2 \rightarrow 2CaCO_3 + 2H_2O}$$

As a result, the pH of the pore fluid drops from a value greater than 12.6 to a value of about 8.0 (Hobbs, 1988; Parrot, 1987). At such low pH values the natural passivity no longer exists. If the cover zone of the concrete is of normal permeability and the depth of cover low, then the carbonation front can reach the level of reinforcement in a relatively short period of time and causes its corrosion.

In addition to a reduction in pH of the concrete, the carbonation changes the microstructure of the cement paste, resulting in a reduction in porosity of the exposed surface, and affects the strength and deformation properties (Parrot, 1987; Pihlajavaara, 1988). This is because the volume of calcium carbonate formed due to the carbonation exceeds that of the parent hydrate (Parrot, 1987).

The rate of carbonation is expressed normally as a function of square root of the time (Parrot, 1987; Beckett, 1986; Darwin, 1985). The equation takes the form:

$$d = A\sqrt{t}$$

where

d = depth of carbonation: mm

t = time: years

The factor A is a function of the material properties (such as permeability, porosity, carbon dioxide diffusivity, etc.) and the environmental conditions (such as concentration of carbon dioxide in the service environment, relative humidity, temperature, etc.) (Parrot, 1987; Darwin, 1985; Richardson, 1986). That is, the rate of carbonation depends on cement content, water/cement ratio, degree of curing and such factors which influence the porosity and permeability of concrete in addition to environmental factors.

Effect of chlorides

As mentioned earlier, chlorides may penetrate into concrete due to one of two processes: diffusion due to a concentration gradient and absorption in solution form (Raharinaivo *et al.*, 1986). The latter process is more relevant in the case of structures subjected to de-icing salts because as the ice melts, large quantities of water are absorbed into the concrete by capillary suction, carrying with it the chlorides. In the initial stages of diffusion of chlorides, the diffusion process is disturbed because the adsorption of chlorides on pore walls leads to the formation of calcium chloroaluminates. Over short periods of exposure (i.e. several months rather than years), absorption leads to considerably higher chloride levels than diffusion, although after longer periods of exposure, both processes tend to result in relatively similar concentration profiles.

Reactions between chloride ions and components of concrete have only very rarely been found to be the principal cause of reinforced concrete deterioration. The fundamental destructive effect of chlorides is their influence upon the reinforcement corrosion process, primarily due to their capacity to negate the corrosion-inhibiting properties of the alkaline cement paste pore solution (Newman, 1986). This risk increases with increasing concentration of free chlorides in the pore solution (Newman, 1986). It is generally believed (ACI Committee 318, 1989; BS 8110: 1989) that there is a threshold concentration of the chloride ions which must be exceeded before corrosion occurs. The type and quantity of cementitious material, water/cementitious material ratio, degree of compaction (air pockets near the reinforcement), degree of curing and the concentration of chloride ions in the exposure environment are known to affect the threshold concentration.

In addition to the influence of chlorides on the corrosion of steel, chloride solutions of high concentration can influence the cement matrix in the concrete (Kayyali, 1989; Plum, 1984). A mercury intrusion porosimetry study (Kayyali, 1989) has indicated that modification of the pore structure occurs as a result of chloride penetration. Therefore, it has been suggested (Kayyali, 1989) that in the case of prolonged exposure of concrete to chlorides, the capillaries in the hardened cement paste (HCP) may become discontinuous and there may be a reduction in the permeability.

Effect of leaching

As soft water from rain or from melting of snow and ice contains little calcium, the contact of such water with Portland cement paste would lead to the dissolution of calcium-containing products in hydrated Portland cement paste (HCP). This process, commonly called hydrolysis of cement paste, would stop once the contact solution attains chemical equilibrium (Mehta and Monteiro, 2005). However, in the case of flowing water, or seepage under pressure, as there is the dilution of the contact solution, the hydrolysis would continue. Due to the relatively high solubility of calcium hydroxide in pure water, it is this hydration product which is most subjected to hydrolysis.

The leaching of calcium hydroxide exposes other constituents of concrete to chemical decomposition and could leave behind silica and alumina gels with little or no strength. In addition, the leaching of calcium hydroxide could decrease the pH of the cement paste and result in the onset of corrosion of steel. As well as the loss of strength and the reduction in pH, leaching of calcium hydroxide may result in the precipitation of calcium carbonate on the surface when the leachate interacts with carbon dioxide present in air. This phenomenon is known as efflorescence.

Freeze–thaw deterioration and salt attack

The three mechanisms by which concrete deteriorates under the action of freezing and thawing (Cordon, 1966) are as follows:

1. Generation of hydraulic pressure due to freezing in capillaries.
2. Diffusion of gel water in capillaries followed by freezing.
3. Osmotic pressure resulting from partial freezing in capillaries of solutions with a local salt concentration.

However, the effect of osmotic pressure on the deterioration of concrete resulting from a salt concentration is generally associated with the ice removal agents. The above mechanisms are briefly described below in order to understand the importance of the pore structure of concrete.

Generation of hydraulic pressure in capillaries

When the mature concrete is wet, the capillary pores are either fully or partially saturated. Water freezes in these pores at temperatures below its freezing point, resulting in an increase in volume by 9% (Cordon, 1966; Adkins and Christianson, 1989). If the required volume is greater than the space available (as in the case of a fully saturated capillary) either the excess water is driven off by the pressure of expansion or dilation of the cavity occurs. If an escape boundary for the excess water is available, the volume of expansion due to freezing can be accommodated. Air voids in concrete ensure such an escape boundary and provide space for the water flowing out of the capillary pores. Therefore, within a certain radius of the air voids, the hydraulic pressure in capillaries is less than the tensile strength of the paste. The space enclosed by this radius may be thought of as a zone of protection for the paste. When the air voids are properly distributed, as in the case of *air-entrained* concrete, these zones overlap and there is no location where the hydraulic pressure can increase to the point of damaging the paste. Hence, the magnitude of the hydraulic pressure within the capillary depends on the degree of saturation, the rate of freezing, the permeability of the paste between the freezing cavity and the escape boundary, and the distance from the capillary to the escape boundary (Cordon, 1966).

If a capillary pore in the paste is sufficiently remote from an escape boundary, the pressure can be high enough to stress the surrounding paste beyond its tensile strength, and causes local cracking. In repeated cycles of freezing and thawing in a wet environment, water will enter these cracks during the thawing portion of the cycle only to freeze again later, and there will be progressive deterioration with each cycle.

Diffusion of gel water in capillaries followed by freezing

Generally, the more rigidly water is held, the lower will be the freezing point. Therefore, the water held rigidly by the calcium silicate hydrate (both interlayer and adsorbed in gel pores) in cement paste cannot freeze. It is estimated that water in gel pores does not freeze above -78°C (Cordon, 1966). Hence, ice in the capillaries of frozen cement paste is surrounded by unfrozen water in the gel pores, but in a supercooled state. This creates a thermodynamic disequilibrium between the frozen water in capillaries, which is in a low energy state, and the supercooled water in the gel pores, which acquires a high energy state. The gel water thus acquires an energy potential enabling it to move into the capillary cavity, where it can freeze. This fresh supply of water from the gel pores to the capillary pores increases the volume of ice in the capillary pores steadily until there is no room to accommodate more ice. Any subsequent tendency for the supercooled water to flow towards the ice-bearing regions would obviously cause internal pressures and expansion of the system.

When the concrete thaws or water enters from an external source through the cracks in concrete, the gel water may be replenished. This theoretically provides moisture for unlimited growth of ice crystals in capillaries, and eventual disintegration due to the hydraulic pressure.

Osmotic pressure in capillaries

Water in the capillaries contains several soluble substances, such as alkalis, chlorides and calcium hydroxide. Solutions freeze at lower temperatures than pure water; generally, the higher the concentration of a salt in a solution, the lower the freezing point. The concentration of salts in capillaries increases as the water begins to freeze. Therefore, the water forced from the capillaries has a higher concentration of salt than the surrounding gel pore water. This produces a high osmotic pressure, causing the water to move towards the coldest zone where freezing takes place. As a result, there will be progressive growth of ice in capillaries.

Freeze–thaw action on aggregates

If porous aggregates are used, such as those obtained from certain cherts, sandstones, limestones and shales, the cyclic freezing and thawing in the saturated state could disintegrate aggregates just as in the case of hydrated cement paste (Mehta and Monteiro, 2005; Cordon, 1966). The mechanism of deterioration in this case also is the same as that explained for the HCP. If aggregates of low permeability are used, the freeze–thaw damage of aggregate could be minimised.

Salt attack

The action of salt may be either physical or chemical. The physical action primarily consists of cracking due to the crystallisation of salts in concrete pores. The chemical action consists of either an attack on the cement (e.g. sulfate attack) or an oxidising action on the reinforcement in concrete (e.g. effect of de-icing salts), converting it to iron oxide. The sulfate attack is explained in the next section. As the de-icing salt is used primarily to melt the ice during winter, its physical action on concrete is associated with the freeze–thaw deterioration.

Effect of de-icing salts

The de-icing salts lower the freezing point and hence prevent the formation of ice. However, the ice removal agents contribute to the deterioration of concrete in several ways:

1. It will cause a substantial drop in temperature at the concrete surface during thawing of ice, known as thermal shock. Repeated thermal shocks may lead to cracking of the surface layer, which in turn increases the frost damage (Adkins and Christianson, 1989).
2. As snow and ice are melted by de-icing salts, the temperature immediately below the surface is reduced significantly because of the comparatively large heat of fusion. This may cause a damaging temperature drop in the saturated zone immediately below it. This subsurface layer can then form lenses of ice which cause subsurface failure and spall the surface off in sheets (Adkins and Christianson, 1989).
3. Placing salt on pavements for ice removal increases the concentration of salt in the capillary voids near the surface of the pavement. This will create a pore system which develops osmotic pressure, causing substantial accretion of ice and eventual disintegration due to the hydraulic pressure in capillaries (Cordon, 1966) by the mechanism explained under the Section ‘Generation of hydraulic pressure in capillaries’.
4. De-icers not only create additional forces through osmosis the but also provide additional moisture from the melting of ice and snow in freezing weather (Adkins and Christianson, 1989).
5. In addition to the above problems with the de-icing salts, the crystallisation of salt in pores may lead to cracking near to the surface of the concrete (Bjegovic *et al.*, 1987).

Worse than all these, de-icing salt is an important source of chlorides increasing largely the possibility of a fast rate of corrosion of the reinforcement (Woods, 1968).

Chemical attack

The effect of some of the most common chemicals on the deterioration of concrete was summarised by the ACI Committee 201. In general, the chemical attack on concrete may be classified into:

- acid attack
- alkali attack
- carbonation
- chloride attack
- leaching
- salt attack
- sulfate attack
- delayed ettringite formation
- thaumasite attack.

In the majority of cases, it is the chemical interactions between the aggressive agents present in the external environment and the constituents of the cement paste that decide the extent of the deterioration of concrete. An exception to this is the effect of salt used as de-icing agents, in which case the effect is both physical and chemical. As carbonation, chloride attack, leaching and salt attack have already been discussed; the mechanisms of sulfate attack, delayed ettringite formation, thaumasite attack, acid attack and alkali attack are briefly described below.

Sulfate attack

The naturally occurring sulfates – mainly calcium, magnesium, sodium and potassium – sometimes found in soil or as solutions in groundwater, can attack concrete. Solid salts do not attack concrete, but when present in solution they can react with two hydration products. The first of these is calcium hydroxide, which the sulfate converts into a large volume of calcium sulfate (gypsum) (Pye and

Hudson, 1986; Thomas and Skalny, 2006). The second hydration product which reacts with the sulfate is tricalcium aluminate hydrate. Under normal conditions, tricalcium aluminate merely contributes to the hydraulic bond, but when in contact with sulfate solutions it reacts to form a sulpho-aluminate hydrate, which has a greater volume than that of the original compound. If enough of these reactants – tricalcium aluminate hydrate, sulfate and water – are present in the concrete, the resultant internal expansive stress may be great enough to cause deformation, cracking and eventually loss of cohesion.

Both calcium sulfate and sodium sulfate attack only calcium aluminate hydrate, forming calcium sulpho-aluminate (ettringite). On the other hand, magnesium and ammonium sulfates can proceed to attack the calcium silicate hydrates as well (Mehta and Monteiro, 2005; Thomas and Skalny, 2006; Pye and Hudson, 1986). Therefore, magnesium sulfate attack is more destructive to concrete than other forms of sulfate attack.

The effect of sulfate on concrete can be divided into two main types: chemical and physical, although in practice they are closely related. The chemical action results in changes in the microstructure and pore size distribution of the cement paste as a result of the chemical reactions explained earlier. It has been reported (Pye and Hudson, 1986; Thomas and Skalny, 2006) that in the early stages of penetration of sulfates and the chemical reactions, the products cause the concrete to gain strength as unoccupied pore spaces are filled. However, disruption and final disintegration of the concrete follow as the reaction continues. Hence, sulfate attack can take the form of a progressive loss of strength and loss of mass due to deterioration in the cohesiveness of the hydration products.

The chemical changes may also lead to the expansion of concrete. When concrete cracks, its permeability increases, which accelerates the penetration of aggressive waters into the interior of concrete, resulting in an acceleration of the deterioration (Mehta and Monteiro, 2005). In addition to the chemical reactions, ACI Committee 201 cites evidence that a purely physical action (not involving the cement) of crystallisation of the sulfate salts in pores of the concrete can account for considerable damage.

Delayed ettringite formation

When calcium sulpho-aluminate (ettringite) is formed homogeneously and immediately after mixing concrete, i.e. when the concrete is in its early stages of hydration, the expansion caused by the conversion of calcium aluminate to calcium sulpho-aluminate (ettringite) does not cause any significant localised damage. Collepardi (2006) states that this situation arises when gypsum added as a set regulator reacts with calcium aluminates of the Portland clinker phase or due to chemical reactions in expansive cements used in shrinkage-compensating concretes. However, if ettringite forms heterogeneously at later ages (months or years) when concrete has attained dimensional stability, the localised expansion due to the formation of ettringite could cause concrete to crack and spall, leading to strength loss. This is termed delayed ettringite formation or DEF. The DEF occurs due to the late release of sulfates from gypsum-contaminated aggregates or sulphur-rich clinker phase (from supplementary cementitious materials used in the manufacture of clinker). While sulfate attack occurs only when external sulfates enter concrete, DEF takes place with no ingress of sulfates from the exposure environment. That is, sulfate attack could be controlled by providing a low permeability concrete, but DEF does not depend on the permeability of concrete. One of the commonly reported causes of DEF is elevated curing of precast products and hence restrictions are normally imposed on the highest curing temperature (somewhere below 60°C) (Thomas and Skalny, 2006).

Thaumasite attack

At temperatures between 0 and 5°C and relative humidity greater than 95%, chemical reactions between cement hydration products, sulfates and carbonates may result in the formation of calcium-silicate-sulfate-carbonate, or more commonly known as thaumasite. Normally, thaumasite is formed when external sulfate ions penetrate into concrete and carbonate ions are present either due to carbonation reaction or the presence of carbonate aggregate in concrete (Thomas and Skalny, 2006). As the chemical reaction involves both calcium hydroxide and calcium silicate minerals in hydrated cement paste, the formation of thaumasite leads to the complete destruction of the structure of concrete.

Acid attack

Acid conditions are obviously harmful to the alkaline concrete, especially where the acidic solutions or groundwaters are able to flow over the concrete surface (Woods, 1968). The severity of the attack depends on the type of acid. The action of acid on concrete is to dissolve away the cement and, in the case of limestone aggregate, the aggregate too. Gravel aggregate concrete subjected to acid attack has an 'exposed aggregate' appearance and is steadily weakened from the surface as the acid penetrates.

In most cases of acid attack, the chemical reaction results in the formation of water-soluble calcium compounds which are then leached away by the aqueous solutions. This results in an increase of the porosity and the permeability of the system. However, oxalic and phosphoric acids are exceptions, because the resulting calcium salts are insoluble in water and are not readily removed from the concrete surface (Thomas and Skalny, 2006).

In the case of sulphuric acid attack, additional or accelerated deterioration results because the calcium sulfate formed will affect concrete by the sulfate attack mechanism, as described in the section on sulfate attack. That is, the reaction between sulphuric acid and calcium hydroxide produces calcium sulfate, which is deposited as gypsum:

$$H_2SO_4 + Ca(OH)_2 \rightarrow CaSO_4 . 2H_2O$$

The calcium sulfate formed by the initial reaction may proceed to react with the calcium aluminate phase in cement to form calcium sulpho-aluminate (ettringite), which on crystallisation can cause expansive disruption of the concrete:

$$3CaSO_4 + 3CaO . Al_2O_3 . 6H_2O + 25H_2O \rightarrow$$
$$3CaO . Al_2O_3 . 3CaSO_4 . 31H_2O$$

The effect of acid attack is initially to decrease the pH, but as the attack proceeds, all the cement compounds are eventually broken down and leached away, together with any carbonate aggregate material, thus increasing both porosity and permeability. If acids or salt solutions are able to reach the reinforcing steel through cracks or pores in the concrete, corrosion of the steel can occur which will in turn cause cracking and spalling of the concrete.

Alkali attack

Portland cement concrete made with non-alkali reactive aggregates is highly resistant to strong solutions of most bases. It is unaffected by continuous exposure to 10% solutions of sodium or potassium hydroxides (Woods, 1968). However, if circumstances are such that sodium hydroxide (NaOH) can penetrate concrete and become concentrated at an evaporating face, physical damage may result from crystallisation of sodium carbonate formed by reaction between the NaOH and carbon dioxide from the air. Alternate wetting and drying of concrete by NaOH solution may bring about the same effect. These effects can be minimised by the use of concrete of high quality – that is, low permeability.

Alkali–aggregate reaction (AAR)

Three forms of alkali–aggregate reaction have been identified (Harrison, 1987; Hobbs, 1986; Somerville, 1985; Institution of Civil Engineers, 1988; Swamy and Al-asali, 1990), viz. alkali–silica reaction (ASR), alkali–carbonate reaction and alkali–silicate reaction. Among them, alkali–silica reaction is the most common form, which is described in this section. When the attack is on certain types of limestone aggregates it is referred to as alkali–carbonate reaction and if the chemical reaction involves layers of silicate minerals, it is known as alkali–silicate reaction.

Alkali–silica reaction in concrete is a reaction between the hydroxyl ions in the pore water of the concrete, which normally results from the sodium and potassium alkalis released during the hydration of Portland cement, and certain siliceous constituents in the aggregates (Harrison, 1987; Hobbs, 1986). As a result an alkali–silica gel is formed, and alterations of the surfaces of the aggregate particles take place. The gel imbibes pore fluid, which causes its expansion. Where the conditions are favourable for the production of sufficient gel and its subsequent expansion, internal stresses are induced within the concrete. These stresses may be occasionally large enough to cause cracking and expansion of concrete. The gel will cause damage to the concrete only when the following three conditions occur simultaneously (Hobbs, 1986):

1. A reactive form of silica is present in the aggregate in significant quantities.
2. Sodium, potassium and hydroxyl ions are present in sufficient quantity in the pore solution within concrete.
3. Sufficient quantity of water is available in concrete.

It is possible for the reaction to take place in concrete without inducing internal stresses and expansion. Damage may not occur, even when the reaction is quite extensive, and the gel may fill cracks induced by some other mechanism (Harrison, 1987).

The physical effect of ASR may be local swelling, expansion, cracking or changes in physical properties (Somerville, 1985). The most direct visual evidence of the progressive chemical reaction within the concrete due to ASR is extensive cracking. This may be due to either localised swelling of reactive aggregates or non-uniform expansion throughout the volume of concrete as a result of the ASR. Apart from cracking, the major effect of ASR is a reduction in compressive strength, tensile strength, modulus of elasticity and density (Institution of Structural Engineers, 1988; Swamy and Al-asali, 1990). The microcracking due to ASR expansion is considered to be the main reason for the change in these physical properties.

Abrasion resistance of concrete surfaces

Abrasion resistance of concrete is defined by ACI Committee 116 (2000) as the ‘ability of a surface to resist being worn away by rubbing and friction’. However, different definitions can be seen in the literature to describe this phenomenon (Mehta and Monteiro, 2005). There are instances where abrasion is of little concern structurally, yet there may be a dusting problem which can be quite objectionable in some kinds of service. The abrasion resistance of concrete is a progressive phenomenon. Initially, resistance is closely related to the strength at the wearing surface. As the paste wears, the fine and coarse aggregates are exposed. Thereafter, the abrasion and impact will cause additional degradation that is related to the aggregate-to-paste bond strength and hardness of the aggregates. The abrasion resistance of a concrete surface is commonly accepted to be influenced by the physical properties of aggregates, water/binder ratio, admixtures, finishing techniques, curing and surface treatments.

If the aggregate in concrete is wear resistant, it is the properties of the cement matrix that control the abrasion resistance (Mehta and Monteiro, 2005). This in turn depends on the porosity of the cement paste and, hence, there is a correlation between the abrasion resistance and the permeability of concrete.

Cracking in reinforced concrete

Cracking indicates the presence of disruptive forces within a mass of material which exceed its tensile strength. These forces may be caused by the application of external load or by internal changes in the material, and, in the case of concrete, they may occur due to either of these causes or by a combination of the two while the concrete is either in the prehardened or hardened state (Richardson, 1987). In order to identify the causes of cracking, cracks formed in the hardened state are generally separated from those formed in the prehardened state (Richardson, 1987; ACI Committee 224, 1985; Caldwell, 1984; Orr, 1971). The various types in each category can be represented by a comprehensive family 'tree of cracks' (Price, 1982) as shown in **Table 2**.

The type, location and extent of cracks could form the basis of identifying the cause of deterioration while carrying out structural investigations. This aspect has been dealt with comprehensively by Bungey *et al*. (2006). It was observed in the previous discussion on mechanisms of deterioration that cracking accelerates the penetration of aggressive substances into the concrete, which in turn aggravates any one or a number of other mechanisms of deterioration.

Crack propagation state	Type of cracks (cause of crack formation)
Pre-hardening state	The early frost damage
	Plastic settlement
	Plastic shrinkage
	Formwork movement
	Subgrade movement
	Constructional movement
Hardened state	Physical
	Drying shrinkage
	Shrinkable aggregate
	Crazing
	Chemical
	Alkali–aggregate reaction
	Corrosion of reinforcement
	Carbonation
	Thermal
	External seasonal temperature variations
	Freeze–thaw
	Early thermal contraction
	Structural
	Accidental overload
	Creep
	Design loads
	Foundation movements

Table 2 Cracks in reinforced concrete structures

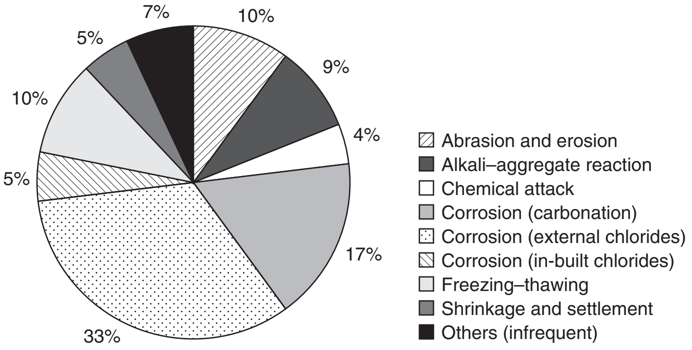

Figure 2 Factors causing deterioration of reinforced concrete structures

Factors influencing durability of concrete

A survey of literature covering 271 cases of deterioration by the authors has shown that 50% of the reported cases of deterioration were caused by corrosion due to either chloride penetration or carbonation (**Figure 2**). Factors which contributed to the deterioration of the cases are summarised in **Figure 3**.

The mechanisms of deterioration and their rate are controlled by the environment, the paste microstructure and the fracture strength of concrete. Environmental factors such as seasonal temperature variations, cycling freezing and thawing, rainfall and relative humidity changes and concentration of deleterious chemicals in the atmosphere/water in contact with the concrete are the main causes of degradation. However, the single most important parameter that leads to premature deterioration is the

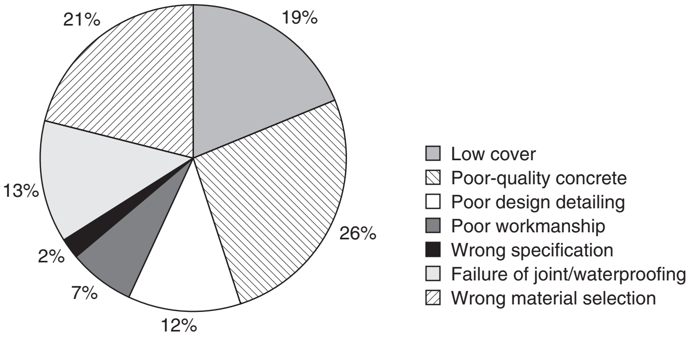

Figure 3 Factors influencing durability of reinforced concrete structures

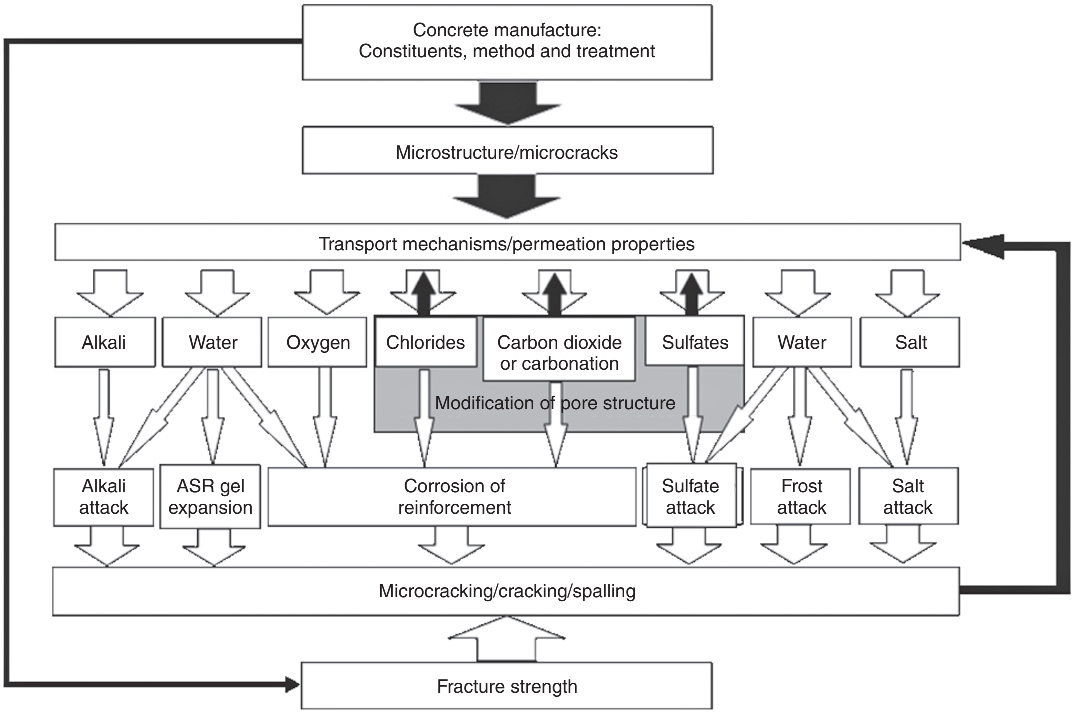

Figure 4 Causes of deterioration–physical properties interaction model (adapted from Long *et al*., 2001)

ingress of moisture into the concrete. The importance of permeability/transport properties to the various deterioration mechanisms and the relationship between the deterioration mechanisms and the properties of concrete have been demonstrated while discussing the mechanisms of deterioration. **Figure 4** illustrates the interactive nature of the influence of material characteristics (both strength and permeability) on durability of concrete (Long *et al*., 2001).

Test methods for assessing the durability of concrete

Numerous tests are available for the laboratory and field assessment of the durability of concrete and concrete structures (Bungey *et al*., 2006; Lamond and Pielert, 2006). The choice of tests very much depends on the mechanism being considered and the nature of investigation. These tests can be grouped under three categories:

1. Tests which measure properties of concrete related to durability.
2. Tests to assess the cause and rate of deterioration.
3. Integrity tests.

If the investigation is to assess the potential durability of either concrete or concrete-making materials, standard laboratory-based tests are normally carried out under controlled environmental conditions (Ramachandran and Beaudoin, 2000). However, if the purpose is to identify the nature and extent of deterioration of concrete structure, it is most likely that more than one type of testing will be required. This is because there are similarities between symptoms of many types of deterioration. Therefore, in many cases where cracking or other defects on structures are being investigated, there will be a need for some quite devious forms of laboratory and in situ testing, not only to establish the cause of the problem, but more importantly, to ensure that the correct decision regarding future action is taken.

The above two types of investigation for determining the durability of concrete are thoroughly dealt with in the references Ramachandran and Beaudoin, 2000 and Malhotra and Carino, 2004 respectively. In addition, for field investigation of the durability of structures, Bungey *et al*. (2006) may also be consulted.

Methods of ensuring the durability of concrete

There are several methods of ensuring the durability of concrete. The most important is to ensure that the concrete is of high quality, with low permeability and sufficient cover to protect the reinforcing steel. **Table 3** summarises the various methods of improving the durability of concrete (Broomfield, 2004).

In order to ensure durability against carbonation-induced corrosion of reinforcement, it is essential to provide

Method	How it works	Advantages	Limitations	Where used
Increase cover over reinforcement	Increases time for chlorides or carbonation to reach steel	Low cost, included in codes	Requires very good quality assurance, increased risk of cracking giving Cl^- and CO_2 more rapid ingress	Everywhere
Decrease water/binder ratio	Reduces concrete porosity to increase time for corrodents to reach steel	Low cost, in codes	Makes concrete less workable and more prone to poor consolidation; higher cement content increases heat of hydration. Plasticisers add to cost	Everywhere
Ground granulated blast furnace slag, microsilica, fly ash	Reduces concrete porosity to increase time for corrodents to reach steel	Moderate cost, increasingly in codes to achieve durable concrete in marine exposure, lower cement content decreases heat of hydration	Makes concrete less workable and more prone to poor consolidation; costs and availability can be problematic	Marine structures and other severe exposure
Corrosion inhibitors	Protective chemical layer on steel	Modest cost, can be used with other techniques	Needs to work after 20 years or more; may be washed out in marine environment. Dosage must reflect exposure	Marine structures, car parks and other severe exposure
Coatings and penetrating sealers	Increases time for chlorides or carbonation to reach steel	Low cost, easy maintenance	Changes appearance, requires maintenance. Application of sealers weather dependent	Bridges, car parks
Membranes and barriers	Increases time for chlorides or carbonation to reach steel	Ideal for bridge, car park decks etc.	Requires maintenance; can hide problems. Changes profile and dead load	Bridge and car park decks
Fibre-reinforced polymer	Corrosion resistant	Theoretically very long life. Easy handling; may reduce concrete specification	Needs new design codes; unknown deterioration rates and mechanisms over 50-year life or more	Experimental
Stainless steel reinforcement	Corrosion resistant	If correct grade is used, indefinite life. Scrap value of solid bar	High first cost. Clad bar production QA/QC still in development	High-value, long life structures or elements
Cathodic prevention	Makes steel cathodic and therefore corrosion resistant	Theoretically up to 100 years life of anodes. Codes available	High first cost, high maintenance, technically complex	High value high chloride exposed structures
Epoxy-coated reinforcement	Coating prevents chloride access to steel surface	Easy, low cost	Durability problems in some highway structures in Florida and Ontario. Handling damage during transportation and placement	Bridge decks, etc. in North America and Arabian Gulf area
Galvanised reinforcement	Higher chloride tolerance and resistance to carbonation delays corrosion; sacrificial protection	Widely available; design, handle and use as black steel	Advantages reduce in poor concrete and high chloride. Risk of accelerating corrosion if coupled to black steel	Everywhere

Table 3 Methods of improving the durability (adapted from Broomfield, 2004)

a concrete cover of sufficient thickness and of adequate quality. If achievement of the required level of cover cannot be guaranteed, or if an exceptionally long service life is required, then consideration should be given to other solutions, such as anti-carbonation coatings, corrosion resistant reinforcement, or corrosion inhibitors.

In the case of chloride ingress, if possible the element should be designed and detailed in such a way as to minimise the severity of the exposure. In bridges, for example, consideration should be given to integral or continuous construction in place of jointed construction. Good drainage should be detailed to prevent build-up of water and de-icing salts and to avoid de-icing salts draining to the foundations. Additional measures should be considered to provide enhanced performance of the concrete cover, such as silane impregnation, increased thickness, the use of controlled permeability formwork, or improved protection of the reinforcement against corrosion such as inhibitors, cathodic protection or corrosion-resistant reinforcement. The Highways Agency states that consideration should be given to all possible means of reducing or eliminating the use of corrodible reinforcement (Taylor *et al.*, 2007). This includes the use of plain (mass) concrete for abutments, wing walls and retaining walls, and the use of arch structures where ground conditions permit.

The strategy of reducing the risk of corrosion of reinforcement by use of concrete as protection relies on the provision of a sufficient thickness of concrete of adequate quality. Careful specification and supply of concrete from a quality controlled source should ensure the adequacy of the materials. Good workmanship is essential in placement, compaction and curing to ensure the full potential of the concrete is achieved in the structure. Good-quality concrete will, however, fail to provide the required durability if it is not there in a sufficiently thick layer.

Resistance to freezing and thawing is traditionally provided either by using concrete of sufficient compressive strength or by the use of air-entrainment. The use of grade C50 non-air-entrained concrete has generally proven satisfactory against the winter climate experienced in the UK; such concrete might, however, not provide sufficient resistance against more severe freezing and thawing conditions. In design practice, the structural form should be chosen, as far as possible, such that water saturation is avoided. Wherever possible, horizontal surfaces on which water is likely to sit should be avoided by using a sloped surface to allow the water to run off and lessen the likelihood of saturation. Poor drainage which allows water to flow down vertical surfaces should be avoided for the same reasons.

The traditional approach to avoid excessive abrasion is to specify concrete of adequate compressive strength made with hard-wearing aggregates. Some of the other options are: (i) use of concrete of adequate quality to resist for required life, or sacrificial layer as a protection; (ii) use of inhibiting agents for surface protection; or (iii) arrange for planned replacement (e.g. wearing surface of roads).

To summarise, it can be said that to improve the conventional service life of concrete structures, there is a need to monitor critical parameters during construction to ensure that they achieve the desired concrete properties which are indicative of long-term durability, such as proper air void structure, low permeability, minimum water content, and low susceptibility to cracking. Equally important is the deployment of innovative technologies that evaluate the in situ properties of concrete quickly and accurately.

Specifications for durable concrete

Concrete will be durable; that is it will serve its purpose for the intended service life with only tolerable need for maintenance and repair, if the specifications are correct and followed strictly. In BS EN 206-1: 2000, durability of concrete relies on prescriptive specification of minimum grade, minimum binder content and maximum water/binder ratio for a series of defined environmental classes. However, in the development of this standard, it has not proven possible within the European Committee for Standardization (CEN) to agree common values for the specification parameters to cover the wide range of climates and cements in use in the EU Member States. Therefore, the standard includes indicative values, and it is left to individual Member States to specify national values where they require them to differ from the indicated values. The Standard expects designers to define their required characteristics in terms of minimum strength class, maximum water/binder ratio and minimum binder content. Consistence specification is left to the judgement of the contractor/user so long as the above parameters are not compromised.

The informative examples of the exposure classes as per BS EN 206-1: 2000 are given in **Table 4**. In total there are 23 exposure classes, spilt into six groups covering:

1. no risk of corrosion
2. carbonation-induced corrosion
3. chloride-induced corrosion resulting primarily from de-icing salts
4. chloride-induced corrosion resulting from seawater exposure
5. freeze–thaw attack
6. chemical attack.

The exposure classes to be selected depend on the provisions valid in the place of use of the concrete. Furthermore, the concrete may be subject to more than one of the actions described in **Table 4** and the environmental conditions to which it is subjected may thus need to be expressed as a combination of the exposure classes.

The staff of the British Cement Association and Building Research Establishment have reviewed the UK literature and, where required, the international literature to establish the minimum specifications for concrete necessary to achieve durable concrete in the exposure conditions defined by the BS EN 206-1: 2000 for the broader range of binders used in the UK (Hobbs, 1998). The minimum requirements for concrete to resist different exposure regimes were summarised in this document, which could be used as a guide while designing concrete mixes for the different exposure classes in BS EN 206-1: 2000.

Service life design of concrete

A structure is considered durable when it performs satisfactorily and maintains acceptable appearance as long as the user needs the structure. The operational way of designing for durability is to define it as a service life requirement. The service life is defined as 'the time during which a concrete fulfils its performance requirements', without non-intended maintenance. The definition of service life is illustrated in **Figure 5**. The service life design (SLD) can therefore be defined as 'the process of designing a structure for durability over a given minimum life in a particular environment'. The essential elements of SLD are:

- a design process
- a minimum performance requirement (durability)
- a specified minimum life
- characterisation of the exposure environment.

Class	Environment	Examples
1. No risk of corrosion or attack		
X0	Concrete with no embedded metal (except where there is freeze–thaw, abrasion or chemical attack) For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2. Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3. Corrosion induced by chlorides other than from seawater		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools, concrete exposed to industrial water containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements, car park slabs
4. Corrosion induced by chlorides from seawater		
XS1	Exposed to airborne salt but not in direct contact with seawater	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spry zones	Parts of marine structures
5. Freeze–thaw attack with or without de-icing agents		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agent	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation, with de-icing agent or seawater	Road and bridge decks exposed to direct spray containing de-icing agents and freezing. Splash zones of marine structures exposed to freezing
6. Chemical attack		
Where concrete is exposed to chemical attack from natural soils and groundwater as given in Table 2 of BS EN 206-1: 2000, the exposure shall be classified as given below. The classification of seawater depends on the geographical location; therefore the classification is valid in the place of use of the concrete		
XA1	Slightly aggressive chemical environment according to Table 2 of BS EN 206-1: 2000	
XA2	Moderately aggressive chemical environment according to Table 2 of BS EN 206-1: 2000	
XA3	Highly aggressive chemical environment according to Table 2 of BS EN 206-1: 2000	

Table 4 Exposure classes from BS EN 206-1: 2000

SLD may not be justified for simple structures, those with a long experience (e.g. traditional housing), or those with a short required service life (e.g. exhibitions). However, SLD may be needed:

- where the required life is significantly greater than ‘normal’ for that type of structure;
- where the exposure environment is particularly severe;
- where there is a particularly low risk of failure, e.g. nuclear installations. Codes are usually based on the assumption of a ‘reasonable’ probability of success with the need for some minor repair;
- for materials without a long track record of performance but whose properties can be characterised or predicted; or
- when considering whole-life-costs for the structure.

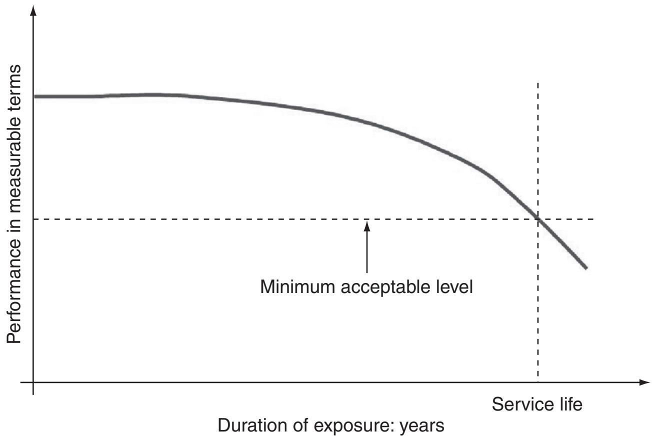

Figure 5 Definition of service life

The possible advantages of SLD are as follows:

- economy through possible avoidance of over-design;
- facilitates selection of an appropriate technical option for resisting particular degradation factors;
- the chances of successfully achieving the required performance over the whole required life should be increased.

SLD is based on predictions of future deterioration. In order to design for service life the following information must be available:

- performance requirements
- environmental conditions
- deterioration mechanisms
- predictive methods.

More information on SLD can be found in Fagerlund (1985), Sjostrom (1985), Clifton (1990), Frohnsdorff and Masters (1990), Singh (1991), Somerville (1992), Khatri and Sirivivatnanon (2004).

Concluding remarks

Among various deterioration mechanisms likely to shorten the service life of reinforced concrete structures, corrosion of rebar has been identified as the most common form of defect. With the introduction of the new standards for classifying exposure regimes, it has been possible to suggest concrete mixes which could resist the various exposure conditions. A combination of laboratory and field testing may be required in order to identify the nature and extent of deterioration of structures, but if the investigation is carried out to determine the potential durability of either concrete or concrete-making materials, standard laboratory methods may be used. Tools are now available to design concrete structures for an expected service life.

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CONTENTS

Chapter 17

Non-destructive testing of concrete structures

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Numerous test techniques are available at present to assess properties of the near-surface concrete in situ. These tests can collectively be grouped under: (i) tests for assessing the strength; (ii) tests for assessing the durability or related properties; and (iii) those for measuring the integrity of the concrete. Guidance for the selection of appropriate test methods for different testing situations can be obtained from a report by the American Concrete Institute Committee 228 (ACI, 2008) on Non-destructive Testing. Some of these tests are now part of various national and international standards, but compared to the number of tests that can be identified in the literature, there are fewer standards on non-destructive tests due to the difficulties of interpreting in situ data obtained with non-destructive testing of concrete structures. The intention of this chapter is to introduce various non-destructive tests and provide an insight into their application for the assessment of various properties of concrete, in addition to highlighting how non-destructive tests can be used to assess the cause and the extent of deterioration of concrete structures.

Introduction

The performance history of concrete, like many other materials, can be described diagrammatically as in **Figure 1**. Generally the performance deteriorates gradually with time (curve 1) and not suddenly (and catastrophically) as indicated by curve 2. The performance depicted in curve 1 (negligible deterioration) may not be achievable and in any case will require high initial expenditure. Most concrete, therefore, follows a durability performance pattern similar to curve 3, in which there is significant deterioration with time but intermittent maintenance changes the performance level or alters the rate of change of the performance. Therefore, it is essential to determine the performance of the structure in measurable terms at the time of completion of the structure as well as during its service life.

On the basis of the mechanism of action, the deterioration of concrete is due to one (or a combination) of: abrasion/erosion, alkali–aggregate reaction, chemical attack (such as: acid attack, alkali attack, carbonation, chloride attack, leaching, salt attack, sulfate attack, thaumasite attack or delayed ettringite formation), corrosion of reinforcement, freeze–thaw and frost damage and cracking. There are similarities between symptoms of different types of deterioration. Therefore, in many cases where cracking or other defects on structures are being investigated, there will be a need to use ingenious forms of laboratory and in situ testing, not only to establish the cause of the problem, but more importantly, to ensure that the correct decision regarding future action is taken. Therefore, the objectives of a site investigation are (Bungey *et al*., 2006):

1 to investigate the current condition of the structure
2 to diagnose the causes of defects or deterioration
3 to select an appropriate solution to the problem.

In situ testing is carried out for other purposes as well. For instance, it could be used for quality testing, compliance testing and secondary testing, as highlighted below (Bungey *et al*., 2006).

- Control testing: quality assurance – by the contractor or concrete producer.
- Compliance testing: to test the compliance with the specification – by or for the engineer.
- Secondary testing: in situ testing or laboratory testing on samples extracted from the structure, not necessarily be part of tests planned before construction.

In situ testing is expensive and, therefore, it is essential that the aims of the investigation are established right from the start. The engineer must be aware of all of the tests available before a test programme is undertaken. The choice of the test method, the number of tests to be performed and the order in which they are performed depend on the aims of the investigation. Therefore, before any testing begins, the development of a properly structured plan is essential, with interpretation as an ongoing activity. **Figure 2** shows a typical suitable approach. Cost escalates as the investigation proceeds; however, the programme needs only be taken as far as necessary to reach firm relevant conclusions.

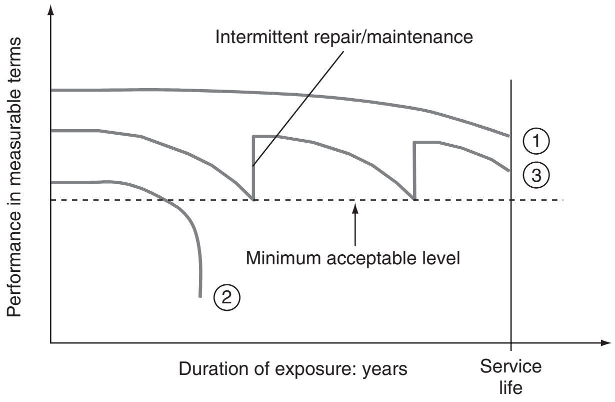

Figure 1 Change of performance of structures with time

The extent of detailed testing may depend on whether litigation is involved and if the problem is one of structural distress or of deterioration. Litigation will generally require considerably more statistical evidence of deterioration or failure, in order to prove that a problem exists and to indicate fault. The legal team may also require proposed solutions, to address costs which may be involved in correcting or repairing the failed or deteriorated structure. Such requirements should be clearly identified and stated by the legal representatives at the outset.

If the problem is one of overloading or excessive deflections, the testing will concentrate on the strength of the concrete and on determining the reinforcement type, size and content. If deterioration has occurred, both the physical and chemical properties of the concrete in both the deteriorated and non-deteriorated locations will be obtained. The type of remedial treatment envisaged also could influence the investigation.

The lack of durability of concrete structures manifests in the form of cracking, spalling, loss of strength and loss of mass. Quite often concrete in an advanced state of deterioration suffers from more than one cause and it is extremely difficult to identify the first cause that led to the deterioration of the structure. To assist with the determination of the cause(s) and the extent of deterioration, numerous non-destructive methods have been developed, some of which are currently recommended as standard techniques for assisting in site inspection and evaluation of concrete structures. In all cases, a sound knowledge of the various types of cracks will be invaluable because the first step in any investigation is a visual inspection (**Figure 2**). This is normally followed by non-destructive testing, assisted with confirmatory investigations using in situ sampling and laboratory testing.

The issue with most structures is not if maintenance is required, but when – and when to schedule it most cost-effectively. In **Figure 3** it is illustrated that during the initiation phase of deterioration, there will be material changes (such as carbonation, chloride ingress, sulfate ingress, etc.), which on reaching a threshold activate the deterioration process. This follows the active stage of deterioration. Therefore, frequent planned in situ testing of the material

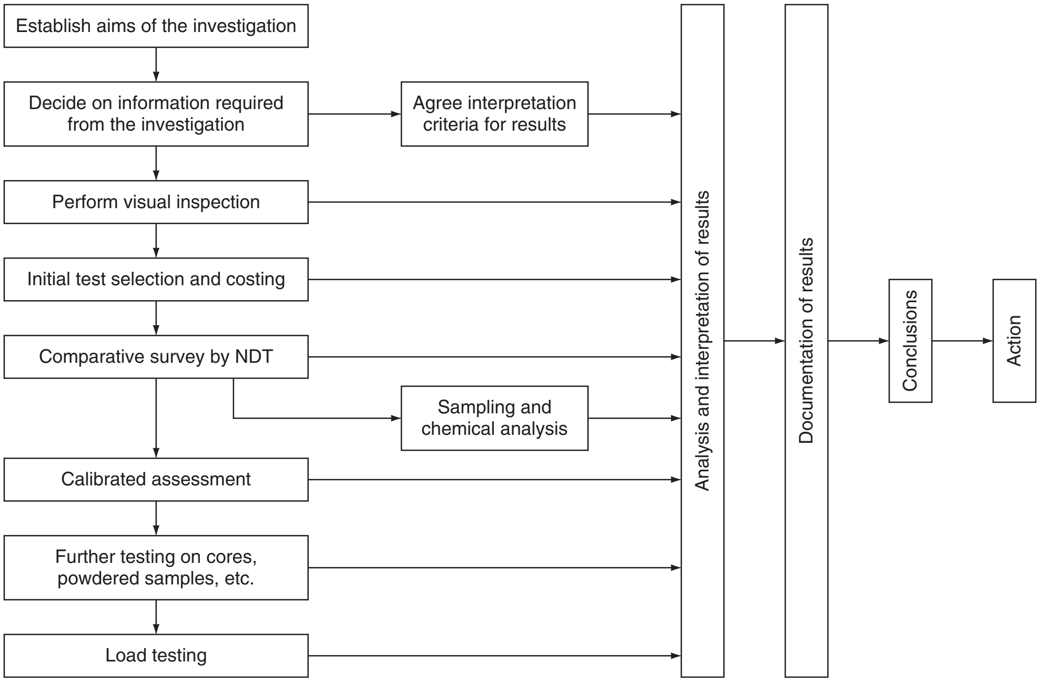

Figure 2 Different stages in a properly planned testing programme (adapted from Bungey *et al*., 2006)

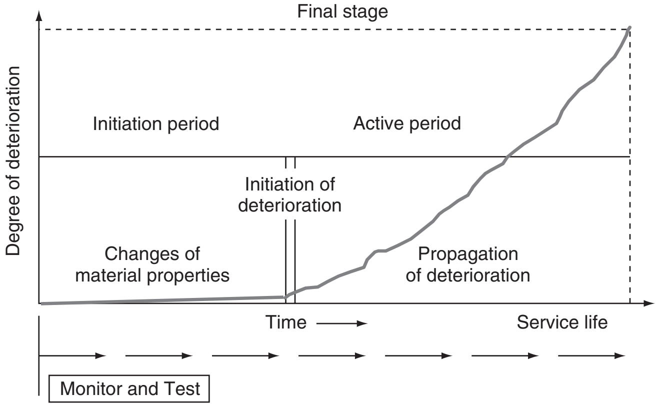

Figure 3 Illustration of the need for frequent in situ testing of structures during their service life

changes is useful to assess the initiation period. During the propagation of deterioration, the extent of the deterioration can be assessed using in situ test techniques which can quantify the extent of deterioration.

Non-destructive test methods

As shown in **Figure 2**, visual inspection is normally one of the first steps in any site investigation. Visual inspection can provide a qualified engineer with valuable information as to the cause and extent of deterioration because almost all forms of deterioration mechanisms eventually lead to the cracking of concrete. The ability to differentiate between different types of cracking is essential in identifying the cause of distress. The UK Concrete Society has illustrated different types of non-structural cracks in a hypothetical structure (Concrete Society, 1982), which can form the basis to identify the cause(s) of the distress in a structure. However, it must be remembered that, despite the fact that each form of deterioration has a definite crack pattern, some of the crack forms are similar and, hence, it will often be difficult to determine the cause by visual inspection alone. However, the results from the visual inspection are essential for the selection of further identification tests.

During an investigation, many tests may be used to provide information for the condition assessment and the prediction of future performance. The test methods can be classified as:

> Destructive tests (DT): conventional methods in which the extraction of samples from a structure is involved.
> Non-destructive tests (NDT): do not impair the intended performance of the element or member under test.
> Partially-destructive tests (PDT): cause localised surface zone damage.

Some tests provide the opportunity to measure the desired property directly and some others give a direct quantitative measurement of the desired property from correlations. A combination of factors, such as access, cost, speed and reliability, influence the test selection. In this section, various test methods are briefly reviewed and their application highlighted.

Assessment of strength

It is generally recognised that due to variability in concrete mixing, compaction and curing conditions, the actual strength of concrete in a structure cannot be the same as strength of controlled test cubes casted at the time of construction. Moreover, in some cases problems in concrete are not detected until it is too late as standard testing is generally carried out at 7 and 28 days. It is these limitations, difficult to assess by conventional strength tests, which gave impetus to the development of non-destructive methods for strength assessment of concrete in situ. A wide range of test techniques is available for estimating the in situ strength of concrete. This includes non-destructive and partially destructive techniques that are based on measuring physical parameters in situ and empirically relating it to the strength properties. Some of the commonly used non-destructive test methods to assess in situ strength of concrete structures are summarised in **Table 1** and briefly elaborated below.

Test method	Intended purpose	Classification	Reference
Rebound hammer test	Surface hardness of concrete	NDT	BS EN 12504-2: 2001; ASTM C805
Ultrasonic pulse velocity	Non-uniformity and density of concrete	NDT	BS EN 12504-4: 2004; ASTM C597
Pull-out tests: Capo and Lok test	Pull-out strength of concrete cover	PDT	BS EN 12504-3: 2005; ASTM C900; Bungey and Soutsos, 2001
Pull-off test	Pull-off surface tensile strength of concrete	PDT	BS EN 1542: 1999; ASTM C1583; Bungey and Soutsos, 2001
Windsor probe penetration resistance test	Probe penetration resistance of concrete cover	PDT	BS 1881: Part 207; Bungey and Soutsos, 2001
Break-off test	Break-off strength of concrete cover	PDT	BS 1881: Part 207; Bungey and Soutsos, 2001
Maturity measurement	Strength development	NDT	ASTM C1074-02; Malhotra and Carino, 2004

Table 1 Tests for assessing strength

Surface hardness test

Concrete surface hardness test gives a measure of the surface hardness of concrete (about 30 mm deep). The rebound hammer method is easy to use and has the ability to take a great number of readings at little expense. The crushing strength obtained for well-controlled cubes or cylinders and the corresponding rebound values from the hammer test are reported to have a high degree of correlation under controlled conditions (Bungey *et al.*, 2006; Malhotra and Carino, 2004). Similar empirical relationships have also been established with other properties of concrete, such as flexural strength and modulus of elasticity (Malhotra and Carino, 2004). However, the estimated standard error in this test method varies between 10% and 25% (Kolek, 1969). The guidelines on the use of the rebound hammer test are covered in detail in BS EN 12504-2: 2001 and ASTM C805, which suggest that the rebound method should not be considered a substitute to strength determinations but only as a useful preliminary or complementary method. This technique can be used to assess uniformity of concrete quality, compare concrete with a reference in statistical terms and to indicate changes in characteristics of concrete with time. The factors that influence the surface hardness test are the type of surface, age and compaction of concrete, surface moisture conditions, type and content of cement, type of aggregate and surface carbonation of concrete. Areas exhibiting honeycombing, scaling, irregular surface and high porosity must be avoided.

Ultrasonic pulse velocity test

The ultrasonic pulse velocity (UPV) method is based on the determination of the time for a pulse of vibrations at ultrasonic frequency to travel through concrete between two transducers. Knowing the direct path length between the transducers, the pulse velocity through the concrete is obtained. The velocity of ultrasonic sound in concrete is related to its quality, uniformity and strength. This method has become widely accepted all over the world due to its speed, ease of carrying out the test and the non-destructive nature of testing concrete quality. Recommendations for the use of this method are given in BS EN 12504-4: 2004 and ASTM C597. Although good correlation between ultrasonic pulse velocity and strength of concrete was established, the guidelines given in international standards suggest that, in order to estimate compressive strength of concrete, similar correlations have to be previously established for the type of concrete under investigation (BS EN 12504-4: 2004; ASTM C597; RILEM, 1972). A typical classification of concrete quality using ultrasonic pulse velocity measurements is given in **Table 2**. The relationship between ultrasonic pulse velocity and strength of concrete depends on the type of aggregate, cement used, water/cement ratio and mix proportions (Bungey *et al.*, 2006). For a particular mix, the relationship depends on age of concrete, curing conditions and the local moisture content.

UPV: m/s	>4500	3500–4500	3000–3500	2000–3000	<2000
Concrete quality	Excellent	Good	Doubtful	Poor	Very poor

Table 2 Quality of concrete as a function of UPV (adapted from Whitehurst, 1951)

The ultrasonic pulse velocity test technique is mainly used in the non-destructive evaluation of concrete strength, uniformity of concrete, depth of surface cracks and honeycombing. The test method is also used in other applications, such as measurement of layer thickness, elastic modulus of concrete, heat of hydration and deterioration of concrete. The factors that affect the pulse velocity measurement in a concrete structure are the location of steel reinforcement, cracks and voids in the concrete, and presence of surface moisture. Presence of steel reinforcement along the path of ultrasonic wave is a particular problem because the pulse velocity through steel is approximately 40% greater than that through the concrete. Failure to take account of these factors can lead to erroneous predictions of concrete strength, for example a 5% change in pulse velocity results in a change in estimated strength of the order of 10 N/mm^2 for a typical correlation (Kay, 1992).

Pull-out test

This test is based on the concept that the compressive strength of concrete is related to the maximum tensile load applied on an embedded insert in concrete (Carino, 2004; Bungey *et al.*, 2006). Compressive strength of concrete can be predicted by obtaining an empirical relationship between pull-out tensile load and compressive strength of concrete under investigation. There are two basic categories of pull-out test: one in which a metal insert is cast into the fresh concrete at the time of construction and another where the metal insert is fixed into a drilled hole in the hardened concrete. Thus the former is used for assessing concrete strength in new construction and the latter is used on existing structures.

Unlike the rebound hammer and ultrasonic pulse velocity tests, the pull-out test subjects the concrete to a static loading and produces a well-defined fracture surface in the concrete and thus static strength property of concrete is measured. The main limitation of the cast-in approach is the fact that the position and placement of formwork inserts need to be pre-planned to ensure reinforcement does not interfere with the conical failure zone. Even in the drilled-hole method of the pull-out test, identifying the reinforcement bar is crucial by using a sensitive covermeter scan. Carino (2004) reports that the pull-out test is influenced by the ratio of mortar strength to coarse aggregate strength, type of coarse aggregate and maximum aggregate size. Another limitation of the pull-out technique is that it is necessary to repair the damage of concrete surface for aesthetical reasons. The guidelines on requirements and use of pull-out test techniques and correlations with compressive strength tests are given in BS EN 12504-3: 2005 and ASTM C 900.

Pull-off test

The pull-off test measures the in situ tensile strength of cover concrete by applying a direct tensile load via a disc bonded on the surface. Through empirical correlations, the compressive strength is predicted from the pull-off strength. This test method is commonly used in application areas such as measuring the bond strength of concrete patch repairs, assessing the effects of curing and carbonation of concrete, and to determine the effect of microcracking. The pull-off test is carried out using two approaches, one with the metal disc directly attached to the concrete surface, where strength of the concrete is measured, and the other where the concrete is cored to a depth so as to eliminate the influence of the carbonated layer of concrete and to measure strength at a depth. The approach of partial coring is used quite extensively for testing the bond strength of concrete patch repairs (BS EN 1542: 1999; ASTM C1583). The guidelines for using the test method are described in BS EN 1542: 1999 and ASTM C1583. The main advantage of the pull-off test is that it is simple and quick to perform, and no pre-planning is required to avoid the reinforcement. The entire process of preparing the surface and applying the metal disc should take no more than 15 minutes. Another advantage is that the damage caused to the concrete surface after a test is not unduly severe. The factor that influences the pull-off strength is the type and size of coarse aggregate used in concrete. The main limitation of this method is the rather long curing time required for most adhesives used for fixing the disc on the concrete surface. However, with the advancements that have taken place in the development of adhesives for the construction industry, this particular limitation is now becoming less of a problem.

Penetration resistance test

The depth of penetration of probe or a pin forced into the hardened concrete by using a specially designed gun is measured in the penetration resistance test. The depth through which the pin penetrates indicates the hardness or penetration resistance of the surface concrete, which can be related to the compressive strength of concrete (Carino, 2004). However, the test method is not recommended for testing concrete having compressive strength above 28 MPa (ACI 228.2R, 2008). The main advantage of the penetration test is that as the probe travels through both mortar and aggregate during the test, the strength properties of both the mortar and aggregate influence the penetration distance. Although results of the test are not usually influenced by local conditions such as texture and moisture content, it is significantly affected by type and size of coarse aggregate and carbonation depth (ACI 228.2R, 2008). Another limitation of this test is that the in situ concrete test member needs to be reasonably thick to withstand the large amount of energy transmitted into it during a test. It is also important to ensure that the use of this test in any situation complies with the requirements of BS 4078: Part 1: 1987 and ASTM C803 in relation to the use of powder-actuated driving units.

Maturity measurement

Maturity method is a technique to estimate in-place strength by measuring the effects of temperature and time (maturity) on strength development of concrete. By establishing the relationship between strength and maturity, the strength of a particular concrete mixture is expressed as a function of a maturity index (Malhotra and Carino, 2004; ASTM C1074-02). There are two main commercially available techniques for measuring the concrete maturity index in situ. One is to use a disposable maturity meter and the other is to use an electronic multi-channel logging device connected to embedded thermocouples or thermistors. The main advantage of the maturity method is that it is a simple non-destructive technique which takes account of the temperature history within the concrete during hydration, provided sufficient moisture is present. This is particularly valuable for construction during adverse weather conditions when knowledge of strength development is important. The main limitation of this approach is that each strength-maturity relationship is very specific to the particular concrete mix and curing regime being used. Thus, considerable expense is involved in producing unique relationships for all possible mixes. Despite this limitation, the principal application of the maturity method is for monitoring the in situ strength development in relation to stripping formwork, removal of props or the application of loads, including prestressing. The maturity method is particularly useful if it is used in conjunction with some other non-destructive testing technique, for example ultrasonic pulse velocity or surface hardness, because the different testing approaches adopted in each of these complement each other very well.

Comparison and selection of test methods

The confidence with which in situ strength assessment can be made depends on the degree of correlation with cube strength, reproducibility of the test method and number of assessments made at a particular location. The effective correlation between the test method and standard cube strength or compressive strength of cores taken from the structure increases with the number of replicate tests

Test method	Range of compressive strength: MPa	Number of valid tests at a location
Rebound hammer	10 to 40	10
Probe penetration	10 to 120	3 to 6
Pin penetration	3 to 30	3 to 6
Pull-out	2 to 130	3
Ultrasonic pulse velocity	1 to 70	5
Break off/pull-off	3 to 50	5
Maturity	No limit	–

Table 3 Compressive strength ranges for in situ strength test methods (adapted from ACI, 2008)

obtained from a particular location. Therefore, the minimum number of replicate tests needed for each test to provide an average value of strength is mentioned in **Table 3**. Further, the comparison of strength ranges for different test methods is shown in **Table 3**. In order to obtain higher strength correlation a combination of test techniques is applied, for example combinations such as pulse velocity and rebound number, maturity method with pull-out test or pull-off test can provide better estimate of in situ strength than when they are applied individually (ACI 228.2R, 2008).

Most of the non-destructive strength assessment methods suffer due to their limitations and hence the accuracy of strength estimate varies for each test method. It is important to understand the relative advantage and limitations of each of the test methods and an appropriate method has to be selected for a particular application. The comparison of performance and merits and limitations of various test methods applicable for both new and existing construction are summarised in **Tables 4** and **5**. This comparison is intended to help in selecting an appropriate test method or combination of methods for applications that depend on the test location, effect of damage and test accuracy required.

Test	Accuracy of strength estimate	Speed of test	Ease of test	Economy of test	Lack of damage to structure
Core test	****	**	**	*	*
Ultrasonic pulse	**	***	***	***	****
Surface hardness	*	***	****	****	***

Table 4 Performance of in situ strength tests (adapted from BS 6089: 1981)

Methods for measuring permeation properties

The following are the main transport processes by which the movement of aggressive substances takes place in concrete:

- *Adsorption*: the process by which molecules adhere to the internal surface of concrete. It can be either by physical forces of adhesion or as a result of chemical bonds.
- *Absorption*: the process by which concrete takes in liquid by capillary suction to fill the pore spaces available. The rate at which liquid enters the pores is termed *sorptivity*.
- *Diffusion*: the process by which a gas or ion penetrates the concrete under the action of a molar concentration gradient. It is generally defined by a diffusion coefficient or a diffusivity value.
- *Permeability*: the property of concrete that describes the resistance to a fluid (liquid or gas) penetration under the action of a pressure gradient. The rate of transport is normally expressed by a coefficient of permeability.

By measuring the permeation properties of concrete, such as absorption, diffusion and permeability, the quality of concrete against the ingress of harmful substances can be assessed. As diffusion is a slow process and determining the diffusion coefficient could take several months, an alternative method of accelerating the flow of ions through concrete by the application of a potential has been developed, which is termed migration tests. The diffusivity of

Test method	ASTM Standard	BS Standard	Accuracy		95% confidence limit of strength estimate in situ (BS 6089: 1981)	Ease of use
			New construction	Existing construction		
Rebound number	C 805	12504-2	+	+	±20% (concrete not older than 3 months)	++
Penetration resistance	C 803/C 803M	1881-207	+	+	±20%	++
Pull-out	C 900	12504-3	++	++	±20%	+
Break-off/pull-off	C 1150	1881-207	++	++	±20% (±15% in laboratory conditions)	+
Pulse velocity	C 597	12504-4	++	+	±20%	+
Maturity	C 1074	–	++	N/A	–	+

Table 5 Comparison of performance of in situ strength tests (adapted from ACI 228, 2008)

Test method	Intended purpose	Classification	Reference
Initial surface absorption test	Absorption of the concrete cover	NDT	BS 1881: Part 208; Basheer, 1999
Figg water permeability test	Absorption in a drilled hole in the concrete cover	PDT	Basheer, 1999; Figg, 1973
Figg air permeability test	Air permeability in a drilled hole in the concrete cover	PDT	Figg, 1973; Basheer, 1999
Torrent test	Air permeability of the concrete cover	NDT	Torrent, 1992; Torrent and Luco, 2007
Autoclam air permeability test	Air permeability of the concrete cover	NDT	Basheer, 1993
Autoclam sorptivity test	Absorption of the concrete cover	NDT	Basheer, 1993
Autoclam water permeability test	Water permeability of the concrete cover	NDT	Basheer, 1993
Gas permeability test using permeability cell	Permeability of concrete	DT	Basheer, 1999
Water permeability test using permeability cell	Permeability of concrete	DT	Basheer, 1999
Water absorption by immersion	Water absorption capacity of concrete	DT	Basheer, 1999
BS water absorption	Short-term water absorption of concrete	DT	BS 1881: Part 122; Concrete Society, 1988
DIN 1048 absorption test	Watertightness of concrete	DT	DIN 1048; ISO/DIS 7031, 1983
Capillary absorption (sorptivity) tests	Rate of water absorption of concrete	DT	Basheer, 1999
Rapid chloride permeability test	Ionic conductivity of concrete	DT	AASHTO-T277, 1983
Permit ion migration test	Ionic transport of concrete	NDT	Basheer, 1999
Ion migration/diffusion test using diffusion cells	Ionic diffusion resistance of concrete	DT	Basheer, 1999; ASTM C1202
Water vapour diffusion test	Water vapour transport of concrete	DT	Basheer, 1999

Table 6 Permeation methods

concrete determined by the migration tests conforms to the diffusivity obtained from the laboratory-based diffusion tests (Nanukuttan *et al*., 2006). Therefore, migration tests which can be performed on site are also included in this section along with other test methods which assess the permeation properties of concrete. **Table 6** gives a summary of all permeation methods and those tests which can be used on site are elaborated in this section.

Absorption tests

Initial surface absorption test (ISAT)

In this test the rate of penetration of water into a defined area of concrete (5000 mm^2) after a stated interval from the start of the test (10 min, 30 min and 60 min) at a constant water head (200 mm height) and temperature (20°C) is measured. There are specifications for preparing the test area, initial moisture condition, surface area to be in contact with water and the time at which measurements have to be taken. The guidelines on the test method, calibration and sample preparation are available in BS 1881: Part 208. Typical ISAT results reported by Concrete Society (1988) are given in **Table 7**. The main advantage of this test is that it is a quick and simple non-destructive in situ test for measuring the water penetration into a concrete surface. This test method provides reasonably consistent results on oven dried samples. The limitations of this test include difficulty of ensuring a watertight seal in site conditions, influence of moisture condition of concrete on measured property and insufficient water head.

Autoclam sorptivity test

The autoclam sorptivity test (Basheer *et al*., 1994; Basheer *et al*, 1995) allows the measurement of the cumulative inflow of water in the first 15 min from a water source of 50 mm diameter (i.e. from a base ring of internal diameter 50 mm) at an applied pressure of 0.02 bar (nearly 200 mm water head). A plot of cumulative volume of water versus square root of time gives a linear relationship and the slope obtained from the graph is reported as a sorptivity index. In the case of less permeable surfaces where flow of water is low, a larger contact area of water source can be used to obtain measurable flow rates. However, the measured data should be normalised to the standard 50 mm diameter water source. **Table 8** shows the typical values of autoclam sorptivity indices for different levels of protective quality of concrete. The moisture content of concrete surface has been reported to influence the auto-

Comment on concrete absorption	ISAT results ml/m^2/s		
	Time after starting test		
	10 min	30 min	1 h
High	>0.50	>0.35	>0.20
Average	0.25–0.50	0.17–0.35	0.10–0.20
Low	<0.25	<0.17	<0.10

Table 7 Typical results of initial surface absorption test (adapted from Concrete Society, 1988)

Permeation property	Protective quality			
	Very good	Good	Poor	Very poor
Clam sorptivity index ($m^3 \times 10^{-7}/\sqrt{min}$)	$\leqslant 1.30$	$>1.30 \leqslant 2.60$	$>2.60 \leqslant 3.40$	>3.40

Table 8 Performance of concrete based on autoclam sorptivity index (Basheer, 1993)

clam sorptivity index, Basheer and Nolan (2001) proposed the test to be carried out when the internal relative humidity of concrete at 10 mm depth is less than 80% to eliminate this effect. The equipment is portable and easy to use on site. Fixing problems identified for the ISAT have been eliminated for the autoclam.

Figg water permeability (absorption) test

This test involves drilling a hole of 10 mm diameter in the near-surface layer of concrete to a depth of 40 mm. After thorough cleaning, the hole is plugged at the top 20 mm depth by a silicon rubber plug, leaving a cavity of 20 mm deep for carrying out the test. Water is admitted to this cavity at a low pressure head of 100 mm and allowed to be absorbed by capillarity. The time required for a certain volume of water to be absorbed is measured in seconds as an absorption index, thus a higher absorption index corresponds to better quality concrete. The relationship between concrete protective quality and Figg water permeability indices are shown in **Table 9**. The main advantage of this test method is that it is simple and relatively low cost compared to other test techniques. The main disadvantage of this test is that drilling, even at slow speed, may introduce microcracks, which may defeat the purpose of the test by altering the flow mechanism.

Permeability tests

Autoclam water and air permeability tests

The autoclam water permeability test involves a procedure similar to that used for the autoclam sorptivity test. The main difference is in the test pressure used, i.e. a pressure of 1.5 bar is used for the autoclam water permeability test. The inflow of water through a test area of 50 mm diameter through a surface-mounted ring is measured at this pressure for a period of 15 min. From a linear plot of the cumulative inflow and square root of time, the slope is determined and reported as the autoclam water permeability index, in $m^3/\sqrt{min}$. As the inflow is measured in this test, the test location has to be dry to obtain meaningful data. Protective quality of concrete based on autoclam air permeability index values are shown in **Table 10**.

Concrete category	Protective quality	Water permeability index	
		Measured time: s	Equivalent WAR* value: s/ml $\times 10^3$
0	Poor	<20	<2
1	Not very good	20–50	2–5
2	Fair	50–100	5–10
3	Good	100–500	10–50
4	Excellent	>500	>50

* Water absorption rate

Table 9 Relationship between concrete protective quality and Figg water permeability indices (Basheer, 1993)

Permeation property	Protective quality			
	Very good	Good	Poor	Very poor
Clam air permeability index (ln(pressure)/min)	$\leqslant 0.10$	$>0.10 \leqslant 0.50$	$>0.50 \leqslant 0.90$	>0.90
Clam water permeability ($m^3 \times 10^{-7}/\sqrt{min}$)	$\leqslant 3.70$	$>3.70 \leqslant 9.40$	$>9.40 \leqslant 13.8$	>13.80

Table 10 Protective quality based on autoclam air and water permeability indices (Basheer, 1993)

The autoclam permeability system, which enables the autoclam sorptivity and water permeability test to be carried out, is used to determine the autoclam air permeability index of concrete (Basheer, 1993; Henderson *et al.*, 2004). This test depends on the measurement of pressure decay in a test reservoir mounted on the surface of concrete from 1.5 bar over a period of 15 min and plotting the natural logarithm of the pressure against time, which gives a straight line graph. The slope of this graph is reported as the autoclam air permeability index. Although moisture influences the test results, Basheer and Nolan (2001) have shown that the quality of concrete can be classified in terms of the autoclam air permeability index if measurements are taken when the internal relative humidity in a cavity in concrete at a depth of 10 mm from the surface is less than 80%.

The main advantage of this test is that it is portable, quick and simple to perform. However, as in the case of the autoclam sorptivity test, this test provides air and water permeability indices rather than coefficients of permeability.

Figg air permeability test

This test, also commercially known as Poroscope, uses a test set-up similar to that used for the Figg water test but, instead of admitting water into the cavity, a pump is used to reduce the pressure in the cavity to 55 kPa below atmospheric. As air flows into the cavity, the pressure increases and the time taken for the pressure to increase by 5 kPa, that is to a cavity pressure of −50 kPa, is recorded using a

Concrete category	Protective quality	Water permeability index	
		Measured time: s	Equivalent AER* value: s/ml
0	Poor	<30	<8
1	Not very good	30–100	8–25
2	Fair	100–300	25–75
3	Good	300–1000	75–250
4	Excellent	>1000	>250

* Air exclusion rating

Table 11 Relationship between concrete protective quality and Figg air permeability indices (Basheer, 1993)

stopwatch and this is reported as the air permeability index. Similar to the Figg water test, a higher air permeability index value corresponds to a less permeable concrete. Typical values of Figg air permeability indices based on different levels of protective quality of concrete are shown in **Table 11**. The advantages and disadvantages of the Figg water absorption test also apply to the Figg air permeability test.

Torrent test

This test is very similar to the Schönlin air permeability test (Schönlin and Hilsdorf, 1987) and uses a surface-mounted reservoir to apply a vacuum to the test surface. Although the concept of the Torrent test is similar to that of the Figg air permeability test, damage caused by drilling a hole in concrete is avoided. The Torrent test uses an annular reservoir along with the central test reservoir in an attempt to direct the flow of air into the test reservoir in a unidirectional manner. Using this assumption of unidirectional flow of air, a permeability coefficient is reported from measurements taken in this test. It is claimed that the electrical resistivity of concrete can be used to adjust the readings from the Torrent test to take account of the moisture variations in concrete (Torrent and Luco, 2007).

There are several other non-destructive test methods for evaluating air and water permeability of concrete, details of which can be found in published literatures elsewhere (Germann Instruments, http://www.germann.org; Guth and Zia, 2001; Basheer, 1999).

Migration-based tests

Rapid chloride permeability test (RCPT)

Whiting (1981) developed an in situ rapid chloride permeability test based on the same principle as the laboratory test (ASTM C1202), where the charge passed during the first six hours after the application of a potential across the specimen is used to characterise the concrete for chloride penetration. A detailed description of the test instrument is available elsewhere (Whiting, 1981). Whiting suggested the test to be carried out at a potential difference of 80 V for a period of six hours. The total charge passed is used as an index to characterise concrete for chloride diffusivity. The disadvantages of this test technique are the same as that for laboratory version, as mentioned below.

- The test result is influenced by the conductivity of the pore solution, which can be highly variable on site.
- The temperature rise due to the high voltage (80 V) will significantly affect the charge passed during the test.
- The depth of concrete cover will affect the test results; for example, if a cover of 50 mm is assumed, results may vary as much as 25% if the actual cover varies by 25 mm from this value.

In addition, unlike the laboratory-based test specified in ASTM C1202, it has been difficult to saturate the test area prior to carrying out the RCPT on site. Although Whiting proposed a vacuum saturation technique, this was not entirely satisfactory for yielding reliable and repeatable results.

Permit ion migration test

This is a unique non-destructive test which is capable of determining the chloride migration coefficient of cover concrete. Detailed descriptions of the instrument, test technique and test area preparation are available elsewhere (Basheer *et al.*, 2005; Nanukuttan *et al.*, 2006). Extensive testing using the permit ion migration test on different concrete samples has indicated that the in situ migration coefficient from this test correlates well with the conventional laboratory-based steady state diffusion and migration coefficients. The main advantage of this test is that it can provide a migration coefficient without having to remove cores from the structure. The main disadvantage of the test is that it introduces chloride ions into the test surface. However, the chloride ions within the test area can be effectively removed by reversing the test voltage, which may take up to four days. It is also to be noted that the test area will be slightly stained by deposition of the ferrous by-products of the electrochemical reaction. However, if this deposit is washed off immediately after the test, the extent of the staining can be reduced.

Comparison and selection of test methods

The selection of appropriate permeation test technique (sometimes the term penetrability test is also used) for assessing the condition of concrete in structures basically depends on the associated transport mechanism that causes deterioration. However, most of the test methods have common limitations and it is essential to overcome the limitations by understanding the factors causing them. These common limitations include: sensitivity to moisture

Transport mechanism	Gas permeability		Water sorptivity
Methods	Autoclam air	Torrent	Autoclam sorptivity
Significant discrimination	5/7	6/7	6/7
Correlation coefficient *R**	0.67	0.97	0.47
Measurements per test condition	3	6	3
Duration per test condition (min)	69	99	69
Visual impact: No. holes × diameter (mm)	9 × 6	0	9 × 6

* With reference test for the same transport mechanism

Table 12 Performance of the different permeability test methods (adapted from Romer, 2005)

and temperature condition in concrete, changes in transport mechanism during the test and influence of drilling on measured values.

A detailed experimental study to compare different NDT techniques for measuring penetrability characteristics of cover concrete and thereby detecting the changes in quality of concrete was carried out by RILEM committee 189 (Romer, 2005). The performance of NDT methods was compared using seven different concrete types and the results are summarised in **Table 12**. It can be seen that in five or six out of seven cases the test methods were capable of detecting expected variations in penetrability of concrete at a significant level.

Methods of measuring corrosion of steel in concrete and corrosion-related concrete properties

In the case of corrosion of steel reinforcement, by the time visible evidence of corrosion damage is seen on the surface in the form of cracks and rust staining, remedial action can be very costly and often ineffective. Therefore, frequent assessment of the condition of the structures is essential for ensuring their serviceability, low cost of maintenance and the operational safety. Different non-destructive methods used to assess the corrosion of steel in concrete and those which measure corrosion-related properties of concrete are summarised in **Table 13**, and some of the most commonly used methods are described in this section.

Electromagnetic cover measurement

Covermeter survey is a non-destructive test method to measure the depth of cover to reinforcement in concrete structures. The covermeter electromagnetically locates the embedded steel and measures the intensity of the magnetic field produced by the embedded steel. This intensity can be correlated to a specific depth from the concrete surface thus measuring the cover. The accuracy of covermeters in measuring cover depth under normal site conditions is within ±5 mm (BS 1881: Parts 201: 1986 and 204: 1988). However, in heavily reinforced members, the effect of secondary reinforcement does not allow accurate measurements of cover. The modern versions of covermeters locate main and secondary reinforcement, determine bar sizes and bar spacings. The output of measurement is given in the form of a scan. Apart from knowing the extent of damage at the low cover region, covermeters are also used to know the exact location of reinforcement for coring or drilling tests and repair detailing. The factors influencing the covermeter test include bar diameter, spacing of bars, aggregate with magnetic properties and other electromagnetic interferences at site. BS 1881: Part 204: 1988 gives recommendations and describes the principles of

Test method	Intended purpose	Classification	Reference
Electromagnetic covermeter	Identification of cover to reinforcement, rebar diameter, bar clusters	NDT	BS 1881: Parts 201 and 204
Half-cell potential apparatus	Corrosion potential (probability of corrosion)	NDT	ASTM C876; Elsener, 2001
Wenner Four Probe resistivity meter	Resistivity of concrete	NDT	BS 1881: Part 201; Polder, 2001
Linear Polarisation Resistance test	Rate of corrosion of the rebar	NDT	Andrade and Alonso, 2001
Galvanostatic pulse measurement	Rate of corrosion of the rebar	NDT	Gulikers and van Mier, 1994
AC Impedance Analysis	Rate of corrosion of the rebar	NDT	Devalapura *et al.*, 1994
AC Harmonic Analysis	Rate of corrosion of the rebar	NDT	Bungey *et al.*, 2006; Malhotra and Carino, 2004

Table 13 Tests for assessing corrosion of reinforcement and corrosion-related properties of concrete

Copper/copper sulfate	Silver/silver chloride/1.0M KCL	Standard hydrogen electrode	Calomel electrode	Corrosion condition
>−200 mV	>−100 mV	+120 mV	>− 80 mV	Low (10% risk of corrosion)
−200 to −350 mV	−100 to −250 mV	+120 to −30 mV	−80 to −230 mV	Intermediate corrosion risk
<−350 mV	<−250 mV	−30 mV	<−230 mV	High (>90% risk of corrosion)
<−500 mV	<−400 mV	−180 mV	<−380 mV	Severe corrosion

Table 14 ASTM specification for corrosion of steel in concrete for different standard electrodes (adapted from ASTM C876; Elsener, 2001)

operation of the electromagnetic devices which are used for estimating the position, depth and size of reinforcement buried in concrete.

Half-cell potential measurement

The half-cell potential method consists of estimating the electrical half-cell potential of reinforcing steel in concrete for the purpose of determining the potential corrosion activity of steel. The corrosion potentials of steel are measured against a reference consisting of electrode of a metal in an electrolyte. The commonly used reference half-cell is copper in copper sulfate or silver in silver chloride and other combinations (ASTM C876, 2007). The criteria for assessing corrosion condition for different reference electrodes as per ASTM C876 are presented in **Table 14**. There is no equivalent British Standard, but BS 1881: Part 201 gives a brief description on limitations and applications of this method. Half-cell potential measurements are often carried out when reinforcement corrosion is suspected or evident in a structure. This method is widely used as a low-cost test method that provides iso-potential contour maps used to easily identify zones of corrosion risk. However, it should be noted that the potentials obtained depend on the presence of moisture, therefore to counter for seasonal variations an average of many readings taken during different weather conditions should be considered. The test could also give misleading results for realkalinised concretes and concretes treated with organic or inorganic inhibitors (Elsener, 2001).

Electrical resistivity method

Corrosion of steel in concrete is an electrochemical process. Measuring electrical resistance of concrete indicates the rate at which corrosion reaction can proceed. A two- or four-point probe is often used to measure the resistivity of concrete. The criteria for assessing risk of corrosion using the electrical resistivity method are shown in **Table 15**. In order to assess the corrosion activity better, in situ resistivity measurements are often carried out in association with half-cell potential surveys. The resistance of concrete is governed by factors such as geometry between the measuring electrodes, porosity and pore connectivity of cover concrete and ionic mobility in the pore system (COST Action 521, 2003). Similarly, the electrical resistivity measurements are influenced by moisture and temperature, presence of reinforcement close to the surface, maximum size of aggregates and carbonation of concrete. Polder (2001) has suggested a method of avoiding the effect of reinforcement on electrical resistivity values.

Measurement of rate of corrosion

There are several methods of measuring true and instantaneous rate of corrosion and many of them are electrochemical methods that rely on measuring half-cell potentials. These methods predict the rate of corrosion using electrochemical theory. One of the most commonly used methods to determine the rate of corrosion of steel reinforcement in concrete is linear polarisation resistance technique. This method eliminates the major drawbacks encountered in other corrosion assessment methods by determining the rate of corrosion directly. The principle of linear polarisation resistance technique relies on applying a small electric current and measuring the shift in corrosion potential, thus monitoring the active corrosion in steel. For non-destructive testing of the rate of corrosion a guard ring system has been developed which takes local corrosion measurement in a defined area. A brief overview of this technique, its advantages and limitations are given in the COST Action 521 Report (2003). The criteria for interpretation of damage due to corrosion using corrosion rate measurements are shown in **Table 16**. Factors which influence the corrosion rate measurements are seasonal variations in temperature and moisture conditions.

Comparison and selection of test methods

Each of the test methods discussed in this section has a specific accuracy and advantages and limitations in

Resistivity: kΩ cm	Likelihood of significant corrosion when steel activated (for non-saturated concrete)
>20	Low
10 to 20	Low/moderate
5 to 10	High
<5	Very high

Table 15 Empirical relationship between resistivity of concrete and likelihood of significant corrosion (adapted from Bungey *et al*., 2006)

Corrosion rate: μA/cm^2	Attack penetration (loss of section of reinforcing bar): μm/year	Corrosion level
<0.1	<1	Negligible
0.1 to 0.5	1 to 5	Low
0.5 to 1	5 to 10	Moderate
>1	>10	High

Table 16 Typical interpretation of corrosion rate measurements (adapted from COST Action 521, 2003)

measuring the corrosion of steel in concrete structures. **Table 17** summarises and compares the advantages and limitations of non-destructive testing methods used for assessing steel in concrete. Comparative tests on commercially available covermeters indicate that all the instruments are capable of measuring cover depth with 100% success at an accuracy of ±10% provided the nominal diameter is known and cover depth is less than 40 mm (Luco, 2005). Resistivity measurements provide a good indication of the quality and density of cover concrete, whereas half-cell potential measurements provide information on the likelihood of corrosion. Both these methods are not capable of measuring directly the corrosion rate. Although the linear polarisation resistance technique can give a direct measure of the corrosion rate, the data interpretation is complex and better understanding is needed on factors influencing the measurements. Generally, in order to make use of the data from all these test methods, a deeper understanding of the instrument as well as method of interpreting the data in terms of damage in structures is essential. It is also important to understand the influence of environmental factors on the measurements as well as methods to overcome them.

The combination of test techniques can improve the accuracy and reliability of corrosion assessment in structures. The choice of preliminary method in assessing corrosion activity is decided by taking into account the practical circumstances and limitations of the test method. In most cases corrosion assessment is started by using a covermeter survey which gives a good indication of position of reinforcement and areas of low cover. The combination of half-cell potential and resistivity methods represents the most effective non-destructive approach for assessing corrosion damage. However, other chemical analysis, for example chloride content and carbonation depth, should accompany such structural assessment.

Integrity evaluation

Integrity evaluation generally deals with detecting and identifying the location of hidden cracks, voids, honeycombing and delamination in concrete structures. These hidden defects affect the structural and durability performance of structures. Hammer tapping survey is a manual

Method	Advantages	Limitations
Covermeter	■ Truly non-destructive technique ■ Lightweight, portable and easy to use ■ Cover depth can be estimated at a site accuracy of ±5 mm ■ Ability to locate reinforcing bar and estimate bar size and depth of cover	■ Calibrations are sensitive to steel type, bar diameter and aggregate type ■ Accuracy of estimated cover depth is affected if bar size is less than 10 mm or greater than 32 mm ■ Ability to identify individual bars is affected by probe orientation, cover depth, spacing of bars, presence of secondary reinforcement and lap joints
Resistivity	■ Simple, lightweight and easy to use NDT technique ■ Can be used to predict chances of reinforcement corrosion	■ Correlation with properties of concrete are dependent on individual mix proportions ■ Influenced by moisture and salt content, temperature and water/cement ratio ■ Practical applications are restricted to comparative measurements
Half-cell potential	■ Non-destructive, portable and lightweight equipment ■ Provides ISO-potential contour maps indicating zones of varying degrees of corrosion risk at time of testing	■ Method cannot indicate actual rate of corrosion ■ Electrical contact with the reinforcement has to be established by drilling a small hole ■ Concrete has to be moist ■ Not applicable to epoxy-coated bars ■ Requires an experienced operator to test and interpret the results
Linear polarisation	■ Direct measurement of corrosion rate of steel in concrete ■ Lightweight and portable ■ Guard ring probe measures localised corrosion rate	■ Measurement is slow compared with other methods ■ Electrical contact with the reinforcement has to be established by drilling a small hole ■ Cover depth has to be less than 100 mm ■ Measurements are influenced by seasonal variations in temperature and moisture ■ Concrete surface has to be smooth, uncracked and free from coatings ■ Requires an experienced operator to test and interpret the results

Table 17 Advantages and limitations of NDT methods for assessment of steel (adapted from ACI, 2008; BS 1881: Part 201: 1986)

method popularly used as an inexpensive technique in identifying areas of delamination of concrete. There are more sophisticated methods, such as infrared thermography, impact-echo method and radar techniques for non-destructive assessment of flaws in concrete (**Table 18**). The most commonly used amongst these methods are described in this section (ACI228, 1998).

Manual method

A range of manual methods is available for identifying areas of delamination in concrete. These methods work on the principle that when a delaminated concrete layer is struck with a metal, it produces a sound indicating hollow cavity behind the surface of concrete cover. In these manual methods, hand tools such as hammer, steel rods and chain drags are used for assessing the structure for possible delamination. Steel chain drags are normally used for large horizontal areas of concrete such as concrete bridge decks. Although the interpretation of sound produced cannot be quantified and is subjective based on the operator, this method is widely used because it is fast, easy to use and less expensive than other sophisticated techniques. The use of manual hammer or steel chain is often carried out during the process of visual inspection of concrete structures. During this survey, delaminated regions are marked and other non-destructive tests are conducted on the identified portions to assess the extent of damage.

Infrared thermography

Infrared thermography is a remote method of scanning the emission of thermal radiation from a surface and producing the thermal signal as a visual image. This technique measures the variations in surface radiance on a concrete member undergoing heating or cooling. Therefore, this method is mainly used to identify the delamination in concrete by measuring the surface temperature differentials between solid and delaminated concrete. Hidden cracks and voids inside the concrete can also be detected using this technique. Best observations using the thermography technique can be made when the concrete is warming up or cooling down, because delaminated concrete heats or cools faster during this period. The main advantage of this technique is its ability to scan large surface areas in a short period of time. Some modern systems have the option to capture both visual and thermal images, which help in identifying the location of defects and to provide indication of percentage of deterioration in a scanned area (Weil, 2004). The major limitation of this test technique is the influence of external environmental conditions such as changes in air temperature, wind, shading or direct sunlight on the test surface and presence of surface water. This is an expensive technique in comparison with traditional methods of identifying delamination in concrete structures. The guidelines and detailed description on the use of thermography technique for detecting delamination in concrete bridge decks are available in ASTM D4788 (2002). Further information on the test method can be obtained elsewhere (Titman, 2001; Maierhofer *et al*., 2005).

Impact-echo and impulse response method

This method is based on the impact-generated stress waves, basically sound waves, which propagate through concrete and are reflected by internal flaws and external surfaces. The instruments based on this technique introduce stress pulse into the test concrete by means of a transducer or by mechanical impact. If a transmitter is used the method is called pulse echo and if mechanical impact is used, the method is called impact echo (Sansalone and Streett, 1997). Thus, using the travel time of pulse and wave velocity in the medium, the location of defect is measured. The main application of this test technique is to detect the location and extent of flaws in concrete, such as cracks, delaminations, voids, honeycombing, bonding of repairs, voids in tendon ducts (Zheng and Ng, 2006) and debonding in post-tensioned members. They are effective in identifying flaws in structures which are inaccessible with other NDT techniques; for example, this technique is popularly used

Test method	Intended purpose	Classification	Reference
Tapping	Delamination of concrete cover	NDT	Bungey *et al*., 2006; ASTM D4580
Pulse echo method	Internal defects in concrete	NDT	Carino, 2004
Impact echo method	Internal defects in concrete and concrete piles	NDT	Cheng and Sansalone, 1993
Acoustic emission	Internal defects in concrete elements	NDT	Manning and Holt, 1980
Infrared thermography	Delamination and near-surface internal defects in concrete	NDT	Manning and Holt, 1980; Holt and Eales, 1987
Radar	Internal defects in concrete	NDT	Bungey *et al*., 2006; McCann and Forde, 2001
Radiography	Internal flaws in concrete	NDT	BS 1881: Part 205: 1986; Bungey, 1992
Radiometry	Internal flaws in concrete	NDT	BS 1881: Part 201: 1986

Table 18 Integrity tests

in evaluation of pile foundations or other deep foundations (Davis, 1997). The major limitations are that the accessibility to the structure determines the accuracy of the results and the data analysis is a very time-consuming process.

Radar method

The detection of deterioration by radar is based on reflections of a high-frequency electromagnetic wave caused by changes in the electromagnetic properties of the material being probed. This method is similar to the pulse-echo technique discussed previously, except that electromagnetic waves such as radio waves and micro waves are used instead of stress waves. Whenever a transmitted wave encounters an electromagnetic discontinuity (deterioration) or change of material dielectric, it is partially reflected. The patterns created by the reflected waves are received by the radar antenna. The radar receiver takes a measurement of the time required for the transmitted pulse to travel to a target discontinuity and for the reflected pulse to return. The time delay of the radar echo from a structural discontinuity is directly related to the depth of the fault. This is a truly non-destructive technique providing a rapid method of analysing defects lying below the surface of concrete structures. However, the main limitation is that both testing and interpretation are highly specialised, which makes it an expensive technique. There are two ASTM Standards on specific application of this technique (ASTM D4748; ASTM D6087).

Other durability related tests

Tables 19 to **23** present tests which are used to supplement the non-destructive tests described above. Further information on these tests can be obtained from the references listed against each. As can be seen from the tables, most of these tests are laboratory-based destructive tests, but they are essential complements of the NDT methods.

Test method	Intended purpose	Classification	Reference
Underwater method	Cavitation/erosion resistance of concrete	DT	ASTM C1138
Rotating-cutter method	Abrasion/attrition resistance of concrete surfaces	DT	ASTM C944
Abrasion resistance of horizontal surfaces test	Abrasion/attrition resistance of concrete surfaces	DT	ASTM C779
Sandblasting method	Abrasion/erosion resistance of concrete surfaces	DT	ASTM C418

Table 19 Abrasion/erosion resistance tests

Test method	Intended purpose	Classification	Reference
Phenolphthalein test	Depth of carbonation of concrete	DT	RILEM, 1985
Sulfate resistance test	Resistance of concrete to sulfate attack	DT	Hime, 1999
Sulfate content	Resistance of concrete to sulfate attack	DT	BS 1881: Part 124: 1988
Chloride content	Risk of corrosion of steel due to chlorides	DT	BS 1881: Part 124: 1988
Cement content	Conformity to specifications	DT	BS 1881: Part 124: 1988; ASTM C85
Alkali content	Resistance of concrete to alkali attack and alkali–silica reaction	DT	BS 1881: Part 124: 1988
Alkali immersion test	Resistance of concrete to alkali–aggregate reaction and resistance of rock sample to alkali–aggregate reaction	DT	BCA, 1992

Table 20 Chemical analysis/tests

Test method	Intended purpose	Classification	Reference
Critical dilation test	Freeze–thaw resistance of concrete	DT	ASTM C671
Resistance to rapid freezing and thawing test	Freeze–thaw resistance of concrete	DT	ASTM C666; Pigeon and Pleau, 1995
Surface scaling resistance test	Scaling resistance of concrete due to cyclic freezing and thawing in presence of salt	DT	ASTM C672; SS 13 72 44, 1988

Table 21 Freeze–thaw resistance tests

Test method	Intended purpose	Classification	Reference
Optical microscopy	Petrographic analysis of materials, air void distribution	DT	ASTM C856
Scanning electron microscopy	Microstructure of polished concrete surfaces	DT	ASTM C457; Sarkar *et al*., 1999
Differential scanning calorimetry and thermo-gravimetry	Chemical phases of powdered concrete samples	DT	Ramachandran, 1999
Powder diffraction analysis using X-ray diffraction	Mineralogical composition of powdered concrete and polished concrete	DT	Chatterjee, 1999

Table 22 Microstructure analysis

Test method	Intended purpose	Classification	Reference
Neutron moisture gauge	Moisture content of concrete	NDT	BS 1881: Part 201: 1986
Microwave absorption	Moisture content of concrete	NDT	Bungey *et al*, 2006
Chilled mirror moisture probe	Moisture content of concrete	PDT	Parrott, 1988
Conductivity probe	Moisture content of concrete	PDT	http://concretendt.com/

Table 23 Moisture measurement tests

Concluding remarks

From the point of view of using the tests for different site applications, they can be grouped as shown in **Table 24**. It may be noted that some of the tests can be used for more than one purpose, but in no way does the table aim to rank the tests in preferential order. With no two investigations being the same, it is the investigator who has to select the best approach and tests to be adopted. The need to have a thorough understanding of the deterioration mechanisms and the causes of distress cannot be overstated in order to select the most appropriate tests from those listed in **Table 23**. Furthermore, there is a need to understand the principle of operation of each of the test methods and their limitations

Application →	Reinforcement location	Corrosion risk and cause	Rate of corrosion	Chemical deterioration	Internal defects	Alkali–aggregate reaction	Fire damage	Erosion and abrasion	Frost damage
Test methods	Electromagnetic covermeter	Visual inspection	Linear polarisation resistance	Visual inspection	Ultrasonic pulse velocity	Visual inspection	Visual inspection	Visual inspection	Visual inspection
	Radiography	Electromagnetic covermeter	Galvanostatic pulse	All chemical tests	All integrity tests	All microstructure methods	All microstructure methods	All abrasion tests	All frost resistance tests
	Radar	Half-cell potentials	AC impedance	All microstructure methods		Alkali content	All strength tests		Ultrasonic pulse velocity
		Wenner Four Probe resistivity	AC harmonic analysis	Surface scaling test		Alkali immersion test	Ultrasonic pulse velocity		Pulse echo method
		All permeation Methods	Half-cell potentials	All permeation methods		All moisture tests	All integrity tests		Impact echo method
		All moisture tests	Electrical resistivity test	All moisture tests			Rebound hammer		Optical microscopy
		Phenolphthalein test		Pull-off test					Scanning electron microscopy
		Cement content		Pull-out test					All permeation methods
		Chloride content		Break-off test					

Table 24 Use of different test methods for different applications

in order to apply the various methods either independently or in combination in an effective manner. Therefore, **Table 24** is intended to be a guide to the tests which may be performed in each case, with the emphasis placed on the engineer to select the most appropriate test(s) for the situation. Quite often the equipment available to the engineer decides the testing procedure. Other factors influencing the testing programme are the cost of the various tests in relation to the value of the project, the cost of delays to construction, and the cost of possible remedial works.

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ASTM C597. Standard Test Method for Pulse Velocity Through Concrete, Philadelphia, PA: American Society for Testing and Materials.

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ASTM C671. Standard Test Method for Critical Dilation of Concrete Specimens Subjected to Freezing, Philadelphia, PA: American Society for Testing and Materials.

ASTM C672. Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals, Philadelphia, PA: American Society for Testing and Materials.

ASTM C779. Standard Test Method for Abrasion Resistance of Horizontal Concrete Surfaces, Philadelphia, PA: American Society for Testing and Materials.

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Chapter 18

Special concretes

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Concrete, in the past few decades, had to change to meet new, more demanding and ever changing needs of the construction industry. The ‘changes’ have been made possible with the concurrent development of new materials, e.g. chemical and mineral admixtures, that have enabled the compressive strengths to quadruple, the expectations of durability performance to increase, and at the same time the ease with which concrete is transported and placed made easier. This new breed is called ‘special concretes’. In this section, the most commonly used special concretes are introduced and their characteristics as a construction material are highlighted.

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Introduction

Concrete compressive strengths range from 20 N/mm^2 (low-strength fill or foundation material) to 200 N/mm^2 (ultra-high-performance fibre-reinforced concrete). Within this range lies concrete with varying water/cement ratios and cell/capillary structures, which are linked to durability qualities, and varying flow/rheological properties, which are linked to ease of placement. A wide range of chemical admixtures can also be used to not only improve the rheological properties of the fresh concrete but also improve its durability in the hardened state. Aggregate type will affect the density, compressive strength, permeability and subsequent durability as well as providing a way of recycling the concrete itself into new concrete products. Cement type, and the use of supplementary cementitious materials, will influence the durability, setting time, heat of hydration and therefore peak temperature rise, of the concrete. It is not therefore surprising that, with all of these possible combinations of materials, we have today a long list of ‘special’ concretes.

Lightweight aggregate concrete

Lightweight aggregate concrete is defined as having an oven-dry density of less than 2000 kg/m^3 and made with aggregates with density less than 2000 kg/m^3. Lightweight aggregates used in construction are produced from a wide variety of materials: shale, clay, slate, blastfurnace slag, fly ash, pumice, diatomite, perlite and vermiculite. Pyro-processing methods include the rotary kiln process (a long, slowly rotating, slightly inclined cylinder lined with refractory materials similar to cement kilns); the sintering process wherein a bed of raw materials, including fuel, is carried by a travelling grate under an ignition hood; and the rapid agitation of molten slag with controlled amounts of air or water (ACI Committee 213). Naturally occurring lightweight aggregates are mined from volcanic deposits that include pumice and scoria. The properties of lightweight aggregates differ greatly, e.g., densities vary between 30 and 900 kg/m^3. Only the aggregates produced from shale, clay, slate, blastfurnace slag and fly ash possess sufficient strength for structural concrete, see **Table 1** (Clarke, 1993). The other lightweight aggregates are used in non-load-bearing but insulating applications.

Structural lightweight concrete is usually semi-lightweight in that the coarse aggregate is lightweight and all or part of the fine aggregate is normal weight (Department of Energy, Mines and Resources, 1979). This semi-lightweight structural concrete incorporating lightweight aggregate sized 19.0 to 4.75 mm and natural sand or minus 4.75 mm will have densities of between 1760 and 2000 kg/m^3 and will have 28-day compressive strengths up to

Name	Type	Process	Shape and texture	Bulk density: kg/m^3	f_{cu}: N/mm^2
Foamed slag	Foamed slag	Foaming bed	Angular vesicular	750	<40
Leca or fibo	Expanded clay	Rotary kiln	Rounded, smooth	425	<30
Lytag	Sintered pfa	Sinter strand	Rounded, fine	825	>40
Pellite	Blastfurnace slag	Pelletisation	Irregular, smooth	900	>40
Liapor	Expanded shale	Rotary kiln	Rounded, smooth	650	>40

Table 1 Lightweight aggregates available in the UK (adapted from Clarke, 1993)

35 N/mm^2 and in many cases up to 40 N/mm^2. Concrete strength will depend on the strength of the lightweight aggregate as well as on the mix proportions. Invariably, a lightweight concrete will require a slightly higher cement content than would a normal concrete of comparable compressive strength. Endeavours have been made to correlate the crushing strength of aggregates to the compressive strength of lightweight concrete, but the relationship is tenuous at best. Absorption of the aggregates also varies, but is generally between 5 and 20%. This is considerably higher than that of normal aggregates and is of importance in proportioning lightweight concrete mixes.

The properties that are a function of tensile strength (shear, development length and modulus of elasticity) are sufficiently different from those of normal-weight concrete to require design modifications (Department of Energy Mines & Resources, 1979). The splitting-tensile strength of lightweight structural concrete is comparable with that of normal concrete of equal compressive strength if continuously moist cured, but if the lightweight concrete is air cured, e.g. after 7 days, the splitting-tensile strength may be about 25% lower. The same generalisation holds true for flexural strength, although the strength of air-cured lightweight concrete may be as much as 40% lower than that of normal-weight concrete. The modulus of elasticity of lightweight concretes is from 53 to 82% of the modulus of the normal concrete, i.e. 2.36×10^4 N/mm^2 at 28 days, and from 44 to 63% of the normal concrete, i.e. 3.45×10^4 N/mm^2 at 6 months. The modulus of elasticity is a function of density and compressive strength and can be determined within 15 to 20% by the formula: $E_c = W^{1.5} \times 0.043\sqrt{f'_c}$ where: E_c = static modulus of elasticity, N/mm^2, W = air-dry weight of concrete, kg/m^3, f'_c = compressive strength at time of test, N/mm^2. Depending on how critically the values of E_c will affect the nature of the design, the engineer should decide whether the values determined by the formula are sufficiently accurate or whether to determine E_c values from tests on the specified concrete.

Lightweight structural concrete has been used in a great variety of structures in many countries. The principal advantage of using lightweight aggregate concrete is its 20 to 30% lower density. Its use can result in: (a) lower foundation cost; (b) reduction in cross-sections of beams and columns; (c) lower reinforcing steel costs; and (d) in case of soil of limited bearing capacity, more storeys may be constructed. The Australia Square, Sydney, completed in 1967, is a circular tower, 50 storeys (184 m) high by 42.5 m in diameter. A 13% saving in construction costs was achieved through the use of 31 000 m^3 of lightweight concrete in the beams, columns and the floors above the seventh level. The concrete had an average compressive strength of 34 N/mm^2 and an average density of 1792 kg/m^3 at 28 days (Department of Energy, Mines & Resources, 1979). One Shell Plaza, Houston, Texas (1969) is an all-lightweight structure of 52 storeys (218 m). The concrete had a density of 1840 kg/m^3. Its 28-day compressive strength was 41 N/mm^2 for shear walls, columns and mat foundation, and 31 N/mm^2 for the floor structures. If normal concrete had been used, only a 35-storey structure could have been safely designed (Department of Energy, Mines & Resources, 1979). Lightweight aggregate concrete has also been used for offshore structures, notably reinforced concrete ships built during World Wars I and II (SS *Selma* oil tanker, Alabama in 1919 (Clarke, 1993)), oil platforms (Heidrun tension leg platform built in 1996) (ECSI, 2001), the gravity base tank built at Hunterston, south-west Scotland, for BP Oil (CEB-FIP, 2000), the Hibernia oil platform built for ExxonMobil Oil in 1998 (ACI Committee 213, 2004) and floating bridge pontoons (bridge near the city of Bergen, Norway (Solland *et al.*, 1993)).

Autoclaved aerated concrete (AAC)

In the early 1920s, Dr Axel Eriksson, then assistant professor for Building Techniques at the Royal Institute of Technology in Stockholm, invented what he called gas concrete. Later this new lightweight concrete was called autoclaved aerated concrete (AAC) or autoclaved cellular concrete and, more recently, porous concrete (RILEM, 1993). AAC is formed by a chemical reaction between

Figure 1 The Heidrun floating platform constructed out of lightweight aggregate concrete (courtesy of Elkem Materials Inc.)

finely divided calcareous and siliceous materials. The calcareous component is usually lime and/or cement. The siliceous component usually consists of natural or ground sand and/or industrial by-products such as slag and pulverised fuel ash (PFA). The cellular structure is achieved by either a chemical process causing aeration (e.g. 5–8% of aluminium powder will react with the calcium hydroxide and water to produce hydrogen bubbles and thus doubling the volume of the concrete), adding a preformed stable foam, or by introducing air voids by mechanical means into a slurry that contains no coarse material. Other processes may also be used. The slurry is usually placed into steel moulds and, after rising and setting, it is cut into the desired products while it is still soft. High-pressure steam curing is needed for the fine siliceous material to react chemically with the calcareous ingredients such as added lime or lime liberated by the hydration of cement. The reaction products are of a micro-crystalline structure with a much smaller surface area than that obtained by normal curing. The characteristic properties of AAC differ from those of other cementitious materials because of this basically different formation of the structure of the reaction products. Due to its porosity and comparatively low alkalinity, AAC does not provide the corrosion protection to the steel reinforcement afforded by dense concrete. The reinforcement should therefore be protected by a suitable surface treatment where it is needed. AAC can be subdivided into classes according to the characteristic strength, as shown in **Table 2** (Newman and Owens, 2003). AAC, which is about one-fourth of the weight of conventional concrete, is available in blocks, walls and roof panels, lintels, and floor slabs.

Property	Low	Medium	High
Compressive strength: N/mm^2	<1.8	1.8–4.0	>4.0
Modulus of elasticity: N/mm^2	<900	900–2500	>2500
Density: kg/m^3	200–400	300–600	500–1000
Thermal conductivity (dry): W/m K	<0.10	0.06–0.14	>0.12

Table 2 Classification of AAC according to characteristic compressive strength (adapted from Newman and Owens, 2003)

Foamed concrete

Foamed concrete is a highly workable, low-density material which can incorporate up to 50% entrained air. It is generally self-levelling, self-compacting and may be pumped. It is made by adding a special foam (foaming agent and foam generator are required) to a cement mortar slurry (ACI Committee 229, 2004). Available equipment ranges from portable foam generators to complete computer-controlled batching systems capable of batching and mixing the base materials with foam and producing the end foamed concrete product. Foamed concrete is ideal for filling redundant voids such as disused fuel tanks, sewer systems, pipelines and culverts – particularly where access is difficult. It is a recognised medium for the reinstatement of temporary road trenches (the filling of trenches dug in roads when pipes are laid or repairs are carried out). The traditional methods of filling trenches in the roads, i.e. the use of granular fill materials, result in settlement and damage to the road and, potentially, to the pipes. Foamed concrete assists in the spreading of the load so that axle loads are not transmitted directly to the services in the trench, so the pipes are not damaged by the weight of traffic. The foamed concrete is very fluid, it will fill any voids and cavities in the trench sides without requiring any compaction normally required for granular materials. Good thermal insulation properties make foamed concrete also suitable for sub-screeds and filling under-floor voids. Roofing is probably the most widespread application of foamed concrete. Foamed concrete has two benefits when it is used for roofing as it provides a high degree of thermal insulation and at the same time it may provide a slope for drainage, i.e. lay a flat roof to falls. Foamed concrete slopes on roofs, despite being much lighter than slopes made from mortar screeds, thus imposing a lower loading on the structure of the building, are strong enough to support foot or even vehicular traffic.

High-density concrete

High-density or heavyweight concrete has an oven-dry density greater than 2800 kg/m^3 and it is made with aggregate having a particle density appreciably greater than 3000 kg/m^3. High-density concrete can be used in counterweights of bascule and lift bridges, and underwater pipeline coverings (ACI Committee 228, 2004). These concretes are also used as specialist products for radiation shielding. It is not only their higher density that needs to be considered for the latter application; attenuation properties also need to be considered that may require special compositions to be used. The density is of prime importance for absorption of gamma rays. However, for attenuation of neutrons, sufficient material of light atomic weight, which produces hydrogen, should be included in the concrete mixture. Some aggregates may be preferred because of their ability to retain water of crystallisation at elevated temperatures and which can thus provide a source of hydrogen.

Various naturally occurring and man-made materials can be used as aggregates. Iron ores of various types include magnetite and haematite. Other minerals that can be used are limonite and goethite. Baryte is a naturally occurring ore, barium sulfate, found in veins, and is often associated with lead and zinc mineral ore deposits. Relative densities vary between 4.0 and 4.3 even if they are taken from different locations within a deposit. Synthetic aggregates include: (a) iron shot – chilled iron or steel available as shot or grit, normally with a maximum size of 8 mm; and (b) lead shot – available in a variety of sizes between 4 and 1 mm. Lead shot is reactive in alkaline conditions and requires special mix designs and cement systems, e.g. very high ground granulated blastfurnace content or high alumina cement (Miller, 2004). All these aggregates are considerably more expensive than conventional aggregates and, in general, the cost increases with density but not necessarily in direct proportion. Blending of two aggregate types may be considered to achieve the required density at a reduced cost.

Mix design procedures developed for normal-weight concrete may be used for high-density concrete but the somewhat non-standard gradings of the aggregates may have to be taken into consideration in order to produce cohesive mixes and avoid segregation and bleeding. Standard mixing equipment may be used for high-density concrete but special care should be taken not to overload it. The volume of high-density concrete to be mixed should be equivalent to the mix weight of normal-density concrete rather than the volume capacity of the mixing equipment. Transporting high-density concrete without agitation tends to cause excessive consolidation and packing. It can be placed using skips but allowance needs to be made for reduced volumes. Baryte and iron oxide mixes can be pumped over considerable horizontal and vertical distances. The concretes can be compacted with conventional poker vibrators although these will have a reduced 'radius of action' and therefore may require longer compaction times as the poker vibrators will need to be inserted at closer intervals (ACI Committee 304, 2004). Under no circumstances should poker vibrators be used to move concrete horizontally. Concrete layers should be reduced in thickness. High-density concrete should be cured in a similar way to normal-density concrete.

High-density concretes can be used for both structural and non-structural purposes. High compressive strength, high modulus of elasticity, but low coefficient of thermal expansion and low elastic and creep coefficients are desirable. Concretes with high-density aggregates can attain high strengths even at the same cement contents as normal-density concretes. Structural designers should establish values for the above mechanical properties for the particular mixes to be used. Shear strengths may be lower than compared with normal-weight aggregates of similar compressive strengths, and relationships between compressive and tensile strength may also not be valid.

High-strength concrete

The development of high-strength concrete has been gradual over many years and, as this development continued, its definition changed. In the 1950s, concrete with a characteristic compressive strength of 40 N/mm^2 was considered high strength in the USA. In the 1960s, concretes with 40 and 50 N/mm^2 characteristic strengths were used commercially. In the early 1970s, 60 N/mm^2 concrete was being produced. However, in recent years, engineers, particularly in North America, have been faced with increasing demands for improved efficiency and reduced concrete construction costs from developers and governmental agencies. As a result, they are now designing larger structures using higher-strength concrete at higher stress levels. The higher strengths have been possible as a result of developments in material technology (ACI Committee 363, 2004). The properties of the cement have improved during the years, especially concerning the early strength development. This has led to higher quality requirements in the codes. For instance, the 3-day mortar strength of Portland cement required in standards has increased from 15 N/mm^2 (1971) to 23 N/mm^2 (1978) and 25 N/mm^2 (1989). The introduction of condensed silica fume and superplasticisers during the 1970s also made it relatively easy to produce ready-mixed concrete with strength levels in the range of 100 N/mm^2.

Cost-effective production of high-strength concrete, defined as concrete with a minimum compressive cube strength of 60 N/mm^2, in construction today is achieved by using only readily available concrete-making materials but carefully selecting, controlling, and combining cement, chemical and mineral admixtures, aggregates and water. Conventional production techniques are used but with high quality control. Production of high-strength concrete, therefore, requires materials of the highest quality and their optimum proportioning. Mix proportions have often been selected empirically by extensive laboratory testing. The requirement of a low water/cement ratio has led to the use of high cement contents (up to 550 kg/m^3). Supplementary cementitious materials such as pulverised fuel ash and ground granulated blastfurnace slag, despite not producing exceptionally high 28-day strengths, are often cheaper than Portland cement and this factor, in combination with possible improvement in workability and long-term strength, and moderation of the heat evolution of the cement-rich mixes, tends to encourage their use. The use of high cement contents and superplasticisers can lead to extremely cohesive or sticky fresh mixes, and the slump test therefore may give the wrong impression of the workability. In order to be able to understand how fresh high-strength concrete will behave differently to traditional concrete and be able to optimise the mix design according to the requirements given by the methods of placement and compaction, there is a need for a fundamental knowledge of how to measure the workability, for example by expressing it in terms of classical rheological models such as those of Bingham or Newton.

Concrete properties such as modulus of elasticity, determined from stress–strain relationships, tensile strength, shear strength and bond strength are frequently predicted from empirical relationships with uniaxial compressive strength. These relationships have been based on experimental data of concretes with compressive strengths less than 40 N/mm^2. Research data have indicated that the modulus of elasticity of high-strength concrete can be overestimated while the modulus of rupture can be underestimated. However, extrapolation of the relationships to higher strengths for predicting values for Poisson's ratio and tensile splitting strength appear to be acceptable. Specific heat, diffusivity, thermal conductivity and coefficient of thermal expansion appear to be similar to normal-strength concretes. While shrinkage values are similar to normal-strength concretes, specific creep is much less for high-strength concretes.

Such high strengths were beyond the scope of national codes of practice and the Design Group of the Concrete Society examined and reviewed the scope for application of various design provisions to structures made with high-strength concrete. State-of-the-art design guidance for areas where existing provisions appeared inadequate or needed updating are given in Technical Report No. 49 (Concrete Society, 1999). The main structural applications of high-strength concrete have been in: (a) high-rise buildings, (b) bridges, and (c) offshore platforms.

High-rise buildings

The economic advantages of high-strength concrete are most readily realised when the concrete is used in the columns of high-rise buildings as has been the case in North America for many years. The advantages of using high strength concrete in these buildings include: (1) the ability to reduce the amount of costly reinforcing steel without sacrificing strength, (2) the number of storeys can be increased while maintaining an acceptable column size at the lower floor, (3) the reduction of the dimensions of columns and beams results in a decrease of the dead weight of the structure, which can substantially lessen the design requirements for the building's foundation, (4) the additional stiffness provided by high-strength concrete with its high modulus of elasticity reduces the amount of sway in the upper storeys of buildings, (5) the relatively high early concrete strengths allow early stripping of the formwork, thus accelerating the construction, and (6) it is mouldable to architectural advantage over steel.

Bridges

Long-span bridges are another area where the qualities of high-strength concrete are proving themselves economically attractive. For long spans, the weight of the structure can be a significant portion of the total weight carried by the bridge. High strength concrete's comparatively greater compressive strength per unit weight and unit volume allows increases in span capability, reduction of the girder depth, lighter and more slender bridge piers, and, therefore, reduces the load that has to be carried by the foundations. In addition, the increased stiffness of high-strength concrete is advantageous when deflections or stability govern the bridge design. The increased tensile strength of high strength concrete is helpful in service load design in prestressed concrete. The low creep of high-strength concrete should be taken into account when considering prestress losses. Since most of the prestress loss is attributable to creep and shrinkage, the losses for high-strength concrete members should be less than for lower-strength concrete members and this should be taken into account in the design.

Offshore concrete platforms

Concrete has been used successfully in marine structures since the early part of this century. It was first introduced on a large scale to the oil industry in 1971 when Phillips chose concrete for the Ekofisk oil production platform in the North Sea. It was located in water 70 m deep and contained 80 000 m^3 of normal-weight concrete with a specified 28-day compressive strength of 45 N/mm^2. In 1971, characteristic compressive strengths of 45 to 50 N/mm^2 were considered a near upper limit for site-produced concrete. Since then a total of 21 major structures containing over 2 million m^3 of concrete have been constructed for the North Sea oil fields, and specified strengths have risen to 70.0 N/mm^2. These very large structures were designed and built according to rigorous specifications and challenged the frontiers of developments in many technological fields including design processes, construction methods and materials technology. The critical demands due to high strength, durability, and constructability requirements, the magnitude of the structures, and the rate of construction have enforced the development of special cements and the use of supplementary cementitious materials, hydraulically processed sand, and refined superplasticisers and other chemical admixtures to achieve satisfactory concrete mix designs. A high topside payload capacity during tow (e.g. 43 000 t for Gullfaks A) and the possibility of oil storage within the platform have been considered as the primary merits of a concrete platform as compared to the traditional competitor, the steel jacket. These features have made the concrete platforms (like the Troll A platform shown in **Figure 2**) attractive and competitive for fields such as Statfjord and Gullfaks.

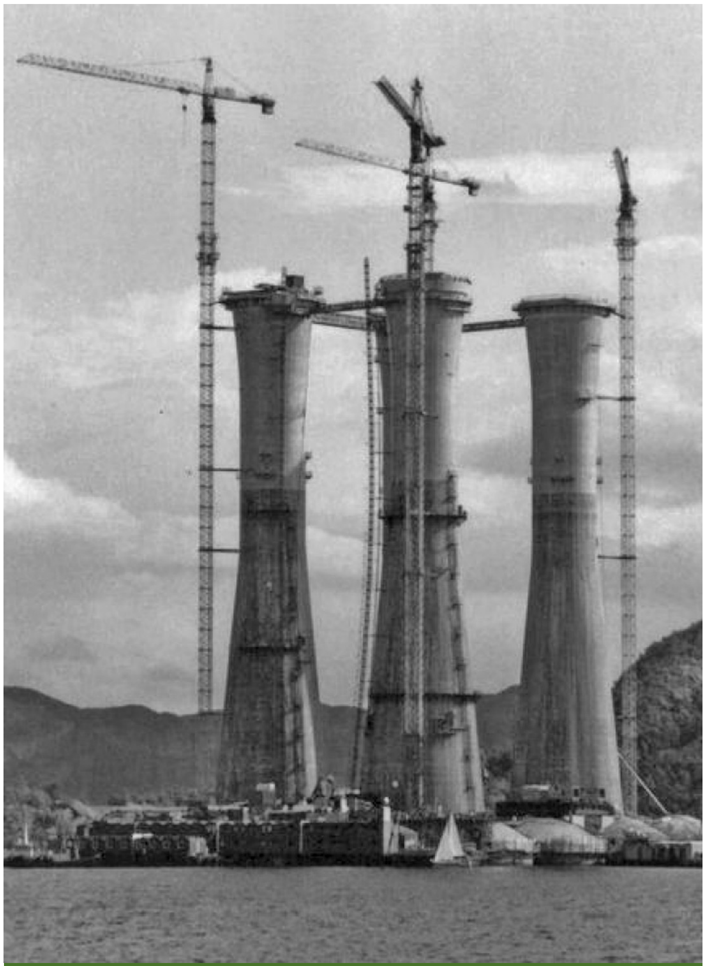

Figure 2 The Troll A platform was built over a period of 4 years, using a workforce of 2000. With a total height of 472 m, Troll A was the largest structure ever to be built and moved. The total weight when launching was 1.2 million tons. A total of 245 000 m^3 of concrete and 100 000 tons of steel for reinforcement were used. The amount of steel corresponds to 15 Eiffel towers. The platform was placed at a depth of 300 m. For stability, it is dug 35 m into the sea floor

High-performance concrete

High-strength concrete (HSC) was developed in the early 1980s and was followed by high-performance concrete (HPC). Increasing emphasis started being placed on durability rather than compressive strength. In many applications, HSC was and is being used because of its high durability rather than the need for the high compressive strength. The American Concrete Institute defines HPC as concrete that meets special performance and uniformity requirements that cannot always be obtained using conventional ingredients, normal mixing procedures and typical curing practices. These requirements may include the following enhancements: (a) ease of placement and consolidation without affecting strength, (b) long-term mechanical properties, (c) early high strength, (d) toughness, (e) volume stability, and (f) longer life in severe environments. Based on the results of a Highway Research Programme funded by the National Research Council (USA), the Federal Highway Administration developed a system for the classification of HPC, based on the level of performance requirements (Goodspeed *et al*., 2006). Such a classification enables design engineers to select appropriate performance criteria of HPC for different highway applications in different environmental conditions. The HPC definition resulted from an investigation of general field conditions that cause concrete structures to deteriorate. The proposed HPC definition uses eight parameters and relates each to deterioration resistance and gives standard tests to evaluate the performance of each parameter, see **Table 3** (Goodspeed *et al*., 2006). To use the definition as a basis for specifying concrete, relationships must be established between the performance parameter and resistance to exposure conditions. To accomplish this it is necessary to identify desired performance grades for the definition parameters and their relationship to project field conditions. Each parameter grade must represent a measure of performance when subjected to a field condition. Using grades to represent performance, an engineer can specify a mixture to yield a desired concrete service life. Performance is represented by test variables such as the percentage of dynamic modulus of elasticity remaining after 300 prescribed cycles of freezing and thawing or a range of compressive strengths. Grades start at low performance levels and small enough increments are defined to allow engineers to begin specifying higher-quality concrete incrementally. The strength grades start at a performance level that is easily attainable. Each parameter can be independently specified by grade: for example, a mixture for a bridge deck subjected to high usage of de-icing salts, high frequency of freezing and thawing cycles, and narrow beam spacing may be specified by a high ‘grade’ to resist freezing and thawing distress, a

Performance characteristic	Standard test method	FHWA HPC performance grade				
		1	2	3	4	N/A
Freeze/thaw durability ($x =$ relative dynamic modulus of elasticity after 300 cycles)	AASHTO T 161 ASTM C 666Proc. A	$60\% \leqslant x \leqslant 80\%$	$80\% \leqslant x$			
Scaling resistance ($x =$ visual rating of the surface after 50 cycles)	ASTM C 672	$x = 4.5$	$x = 2.3$	$x = 0.1$		
Abrasion resistance[6] ($x =$ avg. depth of wear in mm)	ASTM C 944	$2.0 > x \geqslant 1.0$	$1.0 > x \geqslant 0.5$	$0.5 > x$		
Chloride permeability ($x =$ coulombs)	AASHTO T 277 ASTM C 1202	$3000 \geqslant x > 2000$	$2000 \geqslant x > 800$	$800 \geqslant x$		
Strength ($x =$ compressive strength)	AASHTO T 22 ASTM C39	$41 \geqslant x < 55$ MPa ($6 \geqslant x < 8$ ksi)	$55 \geqslant x < 69$ MPa ($8 \geqslant x < 10$ ksi)	$69 \geqslant x < 97$ MPa ($10 \geqslant x < 14$ ksi)	$x \geqslant 97$ MPa ($x \geqslant 14$ ksi)	
Elasticity ($x =$ modulus of elasticity)	ASTM C 469	$24 \leqslant x < 40$ GPa ($4 \leqslant x < 6 \times 106$ psi)	$40 \leqslant x50$ GPa ($6 \leqslant x < 7.5 \times 106$ psi)	$x \geqslant 50$ GPa ($x \geqslant 7.5 \times 106$ psi)		
Shrinkage ($x =$ microstrain)	ASTM C 157	$800 > x \geqslant 600$	$600 > x \geqslant 400$	$400 > x$		
Creep ($x =$ microstrain/pressure unit)	ASTM C 512	( )	( )	( )	( )	

Table 3 Grades of performance characteristics for high-performance structural concrete (Goodspeed *et al*., 2006)

medium to high 'grade' to resist scaling, abrasion, and chloride attack and a low 'grade' for strength and elasticity. The definition is intended to cover all 'grades' of concrete that can be readily used by the highway industry.

Examples of the use of HPC in offshore structures are the platforms completed between 1993 and 1995, i.e. Troll A and Heidrun, the design life of which was extended to 50–70 years compared to only 30 years of previous platforms. HPC is being extensively used for the fabrication of precast pylons, piers, and girders of many long-span bridges in the world, e.g. the Normandie Bridge in France (1993), the East Bridge of the Great Belt link in Denmark – 100 years design life (1994), see **Figure 3** and the Confederation Bridge in Canada (1997).

Figure 3 The East Bridge of the Great Belt Link in Denmark was designed for a life span of 100 years

Fibre-reinforced concrete

The idea of reinforcing relatively brittle building materials with fibres has been known and practised since ancient times. Mud huts constructed from baked clay reinforced with straw, and masonry mortar reinforced with animal hair, are typical examples dating well back into history. Fibres are extremely effective in inhibiting the rapid and unlimited propagation of cracks associated with failure in plain mortar or concrete (Johnston, 2000). Their presence causes crack propagation to develop slowly and progressively with increase in load. They therefore enable stress to be transferred across a cracked section, allowing the affected part of the composite to retain some post-crack strength and to withstand deformations much larger than could be sustained by the unreinforced matrix alone. Under normal static or slow loading, the result of both the aforementioned characteristics is a dramatic increase in the area under the load–deformation curves, see **Figure 4**. The property characterised by this area is commonly referred to as 'toughness'. Under conditions of impact loading, the work of fracture or impact resistance is similarly improved. Under conditions of rapid repeated cyclic loading, the progressive propagation of cracks is again effectively inhibited by fibres, and considerable improvement in fatigue behaviour results. Under conditions of explosive loading, which is basically a form of impact loading, fibres are particularly beneficial in reducing both the amount of fragmentation and the fragment velocities. There are four main types of fibres for use in concrete, i.e. asbestos, glass, polymeric and carbon-steel fibres.

Asbestos fibres

Unlike manufactured fibres, asbestos fibres occur naturally in bundles which require separation, and the fibre length distribution and aspect ratio are quite variable. The maximum length is about 10 mm, but most are considerably shorter, less than 5 mm. The use of asbestos fibres in cement was becoming widespread, especially for fire-resistant boards, but its association with cancer and respiratory diseases has stopped its use.

Glass fibres

Alkali-resistant glass, containing zirconium oxide, although not immune to alkali attack, is quite stable in cement over relatively long periods of time. Glass fibres are inherently brittle and very prone to damage by abrasion, which drastically reduces their strength. As a result of these properties, the use of glass fibres has been restricted to their use in a cement paste matrix where the manufacturing process involves simultaneous spraying of cement slurry and chopped strand to form thin sheet-like products similar to asbestos cement. The 2D-fibre orientation and the relatively high fibre concentration made possible by this process results in high strength, typically 15–20 N/mm^2 in tension and 30–50 N/mm^2 in flexure at a fibre concentration of 6% by volume. The product known as glass-reinforced cement (GRC) is finding a wide range of applications.

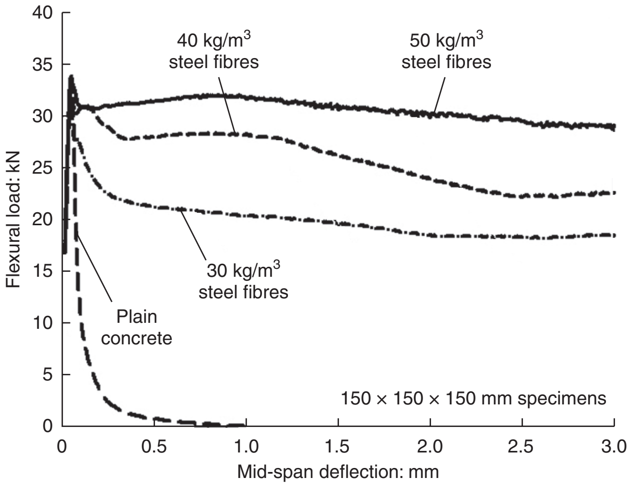

Figure 4 Stress–deflection curves for fibre-reinforced concrete with different dosages of steel fibres

Polymeric fibres

These may be divided broadly into two types: (a) micro fibres, e.g. polypropylene fibres, which mainly influence the early age properties, e.g. shrinkage, and can also improve the fire resistance of concretes, and (b) macrofibres, which can provide the concrete with post-cracking capacity. Experimentation with macro-synthetic fibres dates back to the 1960s but they have only been commercially and readily available much more recently (described by suppliers as 'structural synthetic fibres'). These fibres have dimensions that are similar to those of steel fibres. Dosages can be as high as 12 kg/m^3 compared to 0.9 kg/m^3 for polypropylene fibres.

Steel fibres

Steel fibres are classified into five groups according to the method of manufacture: (1) cold-drawn wire; (2) cut sheet; (3) melt extract; (4) shaved cold drawn wire; and (5) milled from blocks. The problem of efficiently utilising fibres is to maximise fibre pull-out resistance without causing fibre 'balling' during mixing which results when high aspect ratio fibres of uniform cross-section are used. Types of steel fibre therefore aim to improve the pull-out resistance through their shape, i.e. indented, etched, with roughened surface, round with end paddles, round with end buttons, round with hooked ends, crimped (round, flat or any section) and polygonal twisted (Concrete Society, 2007a). A further refinement to minimise the fibre balling which can result during introduction of a large number of individual fibres to the concrete mix is to supply fibres in bundles held together with water-soluble glue which gradually disappears during the mixing process allowing the individual fibres to become uniformly distributed throughout the mix. Fibre characteristics that affect the mechanical properties of concrete can be divided into physical and intrinsic categories. The physical characteristics are: aspect ratio, shape and surface texture, and absolute size. The intrinsic fibre characteristics are: tensile strength, elongation at failure, elastic modulus, interfacial bond shear strength, Poisson's ratio, and elasticity.

Fibres are often used to replace the nominal conventional steel fabric in ground-bearing slabs (Concrete Society, 2003). Increasingly, steel fibres are being used in suspended ground-floor slabs on piles to replace much, and in many cases all, of the reinforcement. Savings in the cost of

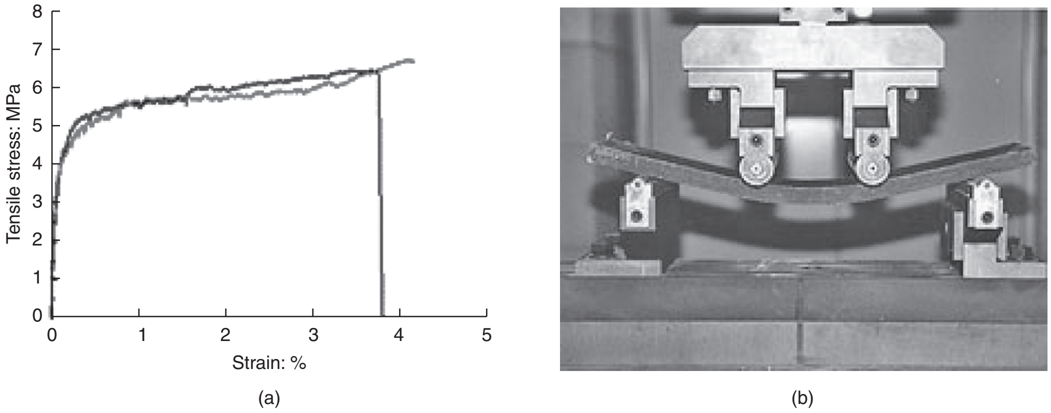

Figure 5 (a) Tensile stress–strain curve of an ECC material with 2% fibres; (b) flexural testing of ECC specimen showing ductile behaviour (courtesy of Professor Victor Li, University of Michigan, 2005)

supplying and fixing the conventional welded fabric reinforcement that is replaced can offset the extra cost of adding fibres to the concrete. Other major applications for steel-fibre-reinforced concrete include external paved areas, sprayed concrete, composite slabs on steel decking and precast elements.

Engineered cementitious composites

An engineered cementitious composite (ECC), or otherwise known as a high-performance fibre-reinforced cementitious composite (HPFRC), is an ultra-ductile mortar-based composite with short random fibres (usually polyvinyl, 2% by volume). ECC, unlike common fibre-reinforced concrete, has tensile strain-hardening behaviour with strain capacity in the range 3 to 7%, see **Figure 5(a)**. The ductile behaviour of ECC under flexural load, see **Figure 5(b)**, is achieved through multiple microcracks, usually no more than 60 µm in crack width, developing along the length of the specimen. ECC, due to its ductility, energy dissipation and damage tolerance when subjected to cyclic loading, has potential applications in: (a) structural elements for improved performance during earthquakes; and (b) bridge decks for link slabs, removing the need for joints, but also as a thin composite ECC/steel deck for longer service life through a tight crack width control (Li, 2005).

Ultra-high-performance fibre-reinforced concrete

Ultra-high-performance fibre-reinforced concrete (UHPFRC) has a characteristic compressive strength in excess of 150 N/mm^2, possibly attaining 250 N/mm^2, and contains steel or ‘structural synthetic fibres’ in order to achieve ductile behaviour in flexure or direct tension (Association Française de Genie Civil, 2002). Commercially available products include: (a) compact reinforced composites (CRC) developed by Aalborg Portland, Denmark; (b) BSI developed by Eiffage group, France; (c) reactive powder concrete (RPC) developed by Bouygues, France; (d) multi-scale fibre-reinforced concrete (MSFRC) developed by Laboratoire Central des Ponts et Chaussées, France (Rossi, 2001); and (e) Ductal® developed by Lafarge, Bouygues and Rhodia, France. A typical composition of UHPFRC is a very low water/cementitious ratio ranging from 0.16 to 0.24 achieved by using high cement content (955 kg/m^3) together with a high silica fume content (240 kg/m^3) and a high dosage of a superplasticiser (15 litres/m^3). The only ‘aggregate’ used is fine (150–400 µm) quartz sand (1050 kg/m^3). A high percentage by volume (2.5 to 10% by volume) of a special type of steel fibre (24 or 13 mm length and 0.20 mm in diameter) is used. Combinations of short and long fibres in ‘cocktails’ incorporating polypropylene fibres have also been used. The basic principles in the selection of the above materials for the production of these UHPFRCs are:

- Enhancement of homogeneity by elimination of coarse aggregate.
- Enhancement of compacted density by optimisation of the granular mixture, i.e. the reason for the high silica fume content and the use of fine quartz sand as the only aggregate.
- Optional enhancement of the microstructure by post-set heat-treating.
- Enhancement of ductility by incorporating small-sized steel fibres.
- Maintaining mixing and casting procedures as close as possible to existing practice for normal- and high-strength concretes.

Application of the first three principles produces a matrix with very high compressive strength, but with ductility no better than that of conventional mortar. The inclusion of fibres improves tensile strength, and also makes it possible to obtain a high level of ductility. Measures relating to composition (homogeneity and granular compacted density) are the basis of the UHPFRC concept, and are applied in all cases. Application of post-set heat-curing appears to be an optional measure designed to enhance performance. The advisability of applying these measures must be assessed for each UHPFRC application, according to the technological difficulties involved (application of pressure) and/or their cost (heat treatment). The compressive strengths of UHPFRCs are likely to be between 170 to 230 N/mm^2 depending on the post-set heat treatment (20 to 90°C). Values for flexural strengths are likely to be between 30 and 60 N/mm^2, fracture energies between 50 000 and 60 000 J.m^{-2} (measured as the area under the stress–deflection curve up to a deflection of 2 mm for $100 \times 100 \times 350$ mm specimens) and moduli of elasticity between 50 to 60 GPa. UHPFRC is a promising new material not only because of its enhanced ductility but also because the mixing and casting procedures are no different to existing procedures in use for normal- and high-strength concretes. UHPFRC has, however, a substantial increase in cost over and above that of conventional and even high-performance concrete and it is therefore appropriate to identify applications which fully utilise its mechanical properties and performance characteristics.

Applications of UHPFRC to date have included: construction of prestressed concrete structures without any secondary steel reinforcement. The Sherbrooke Footbridge in Canada (see **Figure 6**) was the first structure to be built with RPC (Adeline *et al*., 1998) after a suitable mix was developed by the University of Sherbrooke (Canada). The use of RPC in structures is inhibited by the lack of design rules allowing full advantage to be taken of the improved mechanical characteristics of the material and of its ability to be used without secondary reinforcement. Following a performance assessment, BSI (another type of UHPFRC) has been adopted by the French Ministry of Transport and the Roads Department for construction of two raised sections of motorway close to the city of Valence in the Drome Region (Adeline *et al*., 1998). The article that appeared in the *Proceedings of the Institution of Civil Engineers* (2002) describing this material has attracted a lot of interest from UK industrial companies according to the French Technology Press Bureau. Pipe products for the conveyance of water, sewage and other liquids under pressure or gravity flow provide an opportunity to utilise many of the enhanced properties of UHPFRC (Dowd and Dauriac, 1998). The US Army Corps of Engineers has developed (1994–7) pipe prototypes which exhibit greater overall value than pipes fabricated from other materials. The mechanical properties of UHPFRC also make it attractive for the construction of security enclosures such as safes and computer centres, nuclear waste containment vessels, defence structures, and other applications that require high impact resistance.

Sprayed concrete

Sprayed concrete is a mortar or concrete pneumatically projected at high velocity from a nozzle to produce a dense homogeneous mass (Taylor, 2003a). The force of the jet by which the concrete is projected compacts it despite that it normally has zero slump. The concrete stays on the projected surface without sagging despite that, usually, applications may involve vertical, sloping or even overhead surfaces. Sprayed concrete normally incorporates admixtures and may also include steel or polypropylene fibres. 'Gunite' was the name given to this material by Carl Akeley, who developed a double-chambered cement gun in 1910. The term 'shotcrete' to describe this material was introduced in the USA in the early 1930s.

Sprayed concrete is usually classified according to the process used, i.e. wet or dry-mix (ACI Committee 506, 2004). Cement, aggregate and water are batched and mixed prior to being fed into a purpose-made machine. A piston or a worm pump conveys the materials through a pipeline to a nozzle where compressed air is introduced to project the concrete onto the required surface. The dry process only requires the cement and aggregate to be dry mixed as the water is introduced at the nozzle. The materials are conveyed through the pipeline using compressed air. The dry process is capable of producing high-quality concrete but can be more variable, has higher material losses and is a more dusty process. The in-situ water/cement ratio for dry-mix sprayed concrete is normally in the range of 0.30–0.50 and 0.40–0.50 for the wet-mix process. Twenty-eight-day compressive strengths reported are normally in the range of 20–60 N/mm^2 for cement contents of 350–450 kg/m^3.

Sprayed concrete offers advantages over conventional concrete, in a variety of new construction and repair work; it allows complex shapes and structures to be formed without the high costs associated with formwork. It is therefore used for structural repairs, fire protection to steel-framed structures, for tunnel and refractory linings, swimming pools, river walls, domes and shell structures.

Heat-resistant and refractory hydraulically bound concretes

Heat-resistant concrete will not disintegrate when exposed to constant or cyclical heat, up to temperatures at which a ceramic bond is formed, i.e. around 1000°C. Refractory concretes are suitable for use at temperatures as high as 1900°C. Both types of concrete are usually made with high-alumina cement. Examples of non-hydraulic setting binders include waterglass (sodium silicate), phosphoric acid and phosphates. Portland cement is not recommended for use at temperatures higher than 300°C although cases have been reported where, with appropriate selection of materials, it behaved satisfactorily up till 1000°C. The hydrated lime or Portlandite, i.e. $Ca(OH)_2$, which is present in hydrated Portland cement, will dehydrate at around 500°C to form quicklime, i.e. CaO. Exposure to moisture, even atmospheric moisture, will lead to rehydration which is an expansive reaction and will crack the concrete (Montgomery, 2003). Hydrated lime is not formed during the hydration of calcium aluminate cements and this is the reason for their suitability for heat-resistant and refractory concretes. The service temperature limit of calcium aluminate cements is governed by their alumina content, i.e. 36 to 42% of Al_2O_3 for the 'low' grey range, 48 to 60% for the 'middle' buff/brown range, 65 to 75% for the 'high' white range. Free alumina may be added to the last group to increase the percentage to around 80%.

Figure 6 The Sherbrooke Footbridge in Sherbrooke, Quebec, spans 60 m across the Magog River with a precast truss made of reactive low quality powder concrete. The deck slab is 30 mm thick and 3.3 m wide. There are stiffening ribs, 100 mm wide by 70 mm deep, on the two edges of the beam and transverse ones every 1.25 m. These stiffening ribs had internal prestressing for purposes other than load capacity

Underwater concrete

Placing concrete under water can be difficult. The concrete must be cohesive (to minimise cement washout), of flowing consistency and possess self-consolidating characteristics (it is impractical and undesirable to use mechanical vibration under water). Recent developments in materials, especially chemical admixtures, and placement techniques, have allowed the successful placement of concrete in thinner and more complex structural sections, with or without reinforcement, than was previously possible (Taylor, 2003b).

The aim, when placing concrete underwater, has been to keep as much as possible of the concrete out of contact with the water and to avoid any rapid movement or agitation at the exposed surfaces. Tremie concrete is placed under water, one of several placing techniques, using a steel tube, called a tremie pipe, suspended vertically in the water with a hopper (*tremie* in French) fixed to the top end to receive fresh concrete. The hopper also acts as a reservoir to ensure a steady flow down the pipe despite an intermittent supply of concrete. Successful tremie placement relies on keeping the concrete inside the pipe separate from the water. Various methods have been used for providing a barrier between the water and the concrete until the tremie pipe gets filled with concrete, e.g. travelling plug of exfoliated vermiculite or even tennis balls. The concrete pushes the floating plug down and out of the tremie pipe. Once placement starts, the mouth of the tremie pipe must remain embedded in the concrete, otherwise the concrete within the tremie pipe will suddenly discharge and the empty tube will fill up with water. Concrete flow is achieved only by gravity and there may therefore be the need to lift the end of the pipe to maintain the flow. Tremie concrete is best suited for large-volume placements where the tremie pipe does not need to be relocated frequently. Pumped concrete, on the other hand, relies on the pressure of the pump to reach its final position. The discharge hose, which is flexible, can therefore be buried in the placed concrete and large flat areas can be concreted by moving it horizontally, without removing it from the concrete. Pumped concrete is discharged in surges which can result in more cement washout and laitance formation than tremie concrete. However, pumping of concrete has the advantage of eliminating the need for tremie placement equipment (ACI Committee 546, 2004).

Free dump through water is the placement of freshly mixed 'non-dispersible' concrete by allowing it to fall through water without the benefit of confinement such as tremie pipe or pump line (Concrete Society, 1990). Anti-washout admixtures, usually containing ingredients such as high molecular weight polymers, superplasticisers, cellulose derivatives and gums, should be used. These may increase considerably the cost of concrete. Conflicting evidence exists on the quality of the in-place concrete cast with this method. The free-fall height should be limited to that required for opening the bucket so as to reduce the velocity impact with the water surface. Free-fall concrete is most suited in shallow water applications. Other methods of placing concrete under water include: (a) preplaced (grouted) aggregate concrete is a specialist technique requiring a colloidal grout to be injected through vibrated or rodded aggregate placed in a form; (b) placing with a hydrovalve which is similar to the tremie pipe but has a flexible collapsible pipe instead and a rigid bottom section; and (c) placing with a specially designed skip that has bottom opening double doors.

Innovative uses of underwater concrete have resulted in significant benefits not only in cost but also in risk reduction. Such applications have been in the construction of Richmond-San Rafael Bridge which crosses the San Francisco Bay, in the rebuilding of the erosion-damaged stilling basin of the Lower Monumental Dam in Washington, in retrofitting of the North Rankin ‘A’ offshore platform in Australia, and construction of the Braddock Dam in Pittsburgh (Yao and Gerwick, 2004).

Mass concrete

Mass concrete is any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change to minimise cracking (ACI Committee 207, 2004a). Massive structures, such as gravity dams, resist loads by virtue of their shape and mass and therefore compressive strength becomes a secondary rather than a primary concern. Water-reducing, strength-enhancing and set-controlling chemical admixtures, as well as the use of ground granulated blastfurnace slag and pulverised fuel ash, have been shown to be effective in reducing the required cement content to a minimum as well as controlling the time of setting. The cement content of the concrete used for the construction of the Pine Flat Dam in California was only 140 kg/m^3. Additional temperature control measures, in addition to keeping the cement content to a minimum, can include a relatively low initial placing temperature, circulation of chilled water through cooling pipes and, at times, insulation of concrete surfaces as may be required to adjust for these various concrete conditions and exposures (Bamforth, 2003). It should be emphasised that mass concrete is not necessarily large aggregate concrete and that concern about generation of an excessive amount of heat in concrete is not confined to massive dams. Indeed, cracks in the structural concrete of foundations, bridges, tunnel linings and other medium-sized elements have become an increasing problem in the past decades. The heat of hydration has been established as the main cause of restraint stresses leading to cracks in unreinforced as well as reinforced concrete. Although the processes of heat generation and dissipation are familiar to engineers, theoretical prediction of the temperature changes and effects has not been easy before the advent of the computer, and reliance on the experience obtained from previous construction was the alternative. This has led to the recommendation that, as a rule of thumb, the differential should be kept below 20°C to avoid cracking. However, worries have been expressed at the tendency to generalise in terms of overall temperature differentials, forgetting that thermal cracking is a stress concentration problem and temperature gradients are more important. The magnitude of thermal stresses in concrete will increase with an increase in the modulus of elasticity, and a reduction in creep will prevent adequate stress relief. Cracking can be controlled, i.e. limiting the maximum crack spacing and the crack width, by the use of reinforcement and guidance is given in CIRIA Report C660 (Bamforth, 2007) and by ACI Committee 207 (2004b). Computers can be used, where limiting cracking is very important, to look at the many ramifications of the heat problem, and also used to determine the thermal gradients permissible in structural elements made from normal- and high-strength concretes with different binders. Computer programs will, however, have to be based on the properties of materials, the heat of hydration development, the increase of stiffness and the decrease of relaxation capacity, the increasing tensile strength and the coefficient of thermal expansion. All these factors depend largely on age, temperature, cement type and concrete mix composition.

Polymer-modified concrete

Polymer-modified concrete or latex-modified concrete is defined as ‘Portland cement combined at the time of mixing with organic polymers that are dispersed or redispersed in water, with or without aggregates’. The polymer types that are in common use today are: styrene-butadiene copolymers, acrylic ester homopolymers and copolymers with styrene, vinyl acetate copolymers, and vinyl acetate homopolymers. Cement hydration occurs first and the mixture sets and hardens. The polymer particles, concentrated in the void spaces, coalesce into a polymer film, as the water is consumed by cement hydration, coating the aggregate particles and lining the interstitial voids (ACI Committee 548.3, 2004). The improvements from adding polymers to Portland cement concrete include increased bond strength, freeze–thaw resistance, abrasion resistance, flexural and tensile strengths but reduced permeability and elastic modulus. Uses of these materials include tile adhesive and grout, floor-levelling concrete, concrete repair materials and bridge deck overlays. Over 8000 bridge decks in the US have been overlaid with PMC either as part of the original construction or as repair of a deteriorated deck.

Recycled aggregate concrete

In 1991, the European Commission initiated the Priority Waste Streams Programme for six waste streams. One of these was construction and demolition waste (C&DW). The inert fraction or 'core' C&DW, which is essentially the mix of materials obtained when an item of civil engineering infrastructure is demolished, i.e. the fraction derived from concrete, bricks and tiles, is well suited to being crushed and recycled as a substitute for newly quarried (primary) aggregates for many potential uses. Government aims are to reduce demand for primary aggregates by minimising the waste of construction materials and maximising the use that is made of alternatives to primary aggregates. The environmental costs associated with quarrying are being addressed by the aggregates levy which was introduced in April 2002 and which is likely to increase in the future. Although there are many potential uses for C&DW-derived aggregates, the majority is currently used for low-value purposes such as road sub-base construction, engineering fill or landfill engineering. Provisions for their use in higher grade, and therefore high-value, applications such as structural concrete have only recently been included in national standards. BS 8500-2: 2006 'Specification for Constituent Materials and Concrete' only covers the use of recycled coarse concrete aggregates (masonry content <5%), not recycled masonry aggregate, and their use is restricted to the less severe environments. Detailed requirements, based on specified test methods, for recycled masonry aggregates (RA), such as acid-soluble sulfate, chloride content, alkali content and potential for alkali–aggregate reaction, are restricting their use in concrete. The restriction does not relate to mechanical properties; research indicated that up to 30% coarse or 20% fine RCA had no effect on the strength of concrete. There is a concern that most of the gypsum plaster used on internal surfaces would accumulate in the fine recycled aggregate; it has been proven that, gypsum plaster, in amounts greater than the maximum permissible sulfate content, can lead to delayed ettringite formation.

Successful use of recycled aggregate concrete has been reported for major, but nonetheless 'demonstration' projects. One of the biggest problems with using more recycled aggregate is its availability at the right time and at the right place. It therefore has greater potential if it is used for precast concrete products such as building blocks, paving blocks and concrete pavement slabs (Soutsos *et al*., 2005).

Self-compacting concrete (SCC)

Self-compacting concrete (SCC) can be placed without requiring compaction by vibration. It therefore should be able to flow under its own weight, completely filling formwork and achieving full compaction, even in the presence of congested reinforcement (EFNARC, 2005). SCC was developed in Japan in the late 1980s as an alternative to traditional vibrated concrete in order to (a) address the lack of highly skilled concrete placement workers and (b) attempt to address durability problems arising from concrete that was not properly compacted. An added benefit from the use of SCC is the improved working conditions resulting from a reduction of workers' exposure to noise and vibration. Casting times are also shorter, since compaction is not required, and this may lead to increased productivity.

SCC must not only be able to flow under its own weight but must do so without segregation. It is generally accepted (EFNARAC, 2005) that an SCC can be classified according to the following properties: (a) flowability/filling ability; (b) viscosity/flowability; (c) passing ability; and (d) segregation resistance. No single test is capable of assessing all of the key properties that an SCC should possess. Therefore, a combination of tests is required to fully characterise an SCC mix. Test methods developed specifically for assessing the fresh properties of SCC are not currently described in EN 12350: Testing fresh concrete, but are described in 'The European Guidelines for Self-compacting Concrete' (EFNARC, 2005) and in 'Self-compacting Concrete' by De Schutter *et al*. (2008). The key properties required for a concrete to be classified as an SCC can be achieved with a wide range of mix proportions. SCC mix proportions may differ from those of traditional vibrated concrete in that there is a lower coarse aggregate content, increased paste content, low water/powder ratio, increased superplasticiser dosage and sometimes the mix may contain a viscosity-modifying admixture. Typical ranges by mass are as follows: 380 to 600 kg/m^3 of powder (cement and additions), 150 to 210 kg/m^3 of free water, 750 to 1000 kg/m^3 of coarse aggregate, and the fine aggregate content balances the volume of the other constituents and is typically between 48 and 55% of the total aggregate weight (EFNARC, 2005).

The use of SCC in the precast concrete industry is growing faster than its use for in-situ casting of structural elements (De Schutter *et al*., 2008). Not only do factory conditions provide better quality control but the benefits from its use, i.e. reduced noise and vibration while achieving increased productivity, can easily be appreciated. Notable uses of SCC have been in structural walls (gas tanks in Texas, USA and the industrial hall in Veurne, Belgium (De Schutter *et al*., 2008)), in slabs and beams (residential buildings in Brazil (De Schutter *et al*., 2008)), in foundation slabs (Trump Tower, Chicago, USA and the World Financial Centre, Shanghai, China (De Schutter *et al*., 2008)), and anchor blocks for bridges (Akashi-Kaikyo Bridge near Kobe, Japan (De Schutter *et al*., 2008)).

Future trends

The world is pouring around 5 billion tonnes of concrete a year – nearly 1 tonne per person per year – and therefore concrete is probably the most common material in modern construction (Kerman, 2003). Concrete research is continuing to develop and assess new concrete materials, in order to enhance the constructability and durability of Portland cement concretes. The potential economic benefits of applied research in Portland cement concrete are huge because so much of it is used.

Sustainability of the construction industry is currently being examined and ways of improving its credentials are being sought. Ways of recycling materials either in concrete products or in the production of cement are being investigated. The production of Portland cement itself contributes an estimated 5% of the annual global CO_2 emissions (Gartner, 2004). The Kyoto Protocol on Climate Change (1997) commits countries to limiting the emissions of global warming gases and this will profoundly affect the activities and finances of the cement industry. Values of around 0.8 t of CO_2 emissions per tonne of cement produced have been quoted (World Business Council for Sustainable Development, 2002). It is not therefore surprising that the major cement manufacturers have got together and have agreed an agenda for action for *The Cement Sustainability Initiative* (World Business Council for Sustainable Development, 2002). Although cement makes up on average 15% of a concrete, it accounts for 80% of the energy consumption and 66% of fuel demand required in concrete production. Efforts to achieve sustainability in the concrete industry must therefore focus on the cement component of concrete. Urbanisation of countries such as China and India is expected to lead to a total worldwide cement demand of 2 billion tonnes by 2010. This projected increase in cement demand could be met, without any increase in Portland cement production (Malhotra and Mehta, 2002), by the use of materials such as pulverised fuel ash and ground granulated blastfurnace slag as partial cement replacements. The CIA/DETR-funded project on 'Defining and Improving Environmental Performance in the Concrete Industry' concluded that if pulverised fuel ash and granulated blastfurnace slag provided 18% of the cementitious material then there would be significant reductions of 25% in fossil energy use, 2.5% in electrical energy use and 17% in CO_2 emissions (Parrott *et al*., 2000).

Pulverised fuel ash and granulated blastfurnace slag also react with alkaline solutions to produce a dense, compact paste with cementing properties which has been investigated mainly for hazardous waste immobilisation. Several reaction models have been suggested. Low-calcium PFA pastes activated with a solution containing sodium hydroxide and sodium silicate and cured at 65°C have been produced with compressive strengths of 60 N/mm^2 after 24 h (Palomo *et al*., 1999). In general, the chemical reaction of these systems is fast and there is no significant strength gain after 24 h. The higher temperatures needed make this material ideal for precast concrete applications where higher curing temperatures are routinely used to enhance early strength of concrete. Although the mechanism of these reactions has been well established for some pozzolans, considerable further investigation is required to advance this technology to a point where it can be exploited by UK companies and applied at factory scale to produce environmentally friendly and commercially viable alternatives to Portland cement concrete, i.e. cementless 'geopolymer' concretes.

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Further reading

There is generally no shortage of information on special concretes and institutes/societies such as the following are the best places to start one's literature review:

American Concrete Institute (USA)
http://www.concrete.org/general/home.asp

The Concrete Society (UK)
http://www.concrete.org.uk/default.asp?submenu = home

Fédération internationale du béton (Switzerland)
http://www.fib-international.org/

US Department of Transportation – Federal Highway Agency
http://www.fhwa.dot.gov/index.html

RILEM – International Union of Laboratories and Experts in Construction Materials
http://www.rilem.net/

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Chapter 19

Concrete mix design methods

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CONTENTS

The intention of concrete mix design is not only to choose suitably its ingredients, i.e. cement, aggregate, water and admixtures, but also to determine their relative proportions so as to produce concrete which is economical, strong, workable and durable. In routine and small-scale construction projects, concrete mixes may be chosen based on past experience, but when the use of different types of cementitious materials and different types of concretes are required for projects, especially to satisfy differing performance criteria (of both fresh and hardened concretes), more sophisticated techniques (including statistical approaches) may be needed to arrive at suitable mix proportions. The intention of this chapter is to demonstrate the different types of mix designs so that appropriate methods can be identified for different practical situations. However, it must be recognised that mix design is not an absolute science, but an art to satisfy a set of design criteria, by making use of the most appropriate materials for producing concretes which are economical.

Introduction

Concrete is primarily a composite material comprising aggregates of different types and sizes and cementitious paste. The paste binds the aggregates together and when it hardens, concrete becomes a rocklike material. The paste itself is made of cementitious materials, such as Portland cement, pulverised fuel ash, ground granulated blastfurnace slag, microsilica and metakaolin, water and chemical and mineral admixtures. In addition, there will be air either entrapped during mixing or intentionally added as in the case of air-entrained concretes. The properties of the hardened mass are governed to a large extent by the properties of the materials used to manufacture the paste. The aggregate is divided into two types, coarse and fine, and the main difference between them is in their size fractions. Normally fine aggregate size ranges from 150 microns (10^{-6} m) to 4.75 mm and the coarse aggregate ranges from size fractions above 4.75 mm, and the nominal maximum sizes of the coarse aggregate are 10 mm, 20 mm and 40 mm. Some small percentages of size fractions other than those specified are normally allowed. As shown in **Figure 1**, in a unit volume of concrete the volume of paste varies from 7% to 15%, water between 16% to 21%, air content in air-entrained concrete between 4% and 8% and the rest is aggregate (i.e. 60 to 75% of the total volume of concrete is occupied by aggregates) (Kosmatka *et al*., 2003).

Although the term 'mix design' is commonly used for describing the process that enables a decision on the relative quantity of concrete ingredients to be used to satisfy a specified set of design criteria, the term 'mixture proportioning' is sometimes used, as in North America, to describe this process. When a new material is used to replace part of the material in **Figure 1**, or to add a new ingredient such as chemical admixtures to the mix which has been designed for satisfying a certain set of design criteria, the yield equation (Neville, 1996) is normally used to adjust the mix proportion and to yield a unit volume of concrete. In this case, the use of the term mix proportioning is fully justified.

With sustainability and economy as two important design criteria, in addition to performance criteria of both fresh and hardened concretes, there is a desire to combine numerous cementitious materials (such as pulverised fuel ash (PFA), ground granulated blastfurnace slag (GGBS), microsilica and metakaolin), fine aggregates (such as natural sand, furnace bottom ash, crushed rock fines and pulverised fuel ash), coarse aggregate (such as natural gravel, crushed rock, blastfurnace slag and both natural and manufactured lightweight aggregates) and a range of chemical and mineral admixtures for the production of concrete (Mehta and Monteiro, 2005). That is, the concrete technologist now has the option (and considerable scope) to optimise concrete mixes appropriate to specification requirements and the intended uses of the concrete by proportioning the materials available in a number of different ways.

The purpose of the mix design is to obtain proportions of concrete ingredients which will satisfy certain predetermined requirements, i.e.:

- the workability of the fresh concrete (ease with which a concrete can be placed, compacted and finished);
- strength of the hardened concrete (compressive strength and tensile strength are key properties);

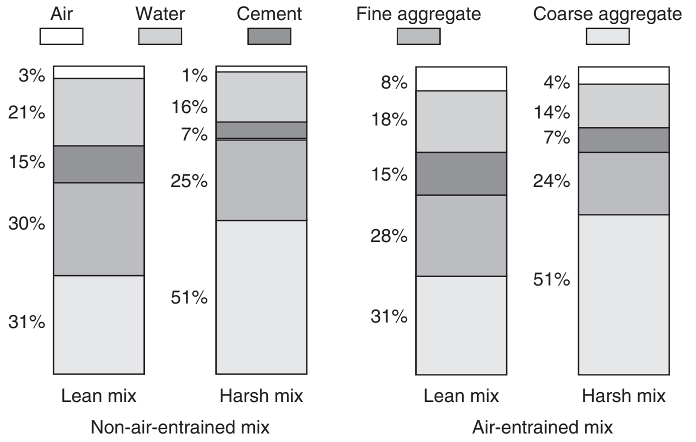

Figure 1 Range of relative volumetric proportions of coarse aggregate, fine aggregate, cement, water and air in a cubic metre of concrete (adapted from Kosmatka *et al*., 2003)

- durability of the concrete in the service environment (resistance to cracking, interactions with the service environment that led to chemical and/or physical processes of deterioration, etc.);
- short-term and medium-term dimensional stability (shrinkage and creep).

The above requirements need to be satisfied at the lowest possible cost. As the cost of cement is much higher than that of aggregates, every effort should be made to reduce the cement content and increase the aggregate content in concretes. If cheaper substitute materials for both cement and aggregates are available, they need to be incorporated without compromising other design considerations.

In other chapters on concrete, specifications to be used for accepting ingredients of concrete are given. This chapter will focus on how these ingredients can be proportioned to meet the requirements for different mix properties and types of concretes. An appreciation of the precise contribution of each of the ingredients is a starting point of any mix design and, hence, first a brief review of the role of each ingredient is given. Furthermore, significant contributions made to the development of the mix design procedures are briefly summarised.

Role of concrete ingredients

Although there is some debate on the precise role of Portland cement content on the durability of concrete, it is now generally accepted that too much Portland cement is unlikely to yield durable concretes. Similarly, the use of supplementary cementitious materials is considered to be beneficial for improving the long-term strength and the durability, but in some cases chemical admixtures are needed to improve the fluidity (workability) of the concrete, as in the case of concretes containing microsilica.

For a given set of materials and particular curing condition, the use of too much water is known to reduce not only the strength but also the durability. Water needs to be kept to a minimum for many reasons (Kosmatka *et al*., 2003):

- to improve the compressive and the flexural strength
- to lower the transport properties, thus lower absorption, diffusion and permeability
- to improve resistance to weathering
- to enhance bond between concrete and reinforcement
- to reduce the drying shrinkage and cracking
- to minimise the volume change from wetting and drying.

If concrete becomes harsh to place and compact due to the reduced water content, chemical admixtures could be used to improve its workability. Chemical admixtures and cement additives are required for manufacturing special concretes, such as self-compacting concrete.

The aggregate should be sound so that deleterious reactions can be avoided. The water demand of concrete mix decreases with increase in aggregate size and a proper grading of different sizes of aggregates is needed for yielding workable and durable concretes. Furthermore, both natural sand and gravel aggregate are known to reduce the water demand of the concrete compared to crushed fine and coarse aggregates. However, the tortuosity of flow path in concrete increases with the use of crushed aggregates and hence these are likely to yield better durability.

Significant contributions in the development of concrete mix design procedures

According to Monteiro and Helene (1994), the overall evolution of mix design techniques can be divided into four broad categories:

1. development of basic technology for cement, mortar and concrete;
2. development of classical methods of mix design;
3. investigation into the use of statistical methods;
4. further developments in concrete technology and improvements to the practical mix design techniques.

Rather than providing a historical review of the developments in mix design procedures based on the above listing, some of the most significant contributions are discussed in greater detail in this section so as to build up the concept in mix design from a more basic level.

Development of basic technology for cement, mortar and concrete

Much of the technology and ideas generated during the initial period still form the basis of many of the current mix design techniques in use today. One topic investigated quite extensively during this period was discontinuous

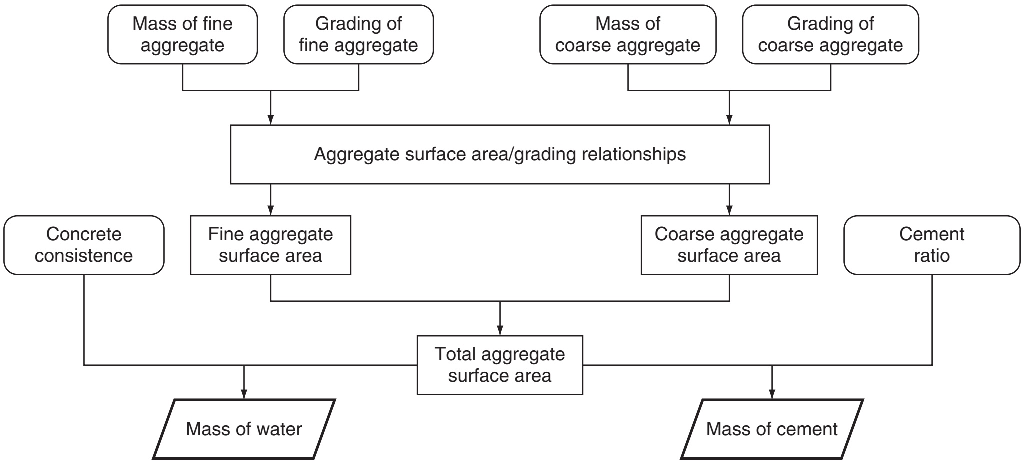

Figure 2 Flowchart of Edwards' surface area of aggregates mix design procedure (adapted from Edwards, 1918)

granulometry. In 1881 Preadeau reported the importance of discontinuous granulometry in terms of the volume of voids in the aggregate (Monteiro and Helene, 1994). From his investigations he suggested that in an 'ideal' concrete the volume of cement paste should be 5% higher than the volume of voids in the fine aggregate and the volume of mortar should be 10% higher than the volume of voids in the coarse aggregate. This approach was later expanded upon by Lecleroc du Sablon in 1927 (see Monteiro and Helene, 1994).

Fuller in 1901 made a very significant contribution when he established the effect of granulometry on compressive strength (Monteiro and Helene, 1994). The main findings from his work were the establishment of 'ideal' aggregate grading curves that ensured the concrete produced had maximum density (maximum compressive strength). At the time, the main criticism of the Fuller curves was that the concrete produced was harsh and unworkable. To try to correct this, Bolomey in 1925 (Monteiro and Helene, 1994) studied the effect of Fuller's curves on workability and, after doing this, modified the curves to include the cement content.

Although discontinuous granulometry had been used quite extensively, the two most significant contributions during this period were undoubtedly the correlation between concrete compressive strength and water/cement ratio w/c reported by Abrams (1919, 1922) and the relationship between water content and the consistency of fresh concrete reported by Lyse in 1931 (Monteiro and Helene, 1994). The significance of these two relationships will become clearer as some of the significant individual contributions are discussed.

Edwards' surface area of aggregates

The fundamental theory behind this technique is that all the particles in the mortar or concrete matrix should be made to act together as a single unit (Edwards, 1918). To achieve this, Edwards concluded that, as cement was the only 'strength-producing' constituent in the mix, sufficient cement should be available to thoroughly coat each aggregate particle. A simple flowchart of the overall procedure is shown in **Figure 2**. The starting point for Edwards' method was to establish a technique for determining the surface area of aggregates of different types and grading. To do this, Edwards assumed that all fine aggregate particles and gravel coarse aggregate particles were spherical in shape and broken stone coarse aggregate particles were made up from one-third cubical and two-thirds parallelepipedal shapes to approximate more closely the areas off the elongated and slab-like shapes of crushed particles. From these assumptions, Edwards developed a series of graphs which can be used in conjunction with the aggregate gradings to determine an approximate surface area value for any combination of aggregates.

The next step in the process is to assume the required ratio between the mass of cement and the surface area of the aggregates. After a series of trial mixes Edwards concluded that a typical range for this would be 1 gram of cement to 10–20 square inches (6450–12 900 square millimetres) of aggregate surface area. Using this assumed value it is easy to calculate the mass of cement required. The mass of water required is calculated from an empirical relationship linking the mass of cement and surface area of the aggregates to the consistency of the fresh concrete. When all of the parameters have been calculated, all the masses of the individual constituents are available.

British approach of standard mixes

At one time it was common practice to specify concrete as one part cement, two parts fine aggregate and four parts coarse aggregate (1:2:4) or 1:1:2 when stronger concrete was required (Day, 2006). The main drawback with this

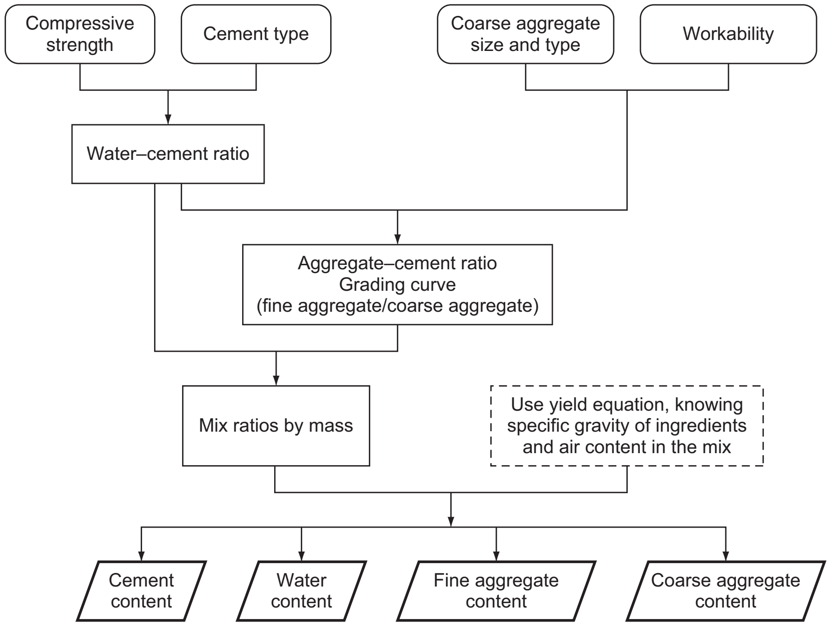

Figure 3 Flowchart of Road Note 4 mix design procedure (adapted from HMSO, 1958)

approach was that these particular mixes could have very different fresh and hardened properties depending on the grading of the fine aggregate used. To control this, a grading specification was published in 1944 (BS 882: 1944) and two grading envelopes were available: class A and class B. The specification suggested that if class A fine aggregate was proportioned in the ratio 1:2:4 then 'good' concrete was produced and if class B fine aggregate was used in the same proportions then 'reasonable' but not 'good' concrete was produced.

Newman and Teychenne (1954) then demonstrated that equally good concrete could be produced from class B fine aggregate provided the relative proportion of fine to coarse aggregate was adjusted accordingly. They proposed the division of fine aggregate into four grading zones instead of two classes (BS 882: 1955). Fine aggregate as a percentage of total aggregate was to range from 40% with the coarsest (Zone 1) to 22% with the finest (Zone 4). Zone 2 at 33% was the old 1:2 ratio (i.e. 1:2:4 and 1:1:2 mixes) and Zone 3 was to contain 25% fine aggregate. This general approach continued into the Road Note 4 (HMSO, 1958) and DoE/BRE methods (Teycheene *et al*., 1975).

Road Note 4 method

For many years, in the 1950s, 60s and beyond, Road Note 4 (HMSO, 1958) was the accepted UK mix design procedure. It was prepared from information obtained during an investigation into the effect of aggregate grading on concrete strength and workability (Glanville *et al*., 1947). Its main purpose was to allow mix proportions to be chosen so as to produce concrete with specified fresh and hardened properties. A flowchart of the overall procedure is given in **Figure 3**.

One important aspect of the Road Note 4 method is in its procedure to combine the fine and coarse aggregates. This is decided based on the individual gradings of the aggregates to be used and the overall aggregate grading curve chosen. One point that is highlighted quite forcibly in Road Note 4 is that the four grading curves were not intended to be considered as 'ideal' grading curves. This is in agreement with the work reported by Newman and Teychenne (1954) which demonstrated that 'good' concrete could be produced with 'inferior' fine aggregate provided the ratio between the fine and coarse aggregates was correct. To deal with these situations, Road Note 4 makes the recommendation that all designed mixes need to be checked by completing trial mixes.

Mix design procedures for strength, workability and durability using Portland cement

DoE/BRE method

The first edition of this method was published in 1975 (Teychenne *et al*., 1975), revised in 1988 (Teychenne *et al*., 1988) and the current second edition published in 1997 (Marsh, 1997). It is restricted to designing concrete mixes to meet workability, compressive strength and durability

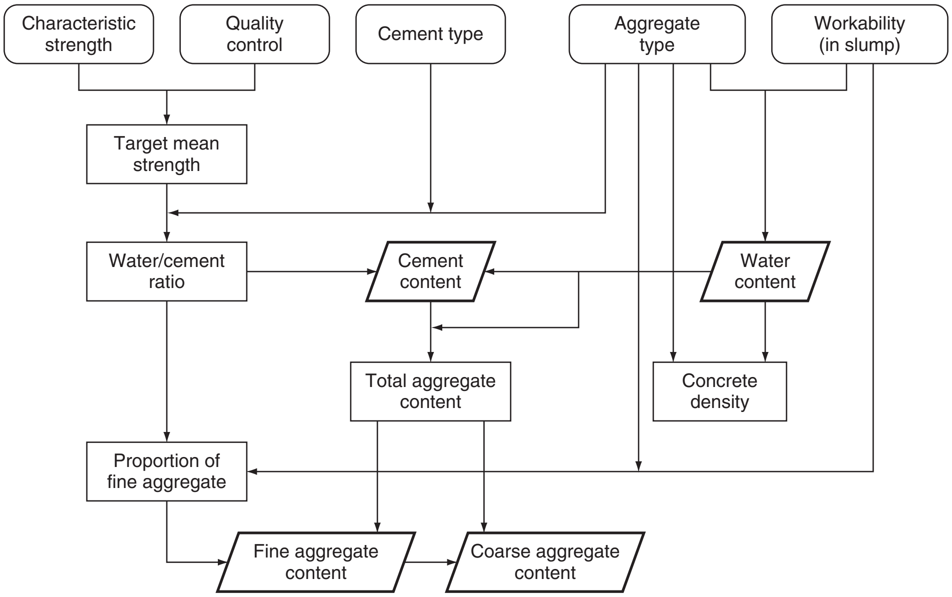

Figure 4 Flowchart of DoE/BRE mix design procedure (adapted from Teychenne *et al*., 1988)

requirements. The method was developed primarily for designing concrete mixes containing Portland cements and natural aggregates, but guidance is also given on the application of the method to mixes incorporating pfa, ggbs and air entrainment. A diagram of the overall mix design procedure for normal concrete is presented in **Figure 4**.

The starting point for the procedure is to specify the characteristic strength (please refer to Marsh, 1997 for a detailed description of statistical concepts used to determine the characteristic strength of concrete) and the workability (in terms of slump) required and the type of cement and aggregates used. From the characteristic strength the target mean strength is calculated. This calculation is based essentially on the level of quality control likely to be achieved.

The next step is to determine the w/c ratio required. The factor that has the greatest influence on the w/c ratio is the specified compressive strength. The graph relationship that is used to determine the required w/c ratio is essentially Abrams' law, modified slightly to take account of aggregate/cement ratio, aggregate and cement type and concrete age.

The next step in the procedure is to determine the required water content. This depends primarily on the specified workability, but the maximum size of aggregate and the type of aggregate can also influence the workability. The required water content is simply read from a table that takes account of these factors. After determining the water content, the cement content is calculated by dividing the water content by the w/c ratio.

The fresh concrete density is the next parameter to be determined. The graph relationship used for this is essentially the yield equation presented in a slightly different format. The information required to determine the concrete density is the water content and the specific gravity of the aggregates. From the concrete density, the total aggregate content is obtained by subtracting the cement and water contents.

The final step in the procedure is to proportion the fine and coarse aggregates. The main influencing factor here is the percentage of the fine aggregate passing a 600 μm sieve determined from the fine aggregate grading curve. The specified workability, the maximum aggregate size and the w/c ratio also influence the required proportion of the fine and coarse aggregates. Combining all this information, the percentage of fine aggregate required is determined. After determining the fine aggregate content, the coarse aggregate content is calculated by simply subtracting the fine aggregate content from the total aggregate content.

Another very important feature of this method is that throughout the method provision is made to allow a durable concrete mix to be achieved. As has been reported in many publications, a major factor in providing durable concrete is the production of a dense impermeable concrete which is fully compacted and properly cured. To achieve this, many codes and standards specify limits on the cement content or w/c ratio to be used. Provision is made in this mix design procedure to override the cement content and w/c ratio values obtained from the specified strength and workability should these calculated values be less stringent than those obtained from durability requirements.

The final step in this procedure is to produce a trial mix to confirm that the calculated mix proportions produce a concrete with the required fresh and hardened properties.

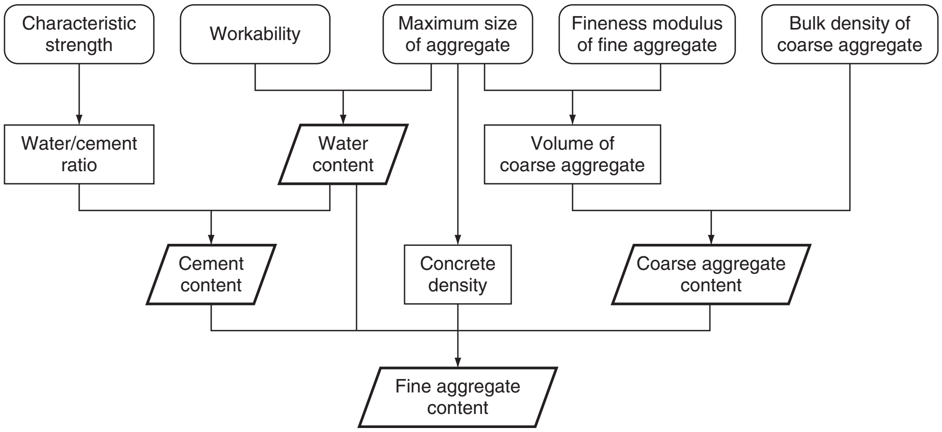

Figure 5 Flowchart of the ACI 211 mix design procedure (adapted from ACI 211, 2002)

If the required properties are achieved then the mix design process is complete; however, if not all of the required properties are achieved then a systematic approach is available for adjusting the mix proportions to achieve the required properties. A full description of the adjustment procedure is presented by Marsh (1997).

American Concrete Institute (ACI) method

This mix design procedure is probably the most widely used throughout the world. The first version was published in 1944 and the most recent version in 2002 (ACI 211, 2002). It allows mix design for air- and non-air-entrained concrete and concrete containing supplementary cementitious materials. A flowchart of the overall procedure for normal non-air-entrained concrete is given in **Figure 5**.

Similar to the previous mix design procedures discussed, the starting point is to specify the required compressive strength and workability and the type of aggregates used. Using a simple table of relationships, from the workability specified and the maximum size of aggregate to be used, the water content is determined. This approach is essentially the same as that used in the DoE/BRE method. The w/c ratio required is determined from the specified compressive strength (Abrams' law). The cement content is calculated by dividing the water content by the w/c ratio.

The next step in the process is to determine the coarse aggregate content. The technique used to do this is to proportion the fine and coarse aggregates independently of the specified strength or workability in such a way that 'particle interference' and 'volume of voids' are kept to a minimum. To do this, the dry-rodded volume of the coarse aggregate per unit volume of concrete is used. This is influenced by the maximum size of aggregate and the grading of the fine aggregate (fineness modulus). The actual coarse aggregate content is determined by multiplying the dry-rodded volume of coarse aggregate with the bulk density of the coarse aggregate. The use of the bulk density is quite clever because, in one number, this allows for the combined effect of grading, specific gravity, void volume and particle shape of the coarse aggregate on the desirable fine aggregate content.

An estimate of the fresh concrete density for different coarse aggregate sizes is the next parameter to be determined. This is then used to calculate the fine aggregate content by subtracting the water, cement and coarse aggregate contents. Similar to most of the other concrete mix design procedures, the facility is also available throughout the procedure to override the w/c ratio and cement content determined from the specified compressive strength and workability, should these be less stringent than those determined from durability considerations.

Mix design used by the UK ready-mixed concrete industry

The Quality Scheme for Ready Mixed Concrete (QSRMC, 1995) is a product certification system for ready-mixed concrete. It operates through the application of formal regulations to all the quality-related processes associated with the design, production and supply of ready-mixed concrete. Compliance with the regulations is enforced through assessment of the producer's systems and records. QSRMC-certified companies are licensed to use the QSRMC Certification Mark on quotations and delivery dockets, confirming to their customers that they have achieved the highest level of product conformity certification possible. The part of the regulations discussed here will be limited to the mix design procedure (QSRMC, 1995).

The QSRMC mix design procedure is not like any of the previous procedures discussed. The main difference is that from the regulations issued by QSRMC it is not possible to determine the mix proportions required to achieve a certain compressive strength and workability. All that is described in the regulations is a technique that allows ready-mixed concrete suppliers to establish their own compressive strength and workability relationships for the materials they plan to use. The easiest way to explain this is by discussing the actual graphs and relationships established.

For most civil engineering jobs, workability is usually specified in bands or groups, for example low, medium or high. As highlighted previously, the factor that has the greatest influence on workability is the water content. Therefore, the first step in the QSRMC method is to determine, either by trial mixes or from past experience, the water content required for each level of workability. To ensure that the mixes are cohesive and not prone to segregation, the volume ratio between the total fines (cement plus fine aggregate) and the coarse aggregate should also be studied (Barber, 1995).

The factor that has the greatest influence on compressive strength is the water/cement ratio. Therefore, for each workability level a ‘family’ of mixes is produced in which the water content remains constant (workability constant), but the cement content increases, causing the water/cement ratio to decrease. Whenever the cement content increases, the fine aggregate content decreases to ensure that the volume ratio between the total fines and the coarse aggregate remains constant. The maximum increment in cement content allowed between mixes is 50 kg/m^3 (Barber, 1995). Whenever these mixes are cast, the workability and compressive strength are measured. This allows the relationship between compressive strength and cement content to be established for each workability level.

Whenever a customer approaches a ready-mixed concrete supplier, the two requirements that are usually specified are workability and compressive strength. The workability requirement determines the water content or family of mixes to be used and, using the strength relationship established, the compressive strength decides the cement content. In view of the fact that the designed mix proportions are based on the actual materials that are going to be used, there is no need to complete trial mixes to adjust the proportions. The maximum water/cement ratio and minimum cement content durability requirements can also be incorporated just as in the previous methods discussed.

Computer programs for mix design

Dewar’s particle interference and void filling

Many of the original concepts and ideas reported 30 or 40 years ago are now being revamped and taken onto a new level largely as a result of computer technology. One of the main objectives of a method introduced by Hughes (1960) was to proportion the mixes in such a way that ‘particle interference’ and ‘volume of voids’ are kept to a minimum. Trying to achieve both of these simultaneously can be quite difficult. For example, if the main objective is minimum volume of voids then the most obvious way to achieve this is to arrange for smaller particles to fill the voids between larger particles. However, this then causes problems with particle interference. Therefore, a very fine balance needs to be struck between both approaches.

To try to overcome these problems, Dewar (1986) has developed a comprehensive theory and associated mathematical/computer optimisation model of particle mixtures that has been validated for powders, aggregates, mortar and concrete. The basis of the theory is an idea from Powers (1968). When particles of two different sizes are mixed together, the smaller particle filling the voids between the larger particles is interfered by the packing of both sizes. The mathematical model developed by Dewar has been computerised by Questjay Ltd and SP computing in the UK. The name of the mix design system is *Mixsim* and the software is described in www.mixsim.net (2008). With this software, as many as three cementitious components, three fine aggregates or fillers and three coarse aggregates along with three liquid admixtures can be used in the mix design. The use of both air-entrainment and plasticisers can be incorporated. Main input parameters for each of the solid components are particle density, mean size and voids ratio.

Computerised mix proportioning techniques by Day

The computer program developed by Day is now produced commercially and sold under the name, Conad (Day, 2006). Although this is now a very sophisticated mix design and quality control software package, Day (2006) emphasises quite strongly that he has used a simple manual version of this for more than 30 years.

Free web-based mix design programs called *KensMix* and *KensQC* developed by Day are available from the website http://www.kenday.id.au (2008). The input starts with entering the aggregate grading of both fine aggregate (FA) and coarse aggregate (CA). The cost and specific gravity (SG) of the aggregate also are to be entered. The specific surface (SS) is calculated by the system. If more than one FA or CA is used they can be combined. For combining the FA and CA, the user has to decide the proportion and the program uses it as one aggregate. Regarding cementitious materials, a strength factor (SF), SG and cost have to be entered. A suitable SF for fly-ash is 0.5–0.9 and for microsilica will be 3–4 when 1 is for Portland cement. Another factor to enter is ‘equivalent cement’ for workability purposes and is not the same as that for strength. At this stage water content is estimated by the program. After this, slump, temperature, air content and silt content are to be entered. Regarding cement quantity, the user has to enter the range, say 300–350 kg and the amount of water content change per 10 kg more or less

MSF	Slump range:		Remarks
	mm	in.	
16			Unusable, too harsh
16–20			Harsh mixes, only suitable for zero slump concrete under heavy vibrations
20–22	0–50	0–2	Hard-wearing floor slabs, precast products under good external vibration
22–25	50–90	2–3.5	Good structural concrete
25–27	80–100	3–4	Good pumpable concrete. Fine surface finish. Heavy reinforced sections
26–28	90–120	4–5	Pumpable lightweight concrete
27–31	200+	8+	Flowing superplasticised concrete

Table 1 Mix suitability factor (MSF) values (reprinted from K. Day, Concrete Mix Design, Quality Control and Specification ©2006 Taylor & Francis)

than the range (say 1 to 2 litres/10 kg). In the case of pozzolanic effect, the expected reduction or increase in water requirement due to the presence of pozzolan is to be input. The final entry, 'water factor', accounts for the effect of admixture, effect of FA and shape. Another value to be known is the mix suitability factor (MSF), which represents the degree of sandiness and is derived from the specific surface (SS) of the combined aggregate (**Table 1**). The predicted strength is dependent on both the strength factor and strength factors of individual cementitious materials.

Automix is another program which uses specific surface to obtain total sand to total coarse aggregate ratio (Kay, 2006). It is based on ideal gradings. FA with ideal grading is the one with its grading normally distributed on a logarithmic scale. For a required mean size the user can select a percentage passing a 75 mm sieve or nominate standard deviation or co-efficient of variation. Any of these entries will calculate a family of normally distributed gradings, one for each mean size. Or it is possible to use old UK sand grading zones. The program, when keying in 'calc', is able to cycle through each curve and every integer combination of the two sands, with these alternate sets of criteria, to find which gives the closest match to one of the curves. Regarding coarse aggregate, the user has to select from continuous, semi-continuous, semi-gap or gap. The user has the option to choose an alternative combination from that suggested by the program. At the next stage of designing the mix, from the MSF value, the desired type of concrete is specified. Again the user has the freedom to choose any MSF number and the descriptions in the table can be edited. Approximate values of slump, air content and concrete temperature are to be input after this. The suggested value for water factor is 0.95. However, the user can overwrite this based on his/her own experience. The program assumes the water requirement for a limited range of cement content, which the user can specify (say 300–350 kg). Now only the steps 'calculate water' and 'calculate' are remaining for the program to proportion the mix.

Mixtable is another program which has been introduced by Day (2006). For this program, the user has to input the CA ratio and FA ratio or import data from *Automix* (Day, 2006). Other inputs include MSF, slump, temperature and air content; also percentage or amount of fly-ash, microsilica or slag has to be specified. The output gives batch quantities, density and the compressive strength.

Rational mix maintenance program covers key mix design criteria and factors (Day, 2006). The input parameters include raw material data, cementitious efficiency and blends/tapers (taking Portland cement efficiency as 1.0). All blends/tapers are input as control and related blends. Also the user needs to input admixtures, yield chart, and control mixes adjustment factors (which include water requirement, coarse aggregate volume and leanness factors). Furthermore, common relational mixes, discrete relational mixes, ratio mixes (which are mixes designed based on ratios or percentages, such as sand/cement mixes, no fines mixes, etc.), discrete manual mixes and plant configurations are to be input. The other parameters required are grade strengths and plant relationships. This program has a sophisticated section to undertake the checking of the proposed mix with the previous mixes uploaded.

Mix design based on strength, workability and durability

The mix design procedures presented in the previous sections are procedures based on compressive strength and workability. None of them allow direct mix design for durability. In some methods, a restriction on the minimum cement content and/or maximum w/c ratio is used to ensure durable concrete mixes. However, these approaches do not necessarily ensure durable concrete at all times.

During the structural design, the durability design takes place when a cover is decided based on the exposure classification and a particular concrete class is read off one of the tables without a great deal of consideration given to the multitude of exposure regimes for which structures need to be designed. For several years many researchers questioned this approach, particularly the reliability of the simple broad-based exposure classes. Finally, in EN 206, 23 different exposure classes were introduced to address this issue (BS EN 206-1, 2001) and Hobbs (1998) suggested minimum requirements of concrete for withstanding these exposure regimes. While partial factors and statistical approaches are used to deal with the variation in compressive strength of concrete between the cube test and that which is achievable in the structure, for durability performance the only check that is applied even now is that the correct cover, w/c ratio and cement content have been used. No durability tests are generally completed to assess the compliance of concrete. This is quite surprising from the point of view of a client taking up ownership of a structure and not really knowing the level of durability achieved. As a result of these problems, clearly a new approach is required for durability standards and specifications.

Several researchers have used transport properties to classify concrete durability (Figg, 1973; Levitt, 1985; Ithuralde, 1992; Basheer *et al*., 1994; Dhir *et al*., 1994). A preliminary attempt to develop a design procedure was made by Basheer *et al*. (1998) based on air permeability, compressive strength and workability, but this needs to be developed further because it is known that a unique transport property cannot be used to specify the durability of concrete in all exposure regimes. More research is required with various cement types and transport processes to develop such a mix design procedure further. The concept of 'performance-based specification' is now an area of worldwide interest and hopefully when results of these studies are available, concrete could be designed for durability as it is done currently for strength and workability.

Mix design for special concretes

The American Concrete Institute's Committee 211 developed many reports describing procedures which can be used to design: (i) no-slump concrete (ACI 211.3R-02, 2002c), (ii) heavyweight and mass concrete (ACI 211.1-91, 2002a), (iii) high-strength concrete containing supplementary cementitious materials (ACI 211.4R-08, 2008) and (iv) structural lightweight concrete (ACI 211.2-98, 2002b). Both in ACI and RILEM, technical committees are currently working on developing design procedures for self-compacting concrete and draft documents and tentative procedures are available from both ACI and RILEM. Design features of some of these special concretes are summarised in this section and readers are requested to consult suggestions provided at the end of this chapter on further reading for detailed procedures for the design of special concretes.

Mix design for high-strength concrete containing pulverised fuel ash (or fly ash)

The fly ash content in a mix normally varies between 15% and 35% by mass of cementitious material. There are two types of fly ash available, i.e. Class F and Class C. Class C fly ash has high rate of hydration at early age compared to Class F fly ash. Recommended use of Class F fly ash is 15–25% by mass whereas for Class C fly ash it is 20–35%, but the use of larger quantities of Class C fly ash has been reported in the literature.

ACI 211.4R-08 (2008) describes the method of designing high-strength concrete containing fly ash. Tables are given in this document for guidance for choosing different components of the mix. High-range water-reducing admixtures (HRWRA) are recommended for these concretes so that the amount of water added can be reduced without losing the workability and hence produce stronger concretes. Once the slump, coarse aggregate size and fractional volume of oven-dry rodded coarse aggregate (VCA) are chosen from the tables, the required mass of coarse aggregate per cubic volume is determined.

Water content is then calculated from tables giving relationship between slump, coarse aggregate size, air content and water content. If HRWRA is added in to the mix, this also should be taken into consideration while deciding the water/cementitious material (w/cm) ratio. The void content of the fine aggregate is then calculated.

From the water content calculated already and from the table showing the relationship between w/cm, compressive strength and coarse aggregate size, for mixes with and without HRWRA, the mass of cementitious materials required per cubic unit of concrete is calculated. The proportion of fly ash in the mix is selected at this stage. The difference between the total cementitious material and the fly ash content will give the cement content in the mix. Once coarse aggregate, water, percentage air content, cement and fly ash contents are decided, the fine aggregate content is calculated for unit volume of concrete using absolute volume method.

The fly ash properties and aggregate properties vary depending on the source and hence it is advised to do trial mixes after the mix proportioning and if required the mix proportions need to be adjusted.

Mix design for high-strength concrete containing microsilica (or silica fume)

As in the case of fly ash high-strength concrete, previous experience of the users is advised to be taken as guidance for a starting point for mix design. A 5% to 15% silica fume by mass of cement is used normally in mixes with w/cm as low as 0.2. Tables are available in ACI 211.4R-08 (2008) for proportioning mixes with silica fume as well. The bulk specific gravity of coarse aggregate and fine aggregate are also required for this calculation. HRWRA is advised to be added into the mix to maintain the workability because the addition of the silica fume into the mix will increase the water requirement. This is because of the fact that particles of silica fume are 100 times less in size than Portland cement. It is also recommended to do trial mixes and adjust the proportions according to the result in addition to doing a real size test mix before actually using the mix in large scale.

Mix design for high-strength concrete containing ground granulated blastfurnace slag (or simply slag)

The proportion of slag used usually in mixes is 40% to 50% by mass of the cementitious material, but higher additions (up to 85%) have been reported in the literature. The workability normally does not reduce when slag is added to the mix. The finer the slag the better the early-age strength gain of the mix. Slightly higher dosages (up to about even 20% for the first trial mix) of the air-entrainment admixture needs to be added to obtain the required air content. All the steps required to design high-strength concrete containing slag are given in ACI 211.4R-08 (2008). The increase in compressive strength due to the addition of certain percentage of the slag is given in graphical form in the document.

Mix design for structural lightweight concretes

Two methods of proportioning structural lightweight concrete are given in ACI 211.2-98 (2002b). One of them is the *weight method*, which uses a specific gravity factor, determined by the pycnometer test with aggregates, to estimate the weight per m^3 of the fresh concrete. The second method, called the *damp*, *loose volume* method, uses the cement content–strength relationship for the design of lightweight concrete mixes. The excess water absorption characteristics of lightweight aggregate should be taken into consideration while designing lightweight concrete mixes.

Mix design for self-compacting concrete

Self-compacting concrete (SCC) has the ability to fill formwork and encapsulate reinforcing bars only through the action of gravity, while maintaining homogeneity. These characteristics translate into a substantial reduction in labour cost and construction time, and a better working environment by eliminating the impact of vibration (Concrete Society, 2005). Such concretes are obtained by using a low water–cementitious materials (w/cm) ratio, incorporating high quantities of fillers and supplementary materials (cement and mineral admixtures, such as ground-granulated blastfurnace slag, pulverised fuel ash and limestone powder), superplasticiser, increasing sand/aggregate ratio, and if needed, a viscosity-modifying admixture (Skarendahl, 2001; Concrete Society, 2005; Grunewald and Walaraven, 2005; Sonebi *et al*., 2007).

Japanese method for designing SCC

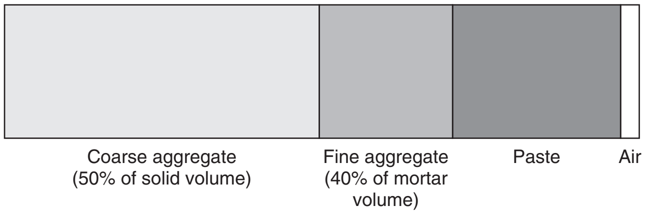

Figure 6 Japanese mix design approach for designing self-compacting concrete (adapted from Okamura and Ozawa, 1995)

The Japanese method of designing SCC (Okamura and Ozawa, 1994, 1995) is a relatively work-intensive method, but provides a fundamental understanding of the interaction of the mix ingredients of SCC. **Figure 6** presents the Japanese approach. This method can be summarised in three steps: performing paste test, optimising the mortar paste and finally adjusting SCC mixes. This method is based on the assumption that moderate-heat Portland cement or belite-rich cement is the only source of powder materials. With this method, if the requirement for self-compactibility is satisfied, the required performance of the hardened concrete is generally achieved. The coarse aggregate content (<20 mm) should be 50% (volumetric coarse aggregate to solid volume in concrete excluding air). The other part should be the mortar phase, which is composed of 40% volume sand (<5 mm) and 60% cement paste (<0.09 mm). The water/powder ratio is assumed to be 0.9 to 1.0 by volume depending on the properties of the powder and, finally, the superplasticiser dosage and the final water–powder ratio are determined in order to ensure self-compactibility.

CBI method for designing SCC

The Swedish Cement and Concrete Research Institute (Petersson and Billberg, 1999) proposed a design method for SCC. This approach considers the concrete as a solid aggregate phase in a liquid paste phase constituted by the powder, water and admixtures and takes into account the void content of the aggregates, the effect of the aggregates on passing ability (risk of blocking) and the characteristics of fine mortar. The effect of a single-sized fraction on the passing ability is experimentally measured with the L-box test (Bartos *et al*., 2002) and the blocking ratio should be less than 0.80.

Mix design for SCC using factorial design

Statistical factorial design approaches have been used to optimise SCC mixes by investigating the influence of mix ingredients on the rheological parameters, the filling ability, the passing ability, the segregation and mechanical performances (Ghezal and Khayat, 2002; Sonebi, 2004a, 2004b; Sonebi *et al*., 2007). The optimisation of SCC often necessitates carrying out several trial batches to achieve adequate balance between the rheological and fresh properties, and the mechanical performances. The factorial design method was successfully used to determine the influence of the key parameters and their interactions on the relevant properties of SCC, and to propose models which simplified the test protocol required to optimise a given SCC mix by reducing the number of trial batches needed to achieve optimum balance amongst various mix variables.

Other approaches for designing SCC

Other approaches have been reported in the literature. The LCPC method from Laboratoire Central des Ponts et Chaussées in France has been based on the solid suspension models to predict the packing density of all dry granular constituents (Sedran and de Larrard, 1999). The University College London method has been developed based on mortar tests (Domone, 2006). The Icelandic Building Research Institute method (Wallevik and Nielsson, 1998) has been based on the rheological properties of SCC (yield stress and plastic viscosity).

Concluding remarks

A discussion on mix design cannot be considered to be complete without dealing with the cost of producing concrete. In addition, the sustainability issues need to be addressed. These two aspects have not been dealt with in this chapter. The intention has been to introduce the developments in mix design for both normal and special concretes, highlighting some of the basic features of the most commonly used mix design procedures. It has been observed that concrete (normal weight, lightweight and heavyweight) can be designed for both strength and workability using recommended procedures, but design for durability is not fully developed.

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Chapter 20

Repair and rehabilitation of concrete

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CONTENTS

Repair and rehabilitation plays an important part in the effective management of assets, but opinions have differed as to the best way to tackle a deteriorating concrete structure. Repair practice is rapidly changing, with a constant stream of new products and systems available, each making a wide range of claims about performance and durability. The new European Standard EN 1504 *Products and Systems for the Protection and Repair of Concrete Structures*, has been developed in response to the Construction Products Directive to help the specifier choose the correct repair products and systems for the job. The new standard came fully into force on 31 December 2008 and conflicting national standards have been withdrawn. CE-marked products and systems, which comply with the EN 1504 series, have now begun to appear on the shelves and in catalogues of materials suppliers. EN 1504 addresses all stages of the repair process, from initial awareness of a problem, to the handover of a properly designed and executed repair and will therefore change the approach to repair of the asset owner, specifier, contractor and manufacturer. All products used will have to be pre-approved as meeting minimum performance requirements for a range of repair applications. This chapter explains the methodology of EN 1504 and discusses the repair principles that can be used to repair concrete structures. A list of the relevant testing standards is also given.

Introduction

This chapter gives practical guidance on the repair and renovation of concrete assets that are operating in normal and adverse exposure environments. At its heart is a summary of the requirements in the new European standard EN 1504 *Products and Systems for the Protection and Repair of Concrete Structures*, which comprises 10 Parts (EN 1504 series, 2008). Developed by the European Standards body, CEN, in response to the Construction Products Directive (Directive 2004/17/EC and Directive 2004/18/EC), the standard provides minimum performance criteria for repair products and systems, cross-references nearly 100 new, dedicated testing standards for repair products and systems, and also sets out a methodology for the successful repair and protection of concrete structures, including the supervision and quality control of site works.

The performance standards, which are contained in Parts 2 to 7 of EN 1504 (EN 1504 series, 2008), have the status of 'harmonised' European standards. The public procurement rules within Europe require that, for concrete repair and protection to concrete structures, only 'CE-marked' products that have been tested to meet the requirements in the performance standards should be used for repair works. One of the main purposes of the Construction Products Directive is to remove artificial or national barriers to trade, achieved by having performance criteria and unified methods of test that apply across the CEN member states. Public procurers have had to review and modify their specifications to ensure they are compatible with the methodology set out in Part 9 of EN 1504 (EN 1504 series, 2008). Those in the private sector will also need to consider the implications of EN 1504 on their concrete repair procurement process.

The various Parts of EN 1504 have been in preparation for the past 20 years. Parts of the standard have been published progressively, starting with the repair methodology of Part 9 that was issued in 1997 (ENV 1504-9, 1997), giving industry time to appreciate the future direction of the standard. The term ENV in the title denotes it was a CEN 'voluntary' standard, equivalent to a 'Draft for Development' issued by the British Standards Institution. Taking input from industry, Part 9 was revised in 2008, and a date of withdrawal of 31 December 2008 was set, by which time any conflicting national standards would have to be withdrawn.

In most cases a single method of test applies to each performance requirement. Extensive consultation has taken place across CEN member states to develop these single methods of test, in some cases adapting existing methods for use with the formulations of repair products and systems that are currently on the market; for example, EN 12190 *Determination of Compressive Strength of Repair Mortar* is based on BS 6319 *Testing of Resin and Polymer/Cement Compositions for Use in Construction*: Part 2 *Compressive Strength*. Other test standards have been rewritten from national or learned guidance, such as the RILEM Thermal Compatibility Tests I and II, which have been adapted to EN 13687 Determination of Thermal Compatibility, with Part 1 being the freeze–thaw cycling with de-icing salt immersion 'test' and Part 2 the thunder shower cycling (thermal shock) 'test'.

This chapter concentrates on the scope and application EN 1504 – *Products and Systems for the Protection and Repair of Concrete Structures*. The term 'products and systems' is significant. A supplier may have either an individual product evaluated, or have a complete system tested. For example, a crack injection resin would be certified in its own right, but a concrete repair system may comprise several products, such as a bonding coat (concrete to repair mortar), a repair mortar, a levelling treatment and a surface protection primer and top coat.

Deterioration processes

The objective of the design and construction process for concrete assets, such as to Eurocode 2 (EN 1992-1-1), is to achieve the required service life without the need for major intervention. Minor repairs, as part of routine planned preventive maintenance, should be expected and routine maintenance is essential. However, significant concrete repair works should not be required over the service life – an objective that is too frequently not being met.

With appropriate design, specification and construction, provided the appropriate exposure conditions are identified and the correct exposure class selected, concrete asset should not deteriorate significantly under the prevailing exposure conditions until the service life is reached (EN 206). With older structures, situations commonly arise where the asset does not achieve the intended service life before deterioration of the concrete becomes apparent. This 'premature' deterioration can be caused by a variety of factors built into the structure at the time of construction, sometimes acting in isolation but commonly in combination, which includes a failure to appreciate the severity of the exposure environment and its effects on durability and service life. Other common deficiencies that can impair the quality of the asset at the start of its life include:

- design – underestimate of the in-service load capacity or late changes in use;
- durability – lack of adequate consideration in the mix design and/or insufficient cover;
- materials – contamination of materials or use of poor or alkali-reactive aggregates;
- specification – poor definition of the works and standards of construction;
- construction – insufficient compaction, mixing and curing;
- supervision – lack of effective supervision, if any at all, leading to misinterpretation of requirements;
- protection – insufficient or defective waterproofing or surface protection systems.

Once in service, failure to appreciate the severity of the exposure environment has been the most common cause of premature deterioration. Most if not all concrete assets that were built before CP110 was introduced were not designed and constructed to resist the exposure environment (CP110, 1974). It was only in the early 1970s that one of the prime causes of deterioration, the chloride ion, was banned from being added to reinforced concrete during the mixing process as a set accelerator (calcium chloride). Industry is also left with older 'legacy' structures that need to be cared for, especially where the asset has to be maintained for reasons of historic or functional importance.

It has only been in the past 30 years that designing for durability rather than strength has been really understood and research put in place to explain why premature deterioration occurs, leading to the standardised designs for durability of EN 206. Even now, certain exposure environments are beyond the provisions of durability codes, requiring that concrete is provided with extra protection from the time of construction, including *surface protection systems* and *active corrosion management*, discussed later in this chapter.

Common deterioration mechanisms

The two most common deterioration mechanisms affecting reinforced concrete assets are carbonation of the concrete and free chloride ions at the surface of the reinforcement, resulting in reinforcement corrosion. These mechanisms have been extensively described and are not reviewed in detail here (Broomfield, 1997; BRE, 2000; CIRIA, 1987) except to note that, in the presence of high pH conditions in concrete, reinforcing bar is unable to corrode, due to the presence of a 'passive' film that forms on the surface of the bar. Either carbonation to the depth of the reinforcing bar, which lowers the pH of the concrete, or the presence of significant levels of chloride ion at the surface of the reinforcing bar, will destroy the passive film and allow corrosion to occur in the presence of moisture and oxygen; in dry concrete (e.g. the inside of buildings) or where no appreciable amount of oxygen can reach the bar (e.g. permanently immersed concrete), corrosion will not occur at appreciable rates.

Other causes of concrete deterioration exist – some that can be expected based on concrete composition (e.g. alkali–aggregate reaction), others that occur as the result of an incident (e.g. overload, fire). **Figure 1**, based on EN 1504-9, sets out these common causes of deterioration.

Before any maintenance or repair work is commenced, the causes of deterioration must be established and quantified using an appropriate condition assessment programme and including a structural appraisal.

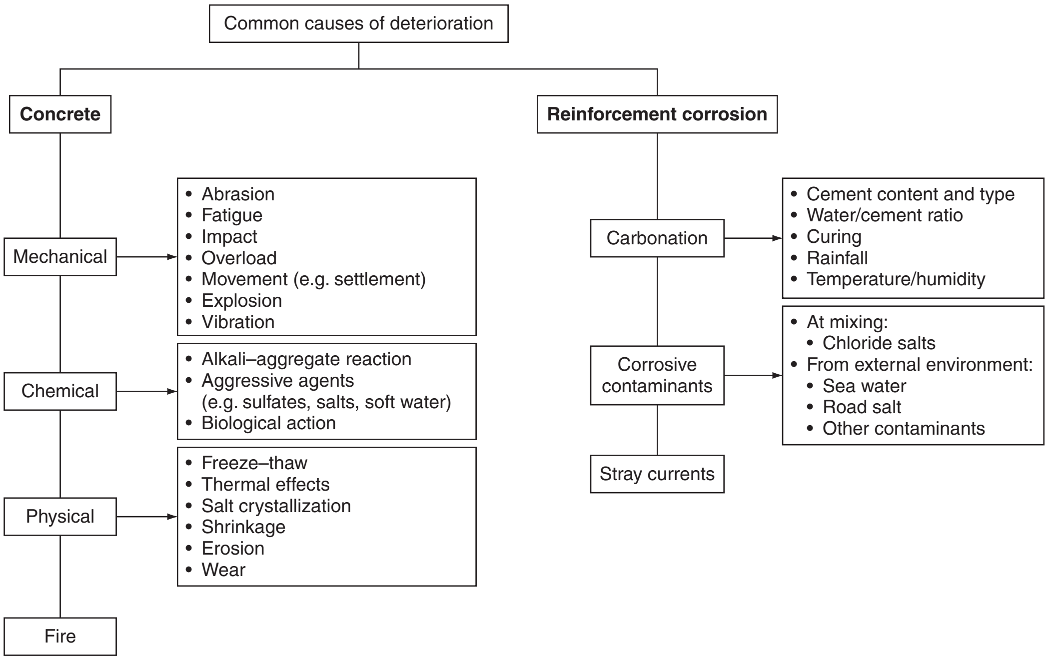


Figure 1 Reinforced concrete deterioration processes (adapted from EN 1504-9, 2008)

Condition assessment

Prior to any repair or assessment work commencing, all previous relevant information on the structure should be collated and reviewed, including evidence of previous inspection, testing and investigation works, and previous maintenance and repair. When new defects are observed, additional testing should be carried out to establish the causes and full extent of the defects, from which the future performance of the asset may be predicted (The Concrete Society, 2000; CBDG, 2002; Bungey *et al.*, 2006). A condition assessment is commonly undertaken in more than one stage, with a preliminary assessment carried out to provide immediate advice on the safety of the asset and any risk to third parties, with a more detailed assessment undertaken immediately before the works are designed.

The results from the review of previous information along with new information on the condition of the concrete and reinforcement before repair, should be documented and available for review by all stakeholders. The data can be conveniently stored in an asset management system that will form part of a life-care plan for the remaining service life of the asset, such as has recently been recommended for car parks (ICE NSC, 2005).

A typical programme of testing to establish the extent of reinforcement corrosion would include frequency distributions around the structure, comparing different elements, elevations and heights around the asset as appropriate. Measurements are made of concrete cover to reinforcement, carbonation depth, chloride ion content with depth into the surface and the presence of other deleterious substances (sulfates, acid attack, etc.), supplemented by selected core samples to measure physical properties and for chemical and petrographic analysis as appropriate (St John *et al.*, 1998; The Concrete Society, 2000).

Electrochemical testing of the reinforcement (e.g. by half-cell potential technique or linear polarisation) may be required in certain situations, particularly where elevated chloride ion contents have been measured and active hidden corrosion may be present. While corrosion of embedded reinforcement usually results in cracking and spalling of the concrete cover, active corrosion may occur for a considerable time before cracks appear; under certain conditions, the corrosion may not be expansive and therefore may not result in cracking. Electrochemical testing is able to detect active corroding reinforcement even though there are no outward visible signs (The Concrete Society, 2004; Bungey *et al.*, 2006).

Structural appraisal

As part of the structural appraisal, the properties of the concrete (e.g. compressive strength and elastic modulus) and the reinforcement detailing (e.g. bar size, type, spacing and cover) may need to be verified as part of the condition assessment work. Rapid scan techniques such as impulse radar can be particularly useful in providing confirmatory data on concrete section size and reinforcement position (The Concrete Society, 1997; Bungey *et al.*, 2006). Recalculation of the remaining load capacity in the deteriorated state may also be required, allowing for reduced cross-sectional areas of steel and reduced bond in areas of corroding reinforcement (Sarveswaran *et al*., 2000; O'Flaherty *et al*., 2008).

Currency of assessment data

While condition assessments and structural appraisals should be carried out in advance of the repair process, sometimes this is done before it has been recognised that a deterioration problem exists. As a result, initial assessments of the structure are often carried out some time before the design of the repair works is considered, due to delays caused by raising funds, for example. During this time, further deterioration in the materials and reduction in the structural capacity may occur. Where delays of more than 12 months occur, an updated assessment may be required before the protection and repair works are finalised.

Qualifications of assessors

All assessments for repair purposes should only be made by suitably qualified personnel with knowledge of investigation methods, structural design, maintenance, material technology and of the mechanisms which can contribute to the deterioration process of concrete structures. Where any doubt exists in the quality or validity of the condition assessment, then the work should be repeated, as a misdiagnosis of the deterioration may result in costly repairs that may otherwise have proven to be unnecessary or which make the deterioration worse.

Structure management strategy

The structure management strategy is not chosen on technical grounds alone but also on economic, functional, environmental and other factors; for example, maintaining or restoring safety is an essential requirement of a structure management strategy. Of equal importance is establishing the owner's requirements for the structure in terms of the remaining service life, which will directly affect decisions about the management strategy and is therefore a key consideration in the design of the protection and repair system.

A checklist of factors to be considered when choosing a management strategy should include the following, based on EN 1504-9 (EN 1504 series, 2008):

- the intended use, required performance and remaining service life of the structure;
- the required ease of access of the structure during the repair works (access, possessions, etc.), including consideration of the effects of interruption to its use;
- the likely service life of the protection and repair works (if less than the remaining service life);
- the acceptability of future maintenance and repairs during the remaining service life, which will result in a longer service life for both the works and the structure, accepting that poor maintenance (e.g. blocked roof drains) can accelerate deterioration in a concrete structure;
- the number and cost of repair cycles acceptable during the remaining service life, related to ease of access;
- the comparative whole-life cost of the various management strategies, including future inspection and maintenance or further repair cycles;
- the final appearance of the protected and repaired structure, because the materials used and the methods of preparing the existing substrate can affect the final appearance;
- the structural loading and how loads will be resisted before, during and after implementation of the strategy, particularly management of areas of concrete and steel cut away from loadbearing members;
- the consequences of structural failure and the need for temporary support or additional collapse protection during the works;
- health and safety requirements, arising from any structural consequences from deterioration and repair works;
- the effect on occupiers or users of the structure and on third parties, including the use of products that contain potentially harmful or malodorous components, noise, dust vibration, water or airborne debris and movement of construction plant and materials;
- the prevailing exposure environment for the structure, assessed as to EN 206, and whether it can be changed in the future by surface protection or other means (EN 206-1);
- the need or opportunity to protect part or all of the concrete structure from weather, pollution, salt spray etc., including protection of the substrate, during the repair work.

A management strategy will normally be selected based on one or more repair options, each having one or more detailed methods of repair. Options range from those that can restore the design life of the concrete structure in a comprehensive single operation, to simpler options that may require repeated maintenance or where components of the repair may need to be reapplied (e.g. surface protection systems), as illustrated in **Figure 2**.

Associated with each option will be a series of methods, each of which will have its own capital cost, life to first maintenance and ongoing future maintenance costs. In many cases, a repair contract specification will be based on more than one option and numerous methods of repair, depending on the severity of the deterioration in each part of the structure (i.e. the elevation, height, element type, etc. – see Section 16.1.3 Condition assessment).

Life extension

Equally important in today's world, where sustainability is high on the stakeholders' agenda, is extension of the life of an existing asset beyond the original service life. Extending the life of an asset by appropriate maintenance and repair works will provide significant savings in terms of raw

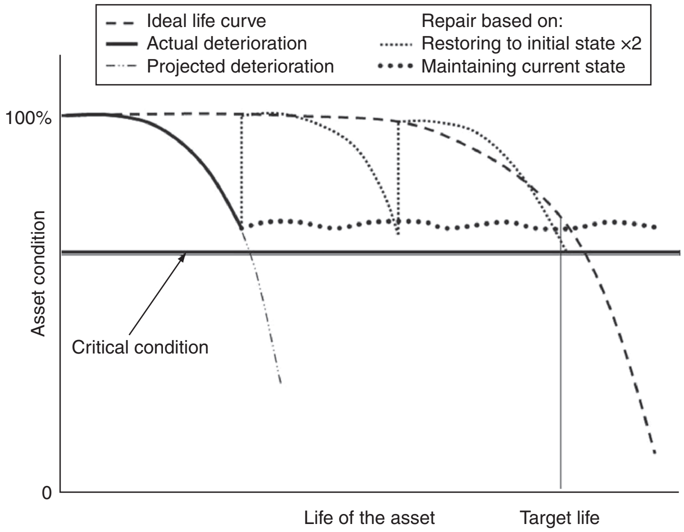


Figure 2. Different repair options applied over the life of a deteriorating asset (Robery, 2006)

materials and the energy and carbon consumed during conventional demolition and replacement works.

With a thorough understanding of the deterioration mechanisms, asset life can be successfully extended. This can be achieved either at the end of the original service life or by suitable intervention during the service life, to counteract the deterioration processes and maintain the net present value. Such assets may include highway bridges that are intended to have a 120-year service life, or decommissioned nuclear structures.

By extending the life of the asset, the value of the asset on the balance sheet is also improved. **Figure 3** shows a typical linear depreciation model for the value of an asset over its expected service life. Accelerated deterioration may begin at point A, due to failure of a waterproofing system or simply a lack of appreciation of the severity of the exposure environment. Once discovered and investigated (point B), a revised 'impaired life' may be predicted if no repair strategy is put in place. This will result in an immediate decrease in the value of the asset to its 'impaired value' at time B and accounting practice requires this impaired value to be declared. By judicious maintenance and repair, the life of the asset may be restored to the original service life, or even extended beyond the original service life. In the latter case, this will produce a revaluation of the asset, which may itself provide sufficient funds to cover the costs of the repair works.

Repair options and methods

A *structure management strategy* will normally be based on one or more repair options, each having one or more detailed methods of repair. A recommended approach to selecting options and methods for the repair of concrete assets is set out in EN 1504-9 (EN 1504 series, 2008) and is summarised below.

Options for repair

The six most common options for concrete repair are set out in EN 1504-9, which are listed below, along with added relevant references to provide more guidance on

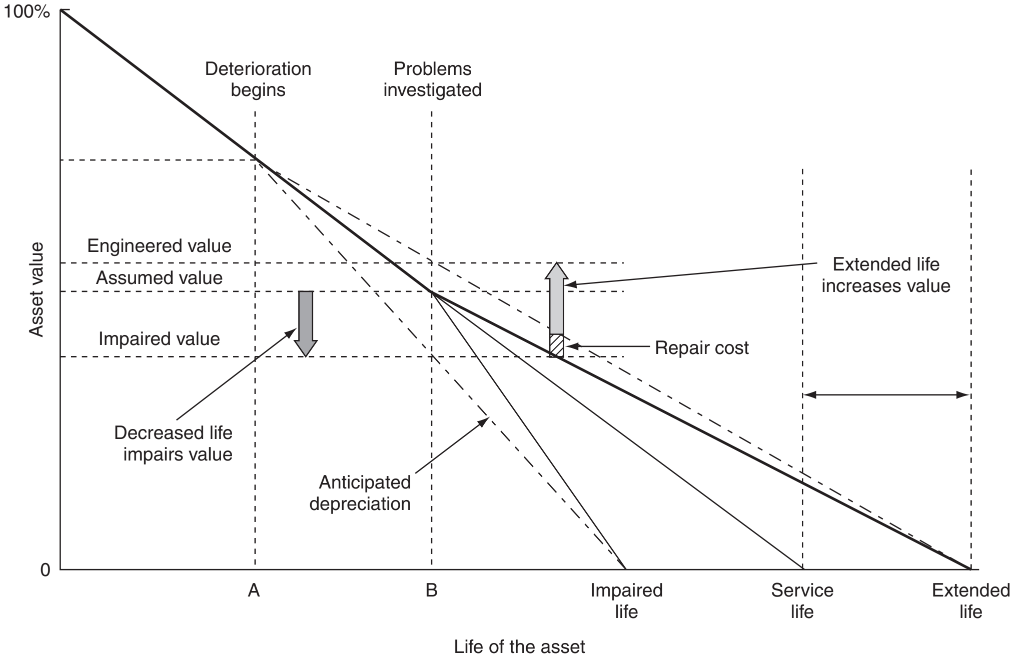


Figure 3 Effective asset maintenance and repair that extends life and increases value (Robery, 2008)

the application of the options:

1. do nothing for a certain time but monitor (CBDG, 2002);
2. reanalyse the structural capacity, possibly leading to downgrading in function (ISE Appraisal);
3. prevent or reduce further deterioration (EN 1504 series);
4. strengthen or repair and protect all or part of the concrete structure (EN 1504 series);
5. reconstruct or replace all or part of the concrete structure (EN 1992-1-1; EN 206);
6. demolish all or part of the concrete structure and replace (EN 1992-1-1; EN 206).

While certain options (i.e. 1, 2, 5 and 6) are reasonably well documented in codes and guidance, options (3) and (4) are wholly within the scope of EN 1504 and are therefore discussed in detail below.

For any deteriorated structure, each option and specific method should be assessed for its efficacy over the remaining life of the asset, which, in other words, is the process of life-cycle costing of the repair options. Consideration of the cost and suitability of the options and their effects will generally include examination of different aspects of cost, for example initial cost, maintenance costs and the possible need to introduce restrictions on the use of the structure before, during and after the works. Each option is also likely to have a different risk of deterioration developing in the future.

Methods of repair

The approach set out in EN 1504-9 subdivides the protection, strengthening and repair of concrete structures into two categories, namely defects in the concrete and defects due to reinforcement corrosion, following the grouping shown in **Figure 1**. EN 1504 further divides these categories into 11 repair 'principles', as summarised in **Table 1**, which can be used to tackle the deterioration.

Principles 1 to 6 in **Table 1** cover defects in the concrete or concrete structures that may be caused by the following actions that may act either singly or in combination:

- mechanical: e.g. impact, overloading, movement caused by settlement, and blast;
- chemical and biological: e.g. sulfate attack, alkali–aggregate reaction;
- physical: e.g. freeze–thaw, thermal cracking, moisture movement, salt crystallisation and erosion;
- damage by fire.

Defects in concrete	Reinforcement corrosion
1 Protection against ingress	7 Preserving or restoring passivity
2 Moisture control	8 Increasing resistivity
3 Concrete restoration	9 Cathodic control
4 Structural strengthening	10 Cathodic protection
5 Physical resistance	11 Control of anodic areas
6 Resistance to chemicals	

Table 1 Principles of protection and repair in EN 1504-9

Principles 7 to 11 in **Table 1** cover reinforcement corrosion caused by:

- physical loss of the protective concrete cover;
- chemical loss of alkalinity in the protective concrete cover as a result of reaction with atmospheric carbon dioxide (carbonation);
- contamination of the protective concrete cover with corrosive agents (usually chloride ions) which were incorporated in the concrete when it was mixed or which have penetrated into the concrete from the environment;
- stray currents conducted or induced in the reinforcement from neighbouring electrical installations.

EN 1504-9 further subdivides the principles into a list of 43 examples of possible 'methods' for the protection and repair of concrete assets, as listed in **Table 2**.

It would be rare for a repair situation to require only a single principle or even a single method. It is common for different principles and multiple methods to be applied to different parts of the asset. Taking the example of reinforcement corrosion due to chloride ingress, parts of the structure may require protection (Principle 1), other parts may have local damage (Principle 7) and other parts may have severe damage requiring active corrosion management (Principle 10). Under each principle, various methods may be explored to find the solution that best meets the repair objectives and delivers the lowest life-cycle cost.

To comply with EN 1504, only listed methods should be selected for repair and protection of concrete. To 'future-proof' the standard, EN 1504 permits other techniques to be used that are not listed as methods in the standard, provided documented evidence exists that they comply with one or more principles. However, the current list is comprehensive and should allow successful repairs in most situations.

As can be seen from **Table 2**, the various Parts of the EN 1504 series cover only some of the methods given in column 2; column 3 (Relevant Part of EN 1504 Parts). In EN 1504-9, the column is left blank for the methods that are outside EN 1504, but **Table 2** shows three additional categories: other relevant European standards; EOTA-certified (European Organisation for Technical Approvals) techniques that are outside EN 1504; proprietary systems offered by one or more companies with no independent performance certification at this time.

Specifying the repair

When the appropriate methods have been chosen, the relevant performance requirements can then be built into

Principle	Examples of methods based on the principles	Relevant part of EN 1504 (where applicable)
Principles and methods related to defects in concrete		
1 Protection against ingress	1.1 Hydrophobic impregnation	Part 2
	1.2 Impregnation	Part 2
	1.3 Coating	Part 2
	1.4 Surface bandaging of cracks	EOTA
	1.5 Filling of cracks	Part 5
	1.6 Transferring cracks into joints	EOTA
	1.7 Erecting external panels	EN 1992-1-1
	1.8 Applying membranes	EOTA
2 Moisture control	2.1 Hydrophobic impregnation	Part 2
	2.2 Impregnation	Part 2
	2.3 Coating	Part 2
	2.4 Erecting external panels	EN 1992-1-1
	2.5 Electrochemical treatment	Proprietary
3 Concrete restoration	3.1 Hand-applied mortar	Part 3
	3.2 Recasting with concrete or mortar	Part 3
	3.3 Spraying concrete or mortar	Part 3
	3.4 Replacing elements	EN 1992-1-1
4 Structural strengthening	4.1 Adding or replacing embedded or external reinforcing bars	EN 1992-1-1
	4.2 Adding reinforcement anchored in pre-formed or drilled holes	Part 6
	4.3 Bonding plate reinforcement	Part 4
	4.4 Adding mortar or concrete	Part 3 & Part 4
	4.5 Injecting cracks, voids or interstices	Part 5
	4.6 Filling cracks, voids or interstices	Part 5
	4.7 Prestressing (post-tensioning)	EN 1992-1-1
5 Increasing physical resistance	5.1 Coating	Part 2
	5.2 Impregnation	Part 2
	5.3 Adding mortar or concrete	Part 3
6 Resistance to chemicals	6.1 Coating	Part 2
	6.2 Impregnation	Part 2
	6.3 Adding mortar or concrete	Part 3
Principles and methods related to reinforcement corrosion		
7 Preserving or restoring passivity	7.1 Increasing cover with additional mortar or concrete	Part 3
	7.2 Replacing contaminated or carbonated concrete	Part 3
	7.3 Electrochemical realkalisation of carbonated concrete	EN 14038-1
	7.4 Realkalisation of carbonated concrete by diffusion	Proprietary
	7.5 Electrochemical chloride extraction	prEN 14038-2
8 Increasing resistivity	8.1 Hydrophobic impregnation	Part 2
	8.2 Impregnation	Part 2
	8.3 Coating	Part 2
9 Cathodic control	9.1 Limiting oxygen content (at the cathode) by saturation or surface coating	Proprietary
10 Cathodic protection	10.1 Applying an electrical potential	EN 12696-1
11 Control of anodic areas	11.1 Active coating of the reinforcement	Part 7
	11.2 Barrier coating of the reinforcement	Part 7
	11.3 Applying corrosion inhibitors in or to the concrete	EOTA

Table 2 Matrix of principles, methods and guidance for protection and repair (based on EN 1504-9)

the specification for the works. Traditionally, full and detailed performance specifications have needed to be produced that list all the key performance requirements, because a product may be offered for use in repair that was not suitable for the task. EN 1504 overcomes this issue, because the CE-marking process is intended to ensure that only suitable repair products and systems are available to satisfy the various methods. In effect, all the specifier needs to quote in the specification is that a product is CE-marked for the required method (e.g. Method 7.2). The exceptions to this are the methods that are not linked to Parts of EN 1504, such as EOTA or 'proprietary' systems and these must be evaluated separately and in greater detail to ensure the products are fit for the intended purpose.

In reality, the EN 1504 specification process is slightly more complicated. Parts 2 to 7 of EN 1504, which

Item	Performance characteristic	Test method	Requirement			
			Structural		Non-structural	
			Class R4	Class R3	Class R2	Class R1
1	Compressive strength	EN 12190	≥45 MPa	≥25 MPa	≥15 MPa	≥10 MPa
2	Chloride content	EN 1015-17	≤0.05%		≤0.05%	
3	Adhesive bond*	EN 1542	≥2.0 MPa	≥1.5 MPa	≥0.8MPa	
4	Restrained* shrinkage	EN 12617-4	Bond strength after test			Not required
			≥2.0 MPa	≥1.5 MPa	≥0.8 MPa	
5	Carbonation resistance	EN 13295	d_k ≤ control concrete		Not required	

* Product or system applied to a reference concrete substrate to EN 1766.

Table 3 Method 7.2, minimum requirements for all intended uses (based on EN 1504-3)

contain the performance requirements, prescribe minimum performance acceptance values for specific critical properties of the products and systems. Each of the parts lists three classifications of performance requirement:

- all intended uses – minimum performance values that every product must meet for the particular method, including physical and chemical properties, and including quality control and environmental issues;
- certain intended uses – recommended minimum performance values for one or more additional tests that may be called up by the specifier for an unusual or difficult repair situation;
- special applications – recommended methods of test, with no recommended minimum performance values, that may be called up by the specifier for a very unusual or difficult repair situation.

Use of EN 1504 in a specification is best illustrated by an example, such as for Method 7.2 mentioned above. **Table 3** is based on EN 1504-3, listing the performance requirements for four strength classes of mortar (Class R1 to R4) and corresponding performance requirements for all intended uses.

Depending on the particular project, the engineer may decide that the repair mortar should be stronger than a class R4 mix, for example to match the strength of the existing concrete, and so specification for Method 7.2. *Replacing contaminated or carbonated concrete* could specify that the repair mortar is to be CE-marked to meet the requirements of Method 7.2, but have a minimum strength of 50 MPa.

Table 4 can be used to add additional properties for the repair mortar, for example where the specific requirements of the project mean that a successful repair application is more demanding.

The following examples illustrate why some of these additional specification items in **Table 4** may be required.

- If the repair is to have a **structural function**, then item 6 (elastic modulus) could be added to the requirements. This may be

Item	Performance characteristic	Test method	Requirement			
			Structural		Non-structural	
			Class R4	Class R3	Class R2	Class R1
6	Elastic modulus	EN 13412	≥20 GPa	≥15 GPa	No requirement	
7	Thermal compatibility Pt 1, freeze–thaw*	EN 13687-1	Bond strength after 50 cycles ≥2.0 MPa	≥1.5 MPa	≥0.8 MPa	Visual inspection after 50 cycles
8	Thermal compatibility Pt 2, thunder shower*	EN 13687-2	Bond strength after 50 cycles ≥2.0 MPa	≥1.5 MPa	≥0.8 MPa	
9	Thermal compatibility Pt 4, dry cycling*	EN 13687-4	Bond strength after 50 cycles ≥2.0 MPa	≥1.5 MPa	≥0.8 MPa	
10	Skid resistance*	EN 13036-4	Class I: >40 units wet tested Class II: >40 units dry tested Class III: >55 units wet tested			
11	Coefficient of thermal expansion	EN 1770	Not required if tests 7, 8 or 9 are carried out. Otherwise declared value			
12	Capillary absorption	EN 13057	≤0.5 kg m^{-2} $hour^{-0.5}$			No requirement

* Product or system applied to a reference concrete substrate to EN 1766.

Table 4 Method 7.2, minimum requirements for certain intended uses (based on EN 1504-3) important, because some resin mortar products may have an elastic modulus that is significantly lower than this figure of 20 GPa for Class R4 mortar.

- If the repair is in an area with particularly severe thermal cycles, then items 7, 8 or 9 could be added, reflecting a severe northern European climate, a wet Mediterranean climate, or a dry southern Mediterranean climate respectively.
- In flooring repairs, skid resistance (item 10) may be important and these Classes are extracted from a flooring standard (EN 13036-4).
- In particularly large applications or where there is high solar gain, limiting the unconfined linear thermal expansion coefficient may be important.
- Where the repair system does not include a final surface protection system, such as a film-forming paint, it may be important to limit the water uptake when wet.

It should be remembered that the Performance Standards do not limit the materials that can be used in the products or systems. In the case of repair mortars, the products are classified into three types:

- CC – mortars made using only hydraulic cements (as defined in EN 197-1) as a binder.
- PCC – mortars comprising hydraulic cements and polymer dispersions, also called ‘polymer-modified cementitious mortar’.
- PC – mortars made using polymer binders only, such as epoxy, acrylic or polyester resin.

Where necessary, the specification should also include the type of binder permitted in the repair.

Test methods for ‘Special Applications’ are in the informative Annex B of the Performance Standards. For repair mortars, EN 1504-3 lists the following methods:

- chloride ion ingress (e.g. seawater immersion) – EN 13396;
- creep in compression (e.g. special structural applications) – EN 13584;
- chemical resistance (e.g. acids or other chemicals) – EN 13529 or ISO 2812-1;
- overhead application (e.g. flexure in the soffit of bridge beams) EN 13395-4.

Now that the CE-marking process has started, new products have now begun to appear on the market carrying the specified label and CE-mark, as set out in the Annex ZA of each Performance Standard. An example of a typical label is shown in **Figure 4**.

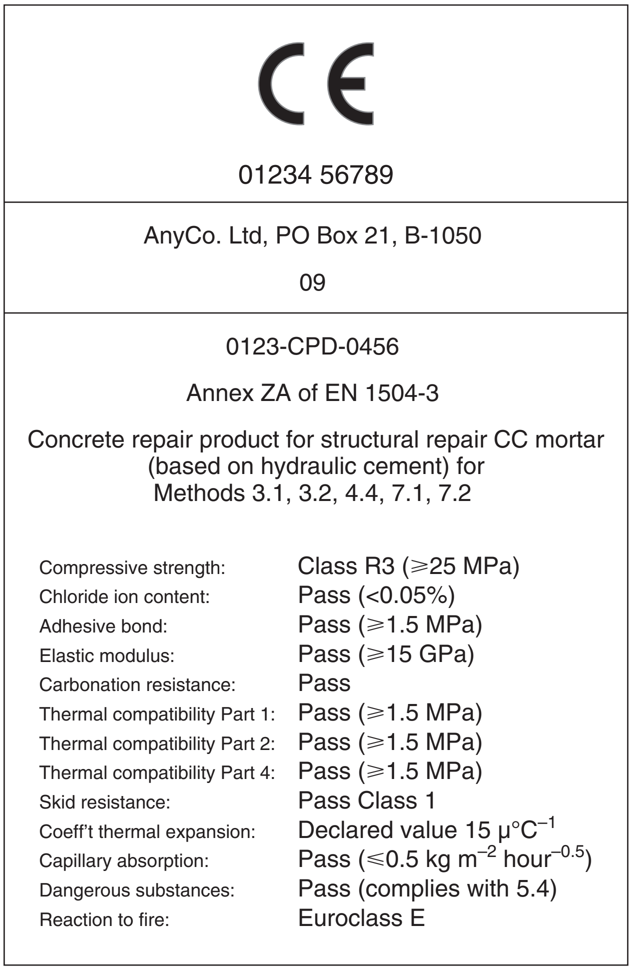
Many manufacturers have opted to test their products and systems for both sets of requirements in the relevant Performance Standards (i.e. for all intended uses and certain intended uses, as listed in **Table 3** and **4** above).

As can be seen for the CE-marking for a repair mortar in **Figure 4**, the repair mortar meets the requirements for several Methods, namely 3.1, 3.2, 4.4, 7.1 and 7.2 and has values for all tests in **Tables 3** and **4**.

It is a requirement of the CE-marking process that the label only carries the fact that the product passes the test: the specifier would have to refer to the manufacturer’s data sheet to find the actual test results for each individual test method.

Within each part of the Performance Standards are methods of identification for the products and systems. Many of the formulations contain proprietary blends of resins, cements, aggregates, additives and admixtures, making it difficult to analyse the product using standard methods. Manufacturers are therefore required to keep on file the results of identification tests such that, in the event of a dispute arising, the results may be used to confirm a sample is an authentic material; such tests may include infrared or thermo-gravimetric traces of the product or system.

The remaining requirements in the Performance Standards are intended to ensure the products and systems are



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Annex ZA of EN 1504-3

Concrete repair product for structural repair CC mortar
(based on hydraulic cement) for
Methods 3.1, 3.2, 4.4, 7.1, 7.2

Compressive strength:	Class R3 (⩾25 MPa)
Chloride ion content:	Pass (<0.05%)
Adhesive bond:	Pass (⩾1.5 MPa)
Elastic modulus:	Pass (⩾15 GPa)
Carbonation resistance:	Pass
Thermal compatibility Part 1:	Pass (⩾1.5 MPa)
Thermal compatibility Part 2:	Pass (⩾1.5 MPa)
Thermal compatibility Part 4:	Pass (⩾1.5 MPa)
Skid resistance:	Pass Class 1
Coeff’t thermal expansion:	Declared value 15 µ°C^{-1}
Capillary absorption:	Pass (⩽0.5 kg m^{-2} hour$^{-0.5}$)
Dangerous substances:	Pass (complies with 5.4)
Reaction to fire:	Euroclass E

Figure 4 Mock-up of CE-marking for a repair mortar

safe (i.e. in terms of release of dangerous substances and reaction to fire) and consistent (i.e. produced under a factory production control system to deliver a quality assured product). Attestation of conformity is set out in Part 8 of EN 1504 and covers quality control and evaluation of conformity for the products and systems.

Build-up of a traditional concrete repair

The detailed steps in the traditional concrete repair process are best illustrated by example. Consider the case of repair to a carbonated concrete structure suffering from reinforcement corrosion, which is relatively straightforward to rectify. Because corrosion will only occur when the bar lies in damp, carbonated concrete, the traditional remedial process involves identifying areas of the structure where the cover is lower than the depth of carbonation and removing these for replacement with new concrete or mortar.

This approach will locate both the damaged areas of concrete cover overlying actively corroding reinforcing bars, and areas of concrete where the reinforcing bars are active but not yet sufficiently active to cause cracking or spalling of the concrete cover. Treating both these areas by repair and replacement will therefore eliminate completely any risk of further corrosion and spalling once the repairs are completed, provided measures are taken. The detailed steps in this traditional repair process are summarised in **Table 5**.

Following inspection and diagnosis, the repair principle is to remove the carbonated concrete and replace it with new material (Method 7.2 in **Table 2**) to restore a highly alkaline environment around the reinforcing bar, such as a cementitious repair mortar. Where significant loss of section has occurred in areas of the reinforcement it may be necessary to supplement these areas with new bars, either lapped onto existing bars, or anchored into the concrete (Methods 4.1 or 4.2).

Step	Description
1	Inspection and diagnosis
2	Concrete breakout
3	Cleaning and supplementing existing reinforced bar
4	Apply anti-corrosion coating to reinforcing bar
5	Apply bonding aid to existing concrete
6	Erect formwork (as required to re-establish the profile)
7	Apply and compact the repair mortar, concrete or sprayed concrete
8	Cure the repair mortar (as required)
9	Apply a levelling coat (as required) and a surface protection system
10	Supervise and carry out quality control checks

Table 5 Steps in traditional concrete repair (Currie, 1994)

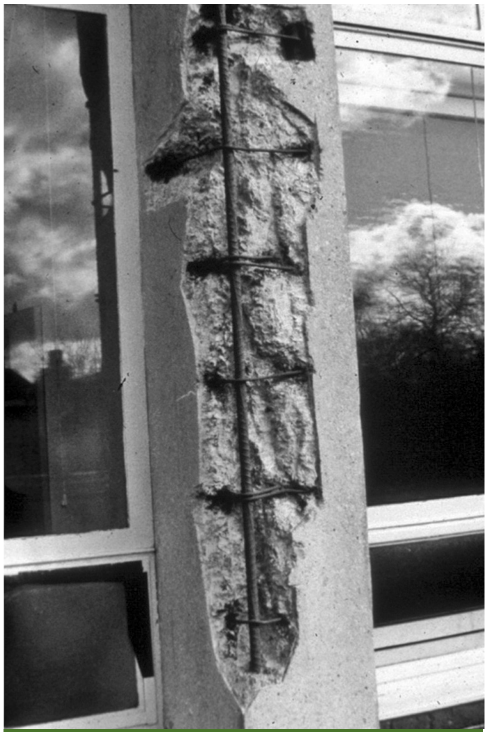

Figure 5 Reinforcement prepared in advance of concrete repair works

Sometimes the chosen repair system may include an anti-corrosion coating, which is applied directly to the cleaned reinforcing bar (selected from Methods 11.1 to 11.2) before the mortar is placed, as illustrated in **Figure 5**.

The completed repair would then be followed by application of an anti-carbonation type of surface protection system, of either standard performance or with crack-bridging capability (Method 1.3). This final treatment will not only prevent further carbonation of the concrete over the remaining service life, preventing corrosion on reinforcing bars that were passive at the time of the repair, but will also significantly improve the appearance of the finished repair (**Figure 6**). Adding extra concrete (Method 7.1) would also work as an alternative to using a surface protection system.

The above approach will give a long-term solution with limited future maintenance costs, which would normally include the reapplication of the anti-carbonation coating every 15 to 25 years, depending on the coating type and the exposure condition.

Where the extent of active bar is significant, due to high levels of carbonation, then alternative approaches will need to be considered, including realkalisation of the concrete (Method 7.3), encouraging diffusion of alkali into the concrete (Method 7.4) or application of corrosion inhibitors to the surface (Method 11.3). In these cases, repair works will still be required to treat delaminated or spalling

Figure 6 (a) Typical concrete appearance following multiple concrete repairs and (b) following application of an anti-carbonation coating

areas of concrete and replace reinforcement that has been eroded by corrosion, followed by applying a surface protection system. In severe cases, it may be necessary to demolish the structure and rebuild it.

Cheaper and probably shorter-lived repair options may also be considered, if the future life of the structure is measured in years rather than decades, as recommended in EN 1504-9. Examples of these options for the repair case above are given below in descending order of longevity:

- Repair only visibly damaged areas and apply an impregnation or coating system (as above), relying on the anti-carbonation coating to reduce the rate of corrosion in active areas by protecting the concrete from direct wetting by rainfall, thereby raising the internal resistivity of the concrete (Methods 8.2 and 8.3 only, as Method 8.1 will not be suitable – see below, 'Adverse reactions').
- Repair only the visibly damaged areas and accept an annual maintenance expenditure for repairing new areas (i.e. just Method 7.2 alone).
- Use holding repairs, whereby the structure is regularly inspected and all cracked and delaminated concrete is removed and either left exposed or reinstated with a repair mortar (Method 7.2), with the plan to undertake full repairs or demolition at a later date.

Note: it is not an option to leave the structure to deteriorate and only attend to damage when bits drop off. This is wholly unacceptable from several aspects, not least being the danger to the users and public of falling concrete and the possible risk of critical weakening of the structure due to section loss of the reinforcement.

Each option should be assessed on its merits and costed over the remaining life of the structure.

Dealing with chloride in the concrete

As is set out in published guidance on repair strategy (Broomfield, 1997; BRE, 2000; The Concrete Society, 2008), the approach needed for a structure that has chloride ion contamination should not be by simplistic patch repairs to the damaged areas. Lasting repair requires either:

- complete removal of chloride-contaminated concrete in contact with the reinforcement until the chloride ion content is below a threshold level for corrosion activation (Method 7.2 in **Table 2**); or

- use of advanced corrosion control methods, following Principles 7 to 11 in **Table 2**.

A brief word of explanation is required on the differences between normal corrosion due to carbonation and chloride-induced corrosion, as the latter is commonly misdiagnosed and as a result incorrect repair systems may be specified that do not tackle the cause of the problem. For corrosion to begin on the surface of a reinforcing bar, the passive film must be broken down, which chloride ions can do if present in sufficient concentration at the bar surface, with no overall drop in pH of the concrete.

Chloride ions can get into concrete from many different sources, but these can be split into two main chloride source classifications:

- **penetrating chloride**, coming from the environment (e.g. salt deposition from de-icing salt or seawater spray, splash or immersion); or
- **admixed chloride**, either intentionally or unintentionally added into the concrete at the time of mixing (e.g. use of chloride-based admixtures, chloride-contaminated aggregates or brackish water).

The amount of chloride that can be tolerated in the concrete before corrosion of reinforcement begins and the severity of any subsequent corrosion both depend on the source classification above.

In simple terms, the presence of chloride ions at the surface of the reinforcing bar affects the stability of the passive film that forms on reinforcement in high pH conditions, breaking it down locally and leading to corrosion in the presence of moisture and oxygen. However, chloride-induced corrosion is not that simple, because whereas long lengths of bar may lie in concrete that has a high chloride ion content, corrosion only occurs at certain positions along the bar (anodes) which serve to protect other areas along the bar (cathodes).

Chloride-induced corrosion can be thought of as a mini battery (or corrosion cell) in the concrete where anodes and cathodes form along the length of the reinforcement: anodes are the corrosion sites, and the cathodes are the other half of the corrosion 'circuit' and are protected against corrosion by the anodes. Corrosion at the anode sites is typically highly localised and over a small length of bar (e.g. 10 mm) the steel cross-section is rapidly converted into corrosion products (e.g. **Figure 7**).

Relevant to repair, if just the anode sites are cut out and repaired with concrete, other lengths of the reinforcement in chloride-contaminated concrete that appeared to be in good condition at the time of the repair, will in fact be anodic relative to the repair. As a consequence, these areas will begin to corrode – the repairs become the cathodes and the unrepaired areas become new anodes. For this reason, reinforcement corrosion caused by chloride contamination alone or in combination with carbonation, is the most difficult form to stop by traditional concrete repair works and is why advanced corrosion control measures have been developed.

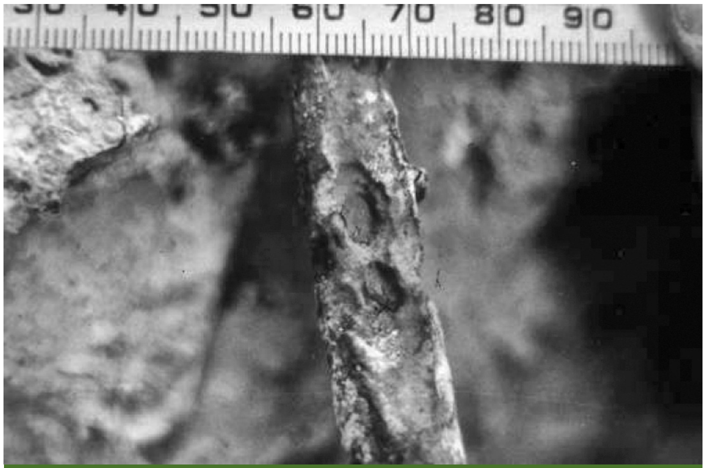

Figure 7 Typical localised pitting corrosion of reinforcing bar (Currie and Robery, 1994)

Considerable technical debate exists about when and why reinforcing bars begin to corrode, as explained in the references listed above. It is generally accepted that for *penetrating chloride*, once the chloride level at the surface of the reinforcement has reached 0.4% chloride ion by weight of cement (bwc), pitting corrosion on the reinforcement will begin. But corrosion is also found to occur at higher or lower threshold values (BRE, 2000). For *admixed chloride*, the threshold for corrosion is generally considered to be 1.0% bwc, but corrosion is also found to occur at higher or lower threshold values.

The main difference in threshold levels between the two source classifications is that at mixing, a large proportion of the chloride ion is bound up as inactive cement hydrate complexes (Currie, 1994). These complexes are broken down during carbonation, liberating the aggressive 'free' chloride ion, which provides part of the explanation as to why older concrete structures with admixed chloride may not corrode at a significant rate for many years, but following carbonation, more rapid corrosion suddenly begins.

Applying the repair principles and methods in **Table 2** to chloride contamination requires further explanation. The methods can conveniently be split into permanent repairs and holding repairs that reduce or stop corrosion rates temporarily but do not stop corrosion permanently.

Permanent repairs

- *Method 7.2 Replacing contaminated or carbonated concrete* – this must include all contaminated concrete above a threshold (e.g. 0.4% bwc for penetrating chloride), and not just areas that are actively corroding (as set out above).

- *Method 7.5 Electrochemical chloride extraction* – a temporary system is installed on the concrete surface and, through connections to the reinforcement, this will remove chloride ions from the concrete over several weeks; additional measures are usually required to prevent recontamination of the concrete by chloride ion, such as by using protective or hydrophobic treatment (Principle 1).
- *Principle 10 Cathodic protection* – this technique is becoming an increasingly common method for treating structures with high levels of chloride ion contamination and is a permanent installation that overpowers the anode and cathode sites on the bar, making all reinforcement cathodic and unable to corrode, even though the chloride ions remain in the concrete.

Holding treatments

- *Principle 8 Increasing resistivity* – all three methods (8.1–8.3) are intended to allow the concrete to lose moisture and thereby increase the resistivity of the corrosion circuit, slowing down the corrosion process, but evidence suggests that while corrosion rates may reduce, the methods will not normally stop corrosion in chloride-contaminated concrete.
- *Method 9.1 Limiting oxygen content (at the cathode) by saturation or surface coating* – concrete permanently immersed in seawater is an illustration of the principle, as oxygen cannot reach cathodic areas of the bar. Consequently, reinforcement at the anodic areas will not corrode at appreciable rates. Immersion is rarely a practical repair solution; application of a coating on the concrete surface is unlikely to stop oxygen access to the reinforcing bar completely.
- *Method 11.1 Active coating of the reinforcement* – coatings based on zinc-rich primers or cementitious slurries with or without corrosion inhibitors may help reduce the risk of patch repairs acting as cathodes where areas of chloride ion contamination are left in the structure, but will not prevent new anodes forming in areas adjacent to a patch repair.
- *Method 11.2 Barrier coating of the reinforcement* – coatings such as epoxy resin will provide an electrical and oxygen barrier on the bar surface, preventing a patch repair acting as a cathode to chloride-contaminated areas that remain, but will not prevent new anodes forming in areas adjacent to a patch repair.
- *Method 11.3 Applying corrosion inhibitors in or to the concrete* – inhibitors added in the repair mix will help prevent corrosion occurring in the patch repair, but will not affect adjacent areas significantly. Inhibitors applied onto or into the concrete surface are intended to penetrate to the reinforcing bar and counteract the effect of the chloride ions, although because of the volatility of these inhibitors the effect is unlikely to be permanent and their effects are unlikely to last for more than ten years without reapplication.

Adverse reactions

As indicated above, in a normal repair situation, several protection and repair methods may be chosen in combination to tackle a range of deterioration processes. Care needs to be taken to consider the possible adverse effects of the chosen methods and the consequences of interactions between them. EN 1504 identifies several examples of possible adverse interactions, which include the following:

- A hydrophobic impregnation (Methods 1.1 and 2.1) that is used to reduce wetting and salt build-up will also reduce the internal moisture content of concrete, which will increase the rate of carbonation.
- An impervious surface coating (Methods 1.3 and 2.3) that is applied to prevent carbonation or chemical attack will stop transpiration at the surface, which may lead to the trapping of moisture in the concrete behind the coating. This may result in the breakdown of adhesion between the coating and concrete or an increased susceptibility to frost damage.
- A post-tensioning system applied to a weakened structural element (Method 4.7) can cause tensile stresses in other parts of the structure, resulting in cracking and accelerating ingress of deleterious materials.
- Electrochemical treatments (Methods 2.5, 7.3, 7.5 and 10.1), if designed incorrectly or inappropriately, may cause embrittlement of susceptible prestressing steel, or initiate alkali–aggregate reaction with susceptible aggregates.

The specifier should therefore ensure that products and systems used in combination are compatible with each other and with the original concrete structure to which they are applied.

Materials for repair

As set out above in the section *Repair, options and methods*, EN 1504 delivers 'CE-marked' products and systems, that have to reach minimum performance standards for a range of engineering properties related to the end use. For example, a surface protection system for concrete, such as a film-forming paint, will have different performance requirements depending on whether it is intended to:

- protect against ingress of chloride ions;
- reduce carbonation of the concrete;
- control moisture penetration into the surface;
- be applied over active cracks in the concrete.

As with repair mortars discussed above, a high-performance coating system may meet the minimum performance requirements of several methods, yet a lower-performance coating system may only pass one specific method category.

EN 1504 is specifically intended to assess systems comprising many component products: properties of a system for concrete repair should be tested and compared with the relevant performance requirements. It is not intended that each component is tested and evaluated individually against the performance requirements unless

Material	Crack injection	Bar coating	Protective coating	Bond coat	Small patch	Large patch	Structural repair
Synthetic resin	◆	◆	◆	◆			
Cement grout	◆	◆	◆	◆			
Epoxy mortar (PC)					◆	(◆)	(◆)
Polymer-modified mortar (PCC)					◆	◆	(◆)
Hydraulic concrete or mortar (CC)						◆	◆

Table 6 Typical applications for materials by generic type

the products can be used by themselves to meet the performance requirement.

For example, the properties of a surface protection system for a car park deck may contain multiple products, such as a primer, elastic layer, sealing layer and wearing layer, each layer being of the thicknesses specified by the manufacturer. Compliance with the performance requirements is measured on the system applied in accordance with the manufacturer's recommendations and this would be stated alongside the CE conformity symbol on the packaging of the products that comprise the system.

The most important aspect for the specifier or end user is the performance of the product or system in service. The specifier must be aware that Performance Standards give minimum levels of performance, below which the product will not be 'fit for purpose' and therefore will not carry a CE mark for the method. While this minimum performance may be suitable for a 'general' intended purpose, it may not necessarily be adequate for all applications.

Following the guidance in EN 1504 for the selection of materials will not be sufficient in itself. Reference will need to be made to the properties of materials needed for specific applications and the inherent physical and chemical properties of the products and systems that may influence decisions about what product to use in any specific circumstance. **Table 6** shows the common application of different generic types of concrete repair materials: items in parenthesis should be considered with particular care, due to potential differences between the properties of PC and PCC materials in large or structural repairs.

The following sections briefly summarise some of the key points about the Performance Standards.

Part 3 – structural and non-structural repair

EN 1504-3 is for grouts, mortars and concretes for the structural or non-structural repair of concrete and is directly relevant to repair methods 3.1, 3.2, 3.3, 5.3, 6.3, 7.1 and 7.2 in **Table 2**, as well as being an integral part of other repair methods (e.g. replacing delaminated concrete before applying cathodic protection). Repair mortars and concretes are therefore commonly used in conjunction with other products and may be part of a system of repair. The detail of EN 1504-3 was discussed in the section *Specifying the repair*, above.

The minimum performance requirements are set out in **Table 3** categorised into four classes based on strength: Class R4 (>45 MPa) and R3 (>30 MPa) are suitable for structural repair, while Class R2 (>25 MPa) and R1 (>10 MPa) are suitable for non-structural work and typically contain low-density fillers for high-build application. The structural mortars and concretes have stronger adhesion to the substrate and, where required, an elastic modulus can be specified of greater than 20 GPa for class R4 and greater than 15 GPa for class R3.

The standard permits three types of binder: CC, PCC and PC. Particular care is needed in the selection of PC or PCC mortars or concretes, as these may have very different physical and mechanical properties to conventional concrete and are significantly more sensitive to in-service temperature. **Table 7** gives some general guidance on the properties of repair mortars based around generic type to illustrate some of the differences that exist between CC, PCC and PC products; this list is indicative only and

Property	Concrete (CC)	Mortar (CC)	Polymer-modified mortar (PCC)	Epoxy mortar (PC)
Compressive strength (MPa)	20–80	20–50	10–55	55–110
Tensile strength (MPa)	1–3	1–3	3–6	55–110
Elastic modulus (GPa)	20–40	15–35	10–25	0.5–25
Thermal expansion coefficient (10^{-6} mm mm^{-1}°C^{-1})	7–13	8–20	8–20	20–30
Maximum service temperature under load (°C)	>300	>300	100–300	40–80

Table 7 Important properties of repair mortars based on generic type

exceptions will be found in the marketplace. Much of the variation in properties is caused by differences in resin binder formulation, such as the exact base and hardener used in an epoxy resin, use of additives and the volume of filler. The maximum service temperature under load reflects the glass transition temperature of the resin used in the PC, which is particularly important for load-bearing and strengthening applications (see also Part 4 below).

Part 4 – strengthening

EN 1504-4 is for adhesives used for the structural strengthening of concrete and is directly relevant to Methods 4.3 and 4.4. Part 4 does not specify performance requirements for the strengthening materials, such as steel plate or glass, aramid or carbon fibre sheets. The scope covers:

- bonding external plates to the surface of concrete;
- bonding hardened concrete to hardened concrete in repair and strengthening situations;
- casting of fresh concrete to hardened concrete, where the adhesive serves as a bonding aid to promote full stress transfer.

The minimum performance requirements cover the following properties:

- suitability for application, including to vertical surfaces and soffits, horizontal surfaces and by injection;
- suitable temperature range for application and curing, and suitability for application to a wet substrate;
- adhesion of plates to plates, concrete and corrosion protected steel, and of hardened or fresh concrete to hardened concrete;
- durability of the complete system under thermal or moisture cycling;
- open time and workable life;
- modulus of elasticity in compression and in flexure;
- compressive and shear strength;
- glass transition temperature, above which the adhesive changes from a hard brittle state to a more rubbery one;
- coefficient of thermal expansion;
- cure shrinkage.

Bonding adhesives may be based on binders comprising either hydraulic cement, or polymer-modified cement or polymer resin. Appropriate guidance must be followed for the successful design and installation of strengthening schemes. In particular, structures strengthened by externally bonded reinforcement must be protected from excessive warming and from heat in the event of a fire. As noted under ‘Glass transition temperature’ above, the particular resin binder used will be susceptible to a change in state above a certain temperature, resulting in a transformation from a structural material to a non-structural material. As indicated in **Table 7**, the glass transition temperature would limit the application of many resin adhesives unless measures are taken to protect the resin from warm temperatures, which for ambient-cured adhesives is likely to be less than 80°C.

Part 5 – injection resins

EN 1504-5 is for products intended for filling of cracks, voids and interstices in concrete. The relevant methods of repair are: protection against ingress and waterproofing by filling cracks (Method 1.5); structural strengthening by injecting cracks, voids or interstices (Method 4.5); and filling cracks, voids or interstices (Method 4.6). EN 1504-5 may not apply to highly specialised applications in extreme environmental conditions, such as cryogenic use, or following earthquake loading, where specific performance requirements may be needed.

The primary performance characteristics of injection products are:

- basic characteristics, related to adhesion, shrinkage, compatibility with steel and concrete, glass transition temperature and watertightness;
- workability characteristics, which indicate the conditions in which the product can be used (width, moisture state of the crack);
- reactivity characteristics, including the workable life and strength development;
- durability of the hardened product under the prevailing climatic conditions;
- chloride content and corrosion behaviour for injection of reinforced concrete.

Injection resins may use binders comprising either hydraulic cement, or polymer-modified cement or polymer resin. The choice of binder and filler in the product will affect many of the properties of the injection resin, including tolerance to moisture in the crack and the ‘injectability’ of the product, i.e. the ability to penetrate into very fine cracks, of 0.2 mm or less. Typically, unfilled epoxy, acrylic or polyurethane resins are used to fill the finest cracks.

EN 1504-5 divides the products and systems into three basic types: Type F for force-filling of cracks; Type D for ductile filling of cracks, allowing some movement; Type S for swelling to fill and block cracks.

Part 6 – anchoring

EN 1504-6 is for products used for anchoring reinforcing steel into existing concrete as part of repair or strengthening work. Anchoring is commonly achieved by inserting new reinforcing bars into preformed or drilled holes and bonding in place with an adhesive (Method 4.2). The anchoring products include hydraulic binders, synthetic resins or a combination of these, installed at a fluid or paste consistency.

The performance characteristics of anchoring products are:

- pull-out resistance;
- chloride ion content;
- glass transition temperature;
- creep under tensile load.

As with any form of structural strengthening, appropriate guidance must be followed for the successful design and installation of anchors.

Part 7 – Reinforcement corrosion protection

EN 1504-7 is for active coatings and barrier coatings applied to reinforcing bar during a repair for the purpose of providing either an active coating on the reinforcement (Method 11.1) or a barrier coating of the reinforcement (Method 11.2). The coated bar is then overlaid with a repair mortar to EN 1504-3.

The primary performance characteristics of anchoring products are:

- corrosion protection;
- glass transition temperature;
- shear adhesion (of coated steel to concrete).

The function of barrier coatings was discussed under ‘Repair options and methods’ above, and while the products are likely to be effective to treat carbonated concrete, they must be selected with care when chloride-contaminated concrete is being partially repaired.

Part 2 – surface protection systems

EN 1504-2 is for products and systems for the surface protection of concrete and includes methods under the headings of several principles (see **Table 2**): Principle 1 – Protection against ingress; Principle 2 – Moisture control; Principle 5 – Increasing physical resistance; Principle 6 – Resistance to chemicals; Principle 8 – Increasing resistivity.

The Standard sets out the three types of surface protection system:

- Hydrophobic impregnation (H) – materials that penetrate into the outer few millimetres of the concrete surface and leave a water-repellent lining on the surface of the pores. They encourage the concrete surface to lose water by evaporation but do not prevent water ingress under significant hydrostatic pressure and do not normally reduce carbonation (and may increase it).
- Impregnation (I) – materials that penetrate into the outer millimetres of the concrete surface and react in the pores to seal them.
- Coatings (C) – materials that form a film on the concrete surface, rather than penetrate into it like (H) or (I) types. They adhere to the outer surface of the concrete and some have the ability to span active structural cracks beneath the film (crack-bridging coatings), typically to 0.3 mm crack width.

The performance requirements for the three types of coating used under the various principles and methods vary considerably.

- For Type (H), performance requirements include the depth of penetration into concrete test blocks, drying rates and resistance to diffusion of chloride ions.
- For Type (I), performance requirements include resistance to carbon dioxide transmission, ease of water vapour escape, resistance to impact loading for surface strengthening, and for slip and skid resistance dependent on the exposure (inside wet surfaces, inside dry and outside).
- For Type (C), performance requirements include resistance to carbon dioxide transmission; ease of water vapour escape; abrasion resistance, for protecting concrete floors; thermal compatibility of the film applied to concrete under a range of temperature cycles and split into trafficked or untrafficked adhesion; crack bridging ability at low temperatures; and impact resistance.

One of the common uses of surface protection systems is as a barrier to prevent carbonation of concrete. The performance requirement for an anti-carbonation coating is to have a resistance to carbon dioxide diffusion or ‘R-value’ of greater than 50 m and a resistance to water vapour transmission or ‘S_D-value’ of less than 4 m, which are parameters that have been proven effective over time (Robery, 1988). It should be noted that both the R-value and the S_D-value are related to the actual dry film thickness of a coating following application and care is required to ensure the necessary minimum dry film thickness is achieved on site.

Active corrosion management

As set out in ‘Repair options and methods’ above, some of the most effective means of treating structures that are suffering from reinforcement corrosion are the electrochemical methods of EN 1504, including:

- Method 7.3 – electrochemical realkalisation of carbonated concrete;
- Method 7.5 – electrochemical chloride extraction of contaminated concrete;
- Method 10.1 – providing cathodic protection through applying an electrical potential.

As can be seen in **Table 2**, these three methods are specified in other European standards, namely EN 12696 and EN 14038 Parts 1 and 2. However, the products and systems covered by EN 1504 are compatible with the requirements of structures treated in accordance with EN 12696-1 and EN 14038, although a few principal points need to be made in relation to the materials selected for repair.

As preparation before application of any electrochemical treatment, all areas of loose, cracked and delaminated concrete need first to be identified and then removed. The exposed reinforcing bar needs to be cleaned of loose adherent rust and a repair mortar applied to restore the profile. The following points are most important:

- Removal of adherent, flaking rust is necessary to identify whether any substantial loss of cross-section of the reinforcing bar has occurred and where necessary additional bar needs to be added.
- Full breakout behind the bar may not be necessary, particularly if the element is lightly loaded or of cosmetic use.
- No active or barrier coatings (Principle 11) should be used on the reinforcing bar, as this will interfere with the electrochemical process.
- Repair mortars and adhesives should be based on hydraulic cements (CC or PCC) which have an electrical resistivity similar to concrete and mortars based on synthetic resins (PC) should not be used.
- Whereas cathodic protection (Method 10.1) requires no additional surface protection, Methods 7.3 and 7.5 will normally require a suitable surface protection system to be applied after treatment to prevent carbonation in the future or to prevent fresh contamination by chloride ions respectively (Principle 1).

Electrochemical methods of repair need careful design by a competent practitioner and as a minimum in the UK would normally be a member of the Institute of Corrosion.

Part 10 – site application and quality control

EN 1504-10 gives details of site application methodology, including assessing the substrate before repair, as well as the quality control of the works. EN 1504-9 requires that a quality system is used by the repair contractor to ensure the specified quality requirements are met and that the correct repair principles, methods, products and systems are used. EN 1504-10 sets the quality standards for each element of the repair installation process, for the safety both of the structure and the operatives, and durability of the scheme. The contents of EN 1504-10 include a review of other parts of the EN 1504 series, and then specific guidance for each method relating to preparation of the substrate and reinforcement, application of the repair products and systems.

EN 1504-10 contains a list of 45 quality control tests and measurements that are specific to each of the repair methods covered by EN 1504, split into the following four categories:

- substrate conditions before and after preparation;
- acceptance of products and systems;
- condition and requirements before and/or during repair;
- final hardened condition (including mortars, adhesives and surface protection systems).

EN 1504-10 also provides extensive informative text, giving the background to the wording in the normative text and is a useful source of reference on inspection, diagnosis and repair practice.

While EN 1504-10 specifically guides the work of the installer, the designer will need to specify and direct the repair scheme with reference to its provisions. Particularly relevant examples include:

- *Structural stability during repair works* – the structural engineer needs to assess the effect of breakout and repair works by the installers on the residual structural capacity of each element and to specify appropriate temporary structural support as may be needed.
- *Aesthetic performance of the repair* – functional repairs, using materials that comply with EN 1504, may not result in an attractive appearance, due to difficulties in colour-matching the repair product or system to the original concrete (e.g. **Figure 8**). The designer will need to assess the owner's requirements for the building and specify final treatments that restore the aesthetic appearance, such as by application of a suitably decorative surface protection system.
- *Concrete removal* – specific technical, health and safety or environmental considerations for operatives and for occupiers of affected buildings will influence the selection of the concrete removal method, with options ranging from hydro-demolition to light mechanical breakers.

Appropriate arrangements should be made for a final acceptance inspection, forming a baseline survey against which the performance of the repair scheme can be compared over time.

Records and ongoing maintenance

To provide guidance for any subsequent maintenance or new works that are carried out in the future, the structure management plan should be updated with details of the works carried out. This would normally comprise as a minimum a record of the protection and repair works which have been carried out, including any test results, and instructions on any inspection and maintenance that needs to be undertaken during the remaining service life of the repaired part of the concrete structure.

Depending on the options, principles and methods chosen, parts of the protected or repaired concrete may have an expected service life that is short in comparison with the rest of the structure. Examples include surface protection systems, sealants, and weatherproofing materials. Should the integrity of the structure depend on the performance of the products and systems, then it is

Figure 8 Colour variation from applying different high-performance mortars over many years

essential that they are regularly inspected, tested and renewed if necessary.

The following list gives information for future maintenance which should be included:

- an estimate of the expected remaining design life of the concrete structure;
- identification of each product and system where the design life is expected to be less than the remaining design life of the concrete structure;
- the date at which each product and system is next to be inspected or tested;
- the method of inspection to be used, including how results are to be recorded and how future inspection dates are to be decided;
- a specification for systems with continuous treatment and monitoring, for example as used in an impressed current cathodic protection system;
- a statement of precautions to be taken or restrictions to be applied, for example maintenance of surface water drainage, maximum pressure for water washing or prohibition of the use of de-icing salt.

For the above reasons, it is normal that on completion of concrete repair works, a maintenance management system is implemented to ensure that the required future maintenance is carried out.

Health, safety and the environment

By following the requirements of EN 1504, using CE-marked products and systems, the specification for protection and repair should comply with the requirements of relevant health and safety, environmental protection and fire regulations. Where there is a conflict between the properties of specific products or systems and environmental protection or fire regulations, then alternative repair principles or methods may need to be considered.

Competence of personnel

Concrete repair has become a highly specialised and technical process, involving the use of complex materials that rely heavily upon sound technical knowledge of how the materials should be used and how not to abuse them. Any repair work therefore requires that personnel have the necessary skill and adequate equipment and resources to design, specify and execute the work and this is a basic requirement of EN 1504.

Personnel involved in the repair process, including the tasks of condition assessment, structural assessment, repair scheme design, repair execution and the inspection and assurance operations, should only be appointed once they have demonstrated they are knowledgeable in protection and repair works and recognised as competent.

Recommendations

Repair and rehabilitation play an important part in the effective management of concrete assets, but opinions have differed as to the best way to tackle a deteriorating concrete structure. Repair practice is rapidly changing, with a constant stream of new products and systems available, bringing a bewildering range of claims before the architect, designer and specifier.

The new European Standard EN 1504, *Products and Systems for the Protection and Repair of Concrete Structures*, has been developed in response to the Construction Products Directive to help the specifier choose the correct repair products and systems for the job. The Standard came into force on 31 December 2008. Conflicting national standards have been withdrawn by the relevant standards bodies and the requirements of the Construction Products Directive, as realised through the CE marking of products and systems which comply with the EN 1504 series, has begun. This is set to change the future direction of the concrete repair industry.

EN 1504 will affect the asset owner, as well as the specifier, contractor and manufacturer. For the first time a standard exists that addresses all stages of the repair process, from initial awareness of a problem, to the handover of a properly designed and executed repair of the building to a satisfied client, incorporating the use of products and systems that are approved as meeting minimum performance requirements for a range of repair applications. However, guidance is still needed on its use and interpretation. EN 1504 assumes the reader is competent in the field of concrete inspection, diagnosis, repair scheme development and material specification, which may not necessarily be the case. It is hoped that this chapter has set out some of the issues that need to be considered when implementing repairs to EN 1504.

Further details of EN 1504 may be obtained from the UK Concrete Repair Association website and joint CPA/Institute of Corrosion/Concrete Society publications.

Appendix: Selected test standards called up in EN 1504-3 (for a complete list see Davies and Robery, 2006)

EN 1542. *Measurement of Bond Strength by Pull-off*.
EN 1766. *Reference Concretes for Testing*.
EN 1770. *Determination of Coefficient of Thermal Expansion*.
EN 12190. *Determination of Compressive Strength of Repair Mortar*.
EN 12617-4: Part 4: *Determination of Shrinkage and Expansion*.
EN 13057. *Determination of Resistance of Capillary Absorption*.
EN 13036-4. *Surface Characteristics – Test Methods* – Part 4: *Method for Measurement of Skid Resistance of a Surface: The Pendulum Test*.
EN 13295. *Determination of Resistance to Carbonation*.
EN 13395-1. *Determination of Workability* – Part 1: *Test of Flow of Thixotropic Repair Mortars*.
EN 13395-2. *Determination of Workability* – Part 2: *Test for Flow of Grout or Mortar*.
EN 13395-3. *Determination of Workability* – Part 3: *Test for Flow of Repair Concrete*.
EN 13395-4. *Determination of Workability* – Part 4: *Application of Repair Mortar Overhead*.
EN 13396. *Measurement of Chloride Ion Ingress*.
EN 13412. *Determination of Modulus of Elasticity in Compression*.
EN 13529. *Resistance to Severe Chemical Attack*.
EN 13584. *Creep in Compression*.
EN 13687-1. *Determination of Thermal Compatibility* – Part 1: *Freeze–thaw Cycling with De-Icing Salt Immersion*.
EN 13687-2. *Determination of Thermal Compatibility* – Part 2: *Thunder Shower Cycling (Thermal Shock)*.
EN 13687-4. *Determination of Thermal Compatibility* – Part 4: *Dry Thermal Cycling*.
EN 14629. *Determination of Chloride Content in Hardened Concrete*.
EN 14630. *Determination of Carbonation Depth in Hardened Concrete by the Phenolphthalein Method*.

Note: In the above list of references, the European Standards are issued by national standards bodies in the CEN member states. In the UK, this is the British Standards Institution (BSI) and so all EN Standards in the UK are prefixed with the term BS (i.e. BS EN 1504). Voluntary standards are prefixed by BSI with the letters DD (i.e. DD ENV 1504: 1997).

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CONTENTS

Chapter 21

Future developments of concrete in the construction materials industry

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Future developments in the construction industry will be characterised by the increasing demand for integral solutions based on process optimisation driven by the design of novel materials with a strong consideration for sustainable development and positive environmental impact. Consequently, the different players in the construction business will face the inevitable challenges associated with the transformation of a conservative industry to one where the introduction of innovations becomes increasingly important due to the need for solutions that satisfy more demanding requirements, be it regulatory or performance driven. Future requirements in the area of concrete technology will likely be concerned with: (a) process improvement – how to produce, transport, and collocate concrete more efficiently; (b) material performance – lighter structures, rapid strength development, improved workability retention and enhanced durability; and (c) environmental impact – reduction of CO_2 footprint, use of recycled waste materials, and energy-efficient buildings. The aforementioned evolution in concrete technology is driven by the increasing focus that will be given to process optimisations that provide solutions: it will no longer be sufficient to supply a product (e.g. concrete), but it will be necessary to present a solution that, when utilising a specifically engineered product, renders the entire process that makes use of it more efficient. Consequently, the different sectors of the industry, be it materials suppliers, contractors, architects, and engineers, will have to learn to work closer together and not as operational islands. This will enable clearer and more efficient establishment of the construction requirements that will lead to a holistic-driven approach in providing solutions.

Introduction

Currently, the construction industry is highly fragmented, characterised by disorganised processes (e.g. the assumption of responsibility), adversarial relationships and inter-dependence of sectors that in the majority of cases operate independently (Jones and Saad, 2003). Primarily conservative and driven by cost leadership, it is challenging to introduce innovations in the construction industry. An example of the latter is self-consolidating concrete (SCC), which makes use of the most advanced dispersing admixture technology (polycarboxylate-based superplasticisers) to produce a material that possesses the following advantages: high flow (>65 cm), no need for vibration (self-levelling and self-compacting) – reduction of labour requirements at the jobsite, shorter cast time, more durable (low water/cement ratios) and high early-strength development (>4 MPa at 8 hours). Despite the advantages in rheological and mechanical performance as well as the ease of application, in the years 2006 and 2007, SCC made up only 1% of the total concrete production (2006, $\sim$310 Mm^3; 2007, $\sim$321 Mm^3) in Europe (ERMCO, 2006, 2007) (see **Figure 1**).

Recently, important movements in the construction industry have evidenced the increasing trend for companies to vertically integrate in the construction materials sector (cement, aggregates, concrete, asphalt and admixtures). This vertical integration has strategically oriented the companies towards providing customers with a holistic approach focused on the integration of as many as possible processes in the proposal of solutions. The challenge will be to understand the level of maturity in the different markets, its demands, as well as the drivers in order to offer sensible and cost-effective solutions without compromising quality. Emerging markets, where demanding constraints pose a challenge for construction projects, will benefit from the integral solution-based approach and this will likely lead to the acceleration of its development.

Recently, a heightened importance and awareness has been given to the environment and developments that are sustainable. Consequently, this will require the development of products that can make optimum use of recycled and/or waste materials as well as the development of eco-efficient solutions. An example of eco-efficiency is the initiative of the World Business Council (http://www.wbcsd.org) to develop eco-efficient buildings (EEBs) producing zero net energy. Surely, such initiatives can be successful only if the building design and erection procedures contemplate processes and materials that will satisfy the requirements for achieving the eco-efficiency of the structure, and, more

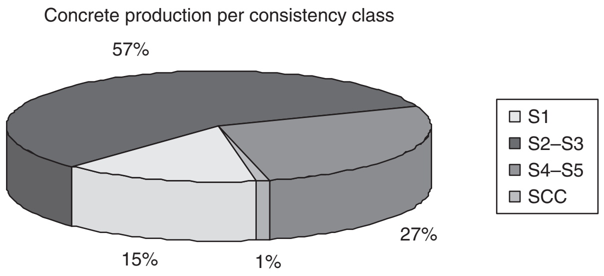


Figure 1 The types of concrete sold in the European market based on consistency class: S1 = slump 10–40 mm, S2 = slump 50–90 mm, S3 = slump 100–150 mm, S4 = slump 160–210 mm, S5 = slump ⩾220

importantly, if there are strong incentives to promote the selection of such environmentally favourable and currently expensive solutions.

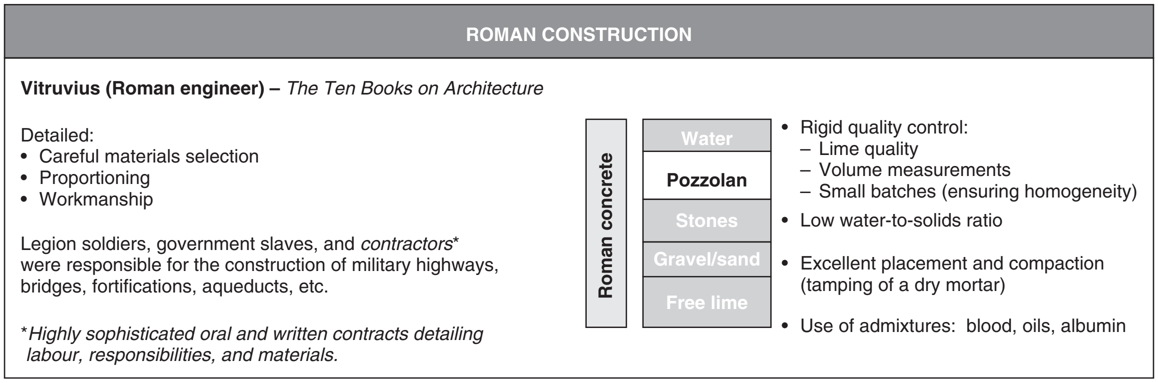
Moving towards a solutions-oriented approach

The needs of the construction industry will shift towards cost-effective, efficient and environmentally friendly solutions using state-of-the-art technology that generates an improvement in the overall process in the production of products.

In the attempt to delineate the road map of the industry, the author finds it ironic that one has to look to the past for future trends in the construction industry despite living in the year 2009. By the year 1 AD, the Romans had been working with concrete more than 'modern man' with cement (Herring, 2002). The Romans were outstanding engineers who had a clear understanding of *process optimisation*. **Figure 2** summarises the processes at the core of Roman construction technology: product development, quality control, and contracting.

The Romans showed a heightened awareness for product development and the importance of quality through the careful selection of materials as the concrete constituents. All of the facets and quality of the construction process were guaranteed through formal contracts, where contractors assumed responsibility for the labour, quality and materials. Today's construction industry is well removed from the highly organised Roman model for the execution of construction projects. Currently, in the construction industry, the responsibilities are poorly defined, there is often a lack of coordination between suppliers, engineers/architects, and contractors, and processes are developed in an isolated manner. However, the construction industry requirements are increasingly becoming more demanding with needs for manufacturing methods that shorten the construction cycle, materials with advance performance as well as multiple property advantages, and a positive impact on environment that will benefit generations to come. **Figure 3** provides an overview of the aforementioned future requirements in concrete technology.

In order to make such requirements possible, it will be necessary for all of the industry players (suppliers, designers, contractors and construction owners) to have an overview of the processes required for the development and execution of a project. Advances in the industry can be only achieved through the continuous improvement and optimisation of the factors that constitute a process: product development, productivity and industrialisation. Product development is the ability to propose novel materials through innovation that will satisfy the need for products with better performance, higher quality and/or lower cost. Productivity refers to the generation of means or methods that will require fewer resources and less time to generate the same if not higher throughput. Whereas, industrialisation is the increasing incorporation of automation to substitute labour and/or improve manufacturing. Therefore, the solution-oriented approach that the industry will adopt will increasingly rely on the continuous process optimisation – the evolution of technology will require a review of the constituents of a process and identification of obsolete or weak steps that can be improved.


ROMAN CONSTRUCTION

Vitruvius (Roman engineer) – *The Ten Books on Architecture*

Detailed:
- Careful materials selection
- Proportioning
- Workmanship

Legion soldiers, government slaves, and *contractors** were responsible for the construction of military highways, bridges, fortifications, aqueducts, etc.

**Highly sophisticated oral and written contracts detailing labour, responsibilities, and materials.*

- Rigid quality control:
  - – Lime quality
  - – Volume measurements
  - – Small batches (ensuring homogeneity)
- Low water-to-solids ratio
- Excellent placement and compaction (tamping of a dry mortar)
- Use of admixtures: blood, oils, albumin

Figure 2 An overview of Roman construction practices: materials selection and processing, quality control, workmanship and responsibilities

CONCRETE TECHNOLOGY
Future requirements

Process improvement

All construction processes will be accelerated and rendered more efficient:

- Concrete manufacturing.
- Transportation and collocation.
- Formwork (moulding) and de-moulding.

Material performance

- Lighter structures with small sections.
- Early and ultimate strength development.
- Ultra-high strength – ductility comparable to steel.
- Total durability (holistic approach).

Environmental impact

- Use of recycled waste materials in cement and concrete production: *disposal of demolished concrete will force the recycling of aggregate and avoid the generation of waste.*
- Lower clinker per m^3 of concrete – use of supplementary cementing materials (SCM) in concrete to reduce CO_2 emissions.
- Constraints in aggregate excavation from rivers – need to find alternative solutions.

Figure 3 Future trends in the construction industry – concrete technology: materials manufacturing, materials performance improvement and environmentally friendly

A solutions-based approach through the optimisation of process can be best illustrated through an example that shows great promise for the future of house construction. At the University of Southern California, Dr Behrokh Khoshnevis (http://www.contourcrafting.org) is proposing a novel and unique solution for the construction of structures that is based on a 'layered fabrication' technology termed contour crafting (CC). Dissecting the CC technology, one realizes that its successful implementation is dependent on the optimisation of the above-mentioned factors that make up a process. In the case of CC, product development, productivity and industrialisation are highly interdependent. The technology relies on fully automated equipment that is able to 'extrude' a material in a continuous manner with uniform properties, and through the movement of the CC machine, elements of the structure are erected with the desired shape and surface finish (**Figure 4**). Such a technology can radically decrease the time it takes to erect a structure and increase productivity – i.e. construction throughput. The CC has also the additional advantage that it enables the incorporation of conduits for electricity, water and air-conditioning in the manufacturing process. However, productivity and efficiency of the automated system is highly dependent on the material properties. Therefore, it is important to develop a product/material that can immediately hold its shape once extruded, develop strength rapidly, has low shrinkage and gives a good surface finish. Hence, the success of the CC process depends on the 'design' of the material to be utilised.

Materials by design

It is the author's belief that one of the critical factors in making a solutions-based proposition successful is the ability to engineer *materials by design*. Currently, the approach utilised in the development of materials, and eventually products, in the construction industry is still quite rudimentary and often based on empirical data or experience. Much can be learned by taking the steel industry as a reference. Metals are designed adopting an approach based on materials science (**Figure 5**) where the constituents of a material are processed in a specific and controlled manner in order to obtain a determined

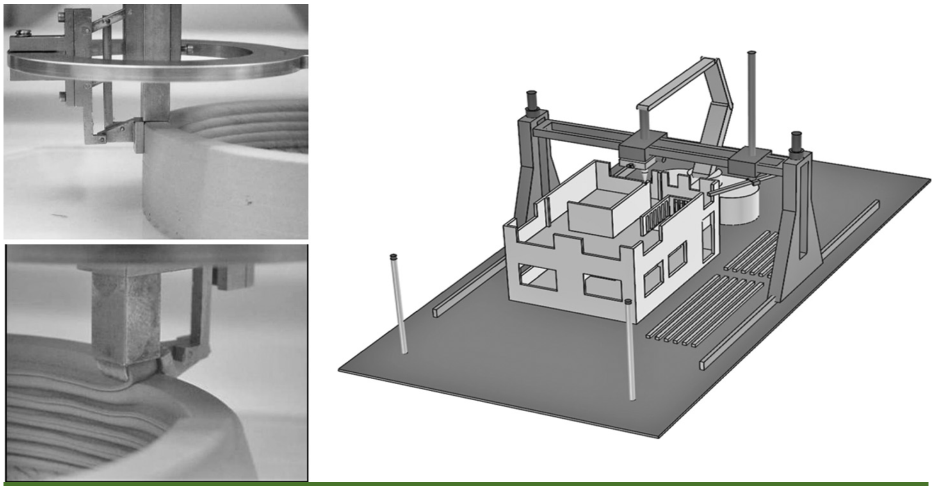


Figure 4 Contour crafting (CC) – an innovative solution for the construction of structures (courtesy of http://www.contourcrafting.org)

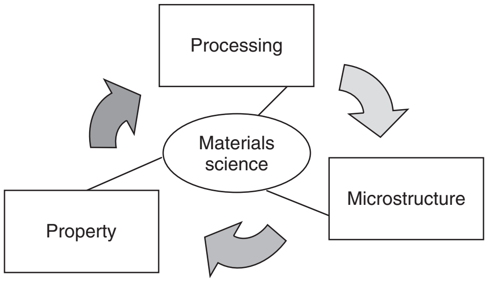
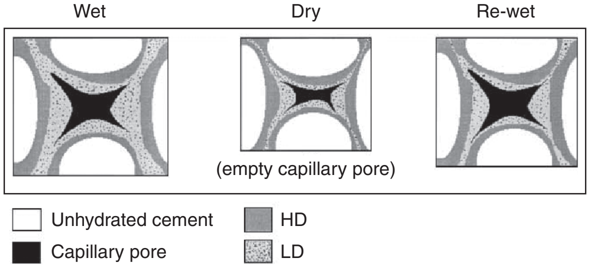


Figure 5 The materials science approach in engineering and development of materials



Schematic diagram of cement paste shrinkage during drying at 50% RH: capillary pores and LD compact when dried, while HD and unhydrated cement (restraining phases) do not deform; on rewetting, capillary pores regain original shape, while LD does not.

Figure 6 Influence of microstructural features on the drying shrinkage and rewetting of cement paste (reprinted from M. Juenger and H. Jennings, Cement and Concrete Research ©2002 Elsevier)

microstructure; in turn, this imparts to the material the desired properties.

For instance, in the construction materials industry, very little consideration is given or control exercised over the processing parameters in the production of a material. As materials become more advanced in performance, more control needs to be applied in the production process in order to achieve the targeted behaviour of the material – be it physical or chemical. An example of this lack of consideration for processing is apparent in the production of ready-mix concrete. Currently, ready-mix concrete is produced by a rather simple mixing procedure where its constituents (sand, gravel, cement, water, admixtures and others) are simply 'thrown' together in a mixer (concrete truck or a pan mixer and then transported to the targeted jobsite) until a homogeneous material is obtained. No consideration is given to the mixing parameters such as time, energy, sequence of addition, time of addition of the various constituents, mixer type, etc. The precedent is just one of the many examples that evidence the potential that exists for the development of construction materials with tailored characteristics if one begins to explore how to apply the materials science approach. Furthermore, the engineering of new construction materials can be achieved through the development of creative and innovative use of fibres, aggregates, alternative processing techniques, chemical admixtures, novel binders, mineral additions and other novel developments. The engineering of the cement-based materials is also possible at different scales, ranging from the nano to meso to macro scale. Fundamental to the advances in materials engineering and design will be the investment and dedication of the industry to research efforts targeted at understanding the fundamentals required for producing materials in a controlled manner.

An example of how fundamental knowledge could foster the controlled design of materials is the investigation by Garci-Juenger and Jennings (2002). In their study, it is shown that, of the possible mechanisms responsible for influencing the rate and extent of drying shrinkage, one is related to the morphology of C–S–H. Tennis and Jennings (2000) have proposed a microstructure model for cement that suggests the existence of two types of C–S–H: high density (HD) and low density (LD). **Figure 6** schematically shows how HD C–S–H does not deform when subjected to drying at a relative humidity (RH) of 50%.

Therefore, the implications of the study's results are that one could control drying shrinkage through the manipulation of HD C–S–H formation. One way to alter the microstructure and consequently the properties of cement-based materials is through use of chemical admixtures.

The utilisation of chemical admixtures in concrete has experienced a steady increase, and trends indicate that worldwide consumption will continue to increase at a rapid pace: growth forecast between 2006 and 2010 is approximately 5%. Chemical admixtures play an important role in tailoring the properties and performance of cement-based materials, and therefore are fundamental to the future developments and innovations in the area of concrete technology (Zampini and Niklaus, 2007). The diagram in **Figure 7** depicts the potential impact of admixtures on various phases that constitute concrete technology. Developments in the area of chemical admixtures will have tremendous impact on cement and concrete innovations, because the properties of the aforementioned construction materials can be controlled but, more importantly, tailored to optimise a process.

Another area that is receiving great interest in terms of future developments and related to design of future materials is the investigation of alternative binder systems to cement. Currently, the cement industry is accountable for roughly 5% of the total emission of CO_2 (1 t of cement produces approximately 1 t of CO_2). Compared with other industries, the CO_2 contribution to the atmosphere is not high, but the construction industry is taking an active role in proposing environmentally sustainable solutions. Binders of interest in the future will most likely be belitic cements, sulfoaluminate cements and geopolymers.

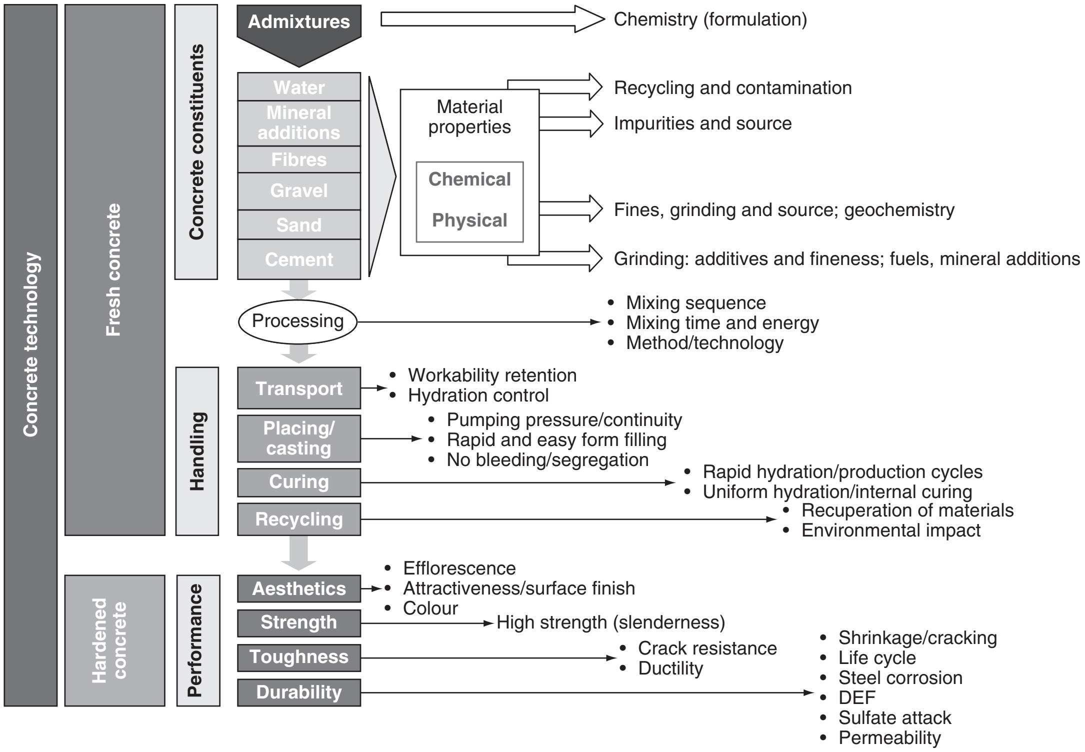


Figure 7 Examples of how chemical admixtures can impact concrete technology

As can be seen in **Table 1**, the production of cements rich in belite or sulfoaluminates leads to an important reduction in CO_2 emissions. It should be noted that sulfoaluminate cements are also easier to grind, thereby reducing the energy consumption during the milling process. The challenge in the production of belitic cements will be to develop means to render the phase more reactive in the early stages of hydration – the belite (C_2S) phase is responsible for imparting long-term strength to cement-based materials, but in the first 28 days it is the alite (C_3S) that primarily contributes to strength development. Thus, the production of a more reactive belite will lead to cement with a lower CO_2 emission and energy consumption.

Some of the potential applications of sulfoaluminate cements are ready mix (repair work and flooring, dry mortars); self-levelling screeds, repair/restoration work and flooring, glass-fibre-reinforced products (no risk of alkali–silica reaction ASR), marine concrete, and sprayed applications. However, the potential applications of sulphoaluminate cement still face challenges since the material sets rapidly (i.e. short open time), and therefore admixture developments that can control rheological properties will be fundamental for favouring the use of such cement.

Compound	Phase	ΔH formation: kJ/kg	CO_2 release: kg/mol of product
C_3S	Alite	1848	0.58
CA	Calcium aluminate	1030	0.28
β-C_2S	Belite	1030	0.51
C_4A_3S	Calcium sulfoaluminate	800	0.22

Table 1 Amount of CO_2 generated in the production of phases that are present in OPC, belitic, and sulphoaluminate cements (adapted from Zhang and Glasser, 2002)

Another binder system of great interest for the future of the construction industry is geopolymers, which were first proposed by Davidovits in 1978 (Davidovits, 1994; http://www.geopolymer.org). Materials that can be utilised as geopolymers are typically composed of silico-aluminate compounds. Geopolymers are often referred to as poly(sialates), a designation that has been given by Davidovits, where sialate is an abbreviation for silicon-oxo-aluminate. Poly(sialates) consists of inorganic polymers consisting of chain and ring structures with Si^{4+} and Al^{3+} in fourfold coordination with oxygen. Poly(sialates) with empirical formula M_n $(–(SiO_2)_z–AlO_2)_n \cdot wH_2O$ (z: 1–32; M monovalent cation – K or Na; n: degree of polycondensation) can range from semi-crystalline to amorphous. As indicated in **Figure 8**, poly(sialates) exist as three distinct types, and it is the polycondensation reaction between these poly(sialates) with a source of alkalis or alkali polysilicates that produces a geopolymerisation reaction consisting of Si–O–Al bonds.

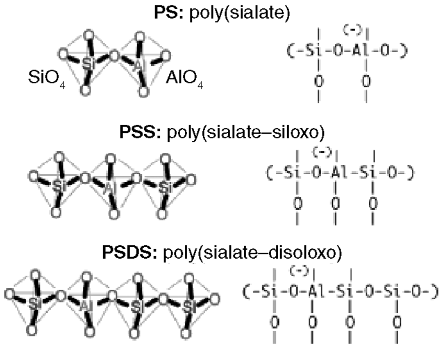


Figure 8 The different types of poly(sialate) – (courtesy of http://www.geopolymer.org)

Si : Al ratio	Applications
1	Bricks Ceramics Fire protection
2	Low CO_2 cements and concretes Radioactive and toxic waste encapsulation
3	Fire protection fibre-glass-reinforced composite Foundry equipment Heat-resistant composites (200–1000°C) Tooling for aeronautics titanium process
>3	Sealants for industry (200–600°C) Tooling for aeronautics SPF aluminium
20–35	Fire- and heat-resistant fibre-reinforced composite

Table 2 Potential uses of geopolymers. Only through controlled design of the material can the various applications can be achieved (adapted from Wallah and Rangan, 2006)

The formation of geopolymer-based binders, unlike ordinary Portland cement (OPC), does not form C–S–H, but relies on polycondensation of materials rich in silica and alumina combined with a source of alkalis (**Figure 9**).

As mentioned previously, geopolymers are the result of the reaction of materials rich in silica (Si) and alumina (Al) and alkalis (Na and K). Interestingly, there is a large range of minerals that are present in nature, which lend themselves well to the production of geopolymers. Xu and van Deventer (2000) investigated 16 different natural materials that included ring, chain, sheet and framework crystal structures. Several Si–Al minerals were identified as possible candidates for the production of geopolymers: garnet, mica, clay, feldspar, sodalite, zeolite, andalousite, spinel and kaolinite. In addition to the earth's mineralogy, there are also Si- and Al-rich waste products such as fly ash and slag that can be used for the synthesis of geopolymers. One of the limitations in the use the aforementioned materials are factors such as availability, cost and type of application. Despite the environmental advantages of geopolymer-based binders, there is still much to be learned about the chemical and physical factors that influence the reactions. For instance, it has been shown by van Jaarsveld *et al.* (2002, 2003) that the particle size, calcium content, alkali metal content, degree of crystallinity, morphology and origin of fly ash affect the geopolymerisation reactions. Thus, the design of materials using geopolymers is still in the developmental phase due to the difficulty in controlling the reactions. Despite the challenges, potential areas of application for geopolymers have been identified and are summarised in **Table 2**.

An area of science and engineering that will accelerate the development and application of novel materials is computational materials science – the use of computer modelling to design microstructures and predict properties. Geoffrey Frohnsdorff and co-workers (Frohnsdorff *et al.*, 1995)

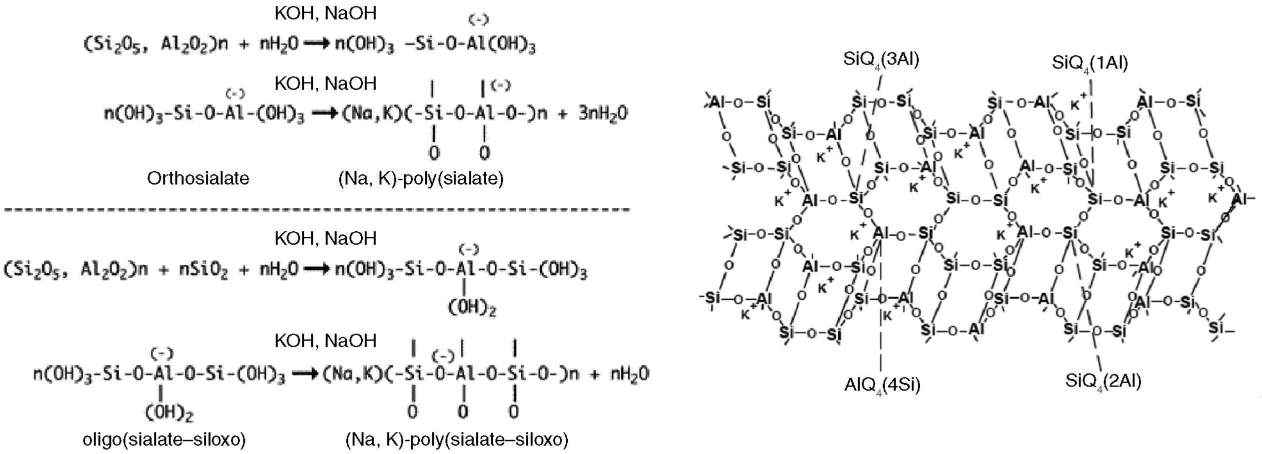


Figure 9 Examples of polycondensation reaction and the proposed structure for K-poly(sialate–siloxo) – (courtesy of http://www.geopolymer.org and Davidovits, 1994)

stated the following on usefulness of computerised knowledge: 'The computerisation of knowledge will be among the most important technological developments because of the pervasiveness of its impacts: it will benefit most technologies, including those of cement and concrete.' In fact, the ever-increasing power of computers has made it possible to store, manipulate, transfer and perform intensive calculations more rapidly and efficiently, thereby accelerating as well as facilitating the advancement of computational materials science. Computational models will enable material designers to explore 'what if?' questions, and observe in a short time (hours) the development of microstructure and evaluate the prediction of properties given specific conditions. The powerfulness of a model is only as good as the knowledge underlying the mechanisms responsible for the microstructure development. Therefore, efforts must be made to foster scientific knowledge related to materials.

This section highlights the importance of adopting a materials science approach in the development of novel construction materials. The possibilities are numerous, and it is important to have the know-how that enables the intelligent use of materials for the elaboration of solutions through the optimisation of processes.

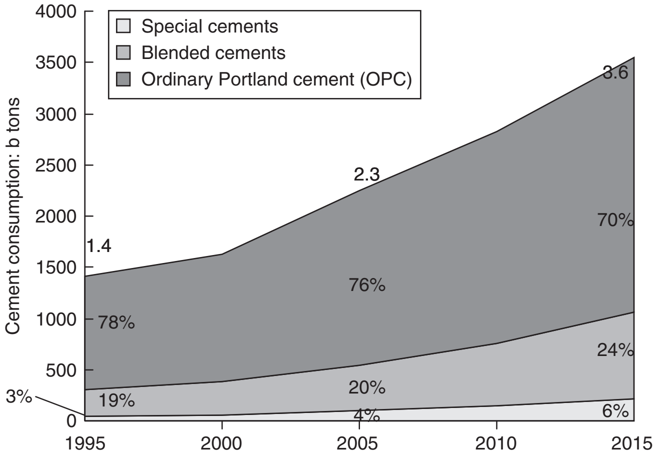


Figure 10 Predicted trends for the consumption of special cements, blended cements, and ordinary Portland cement (OPC) from 1995 until 2015

Environmental impact and sustainability

The graph in **Figure 10** shows the predicted worldwide consumption of cement, and it is evident that in the future blended cements will acquire an increasing importance. The driving force behind this trend is the ever-increasing need to reduce CO_2 emissions as explained above. In addition to the blended cements, the aforementioned geopolymers would constitute the special cements, which would also include sulphoaluminate cements, belitic cements and calcium aluminate cements.

Often associated with the use of materials, products, buildings and processes are environmental and energy burdens that can be assessed by applying a life-cycle analysis (LCA). An important three-year LCA-based initiative, taken on by the World Business Council for Sustainable Development since 2006, is the Energy Efficiency in Buildings (EBB) project, where the goal is to achieve the construction of buildings with zero net energy consumption (www.wbcsd.org). Today the awareness of the environmental impact of buildings (40% of worldwide energy consumption is from buildings) in different markets is high (in most countries above 80% except for India and Japan), but the involvement in activities related to green buildings is 10% or less (except for the USA and Germany). For instance, the market perception of green buildings is characterised by low awareness of environmental impact and the cost of green, lack of leadership – the sector is complex and fragmented, and there is poor know-how with little experience. Consequently, from a business perspective view the initiative is unattractive due to lack of financial instruments, energy efficiency not being an investment criterion, behavioural and cultural factors not being favourable, and the need for a holistic approach. The graph in **Figure 11** is an interesting plot

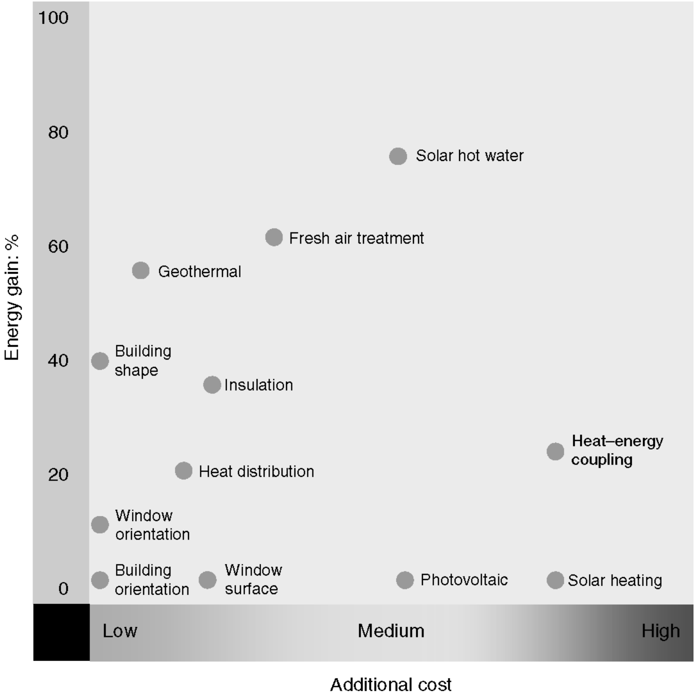


Figure 11 Impact of the choice of technology on building energy efficiency (courtesy of http://www.minergie.ch)

(http://www.minergie.ch) showing how different energy-saving solutions impact the cost of a building.

An important message from the graph in **Figure 11** is the fundamental role of a technology-driven holistic approach towards net zero energy consumption in buildings. Such energy attractive buildings are not far out of reach, and there are increasing examples of housing and building concepts that target environmentally sustainable and energy-efficient solutions. The challenge will be to make such initiatives accessible to everyone.

Conclusion

As repeatedly pointed out, the construction industry is highly fragmented, and only through the encouragement and strong drive towards a holistic approach can important innovative solutions be generated. An integral design process involving owners, architects, engineers, suppliers and others will enable the construction of structures and buildings with improved building performance, lower costs, and fewer disruptive changes. Looking into the future of the construction industry and specifically the construction materials sector, there will be emphasis on the production of materials that are engineered by design, tailored not only to meet the performance requirements but also to optimise the processes associated with its use. Computational materials science will increasingly play an important role in the design and engineering of novel cement-based materials. Furthermore, the environmental stresses affecting our planet will also motivate the research for alternative mineral-based binders that contribute to reducing CO_2 emissions. The use of future materials in the construction process will have to contemplate their contribution to making a building more energy efficient and more durable, the ease of recycling and lower CO_2 emissions. Forecasts in admixtures and cement consumptions indicate that there will be an important growth in the construction sector. Consequently, the players in the construction industry will need to place their efforts in how to better develop new materials to provide holistic use as well as solutions that are sustainable, thereby allowing future generations to reap the benefits.

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Chapter 22

Asphalts: an introduction

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Asphalt has a long and successful track record as a material used in road construction and maintenance. It typically comprises graded aggregate mixed with a bituminous binder which results in a strong but flexible material with good durability. A range of asphalt mixtures can be produced by varying the gradation of the aggregate skeleton and the grade (hardness) of the bituminous binder. In road construction, asphalt is typically used in the surfacing and the main structural layers (binder course and base) where it helps spread the traffic loading and protects the weaker lower-layer materials. Modern asphalt surfacings have been shown to reduce traffic noise and improve safety as they provide excellent levels of skid resistance in both dry and wet weather. According to the UK government 'sustainability is the future' and, because asphalt is 100% recyclable and reusable, its use in road construction results in a highly sustainable solution.

Background

Asphalt has been used in road construction since the early 1800s. Initially tar was used as the binder which was later replaced by bitumen which is refined from crude oil. Today, a wide range of asphalts can be produced to suit different conditions including asphalts made using lower-temperature binders such as foamed bitumen and bitumen emulsion.

The asphalt industry in the UK produces in excess of 25 million tonnes annually of which approximately 50% is used for the surface course and approximately 50% is used for the lower road layers (European Asphalt Pavement Association, 2007).

Physical and mechanical properties

Asphalt typically comprises a bituminous binder mixed with a graded aggregate. The properties of an asphalt will depend on the type and grade of the binder used and the type and gradation of the aggregates.

Bitumen

The origins of bitumen as an engineering material date from 3800–3000 BC when it was used as a mortar for masonry and waterproofing. Bitumen is distilled from crude oil which is a sophisticated process requiring extensive expertise and considerable financial investment to provide consistent products meeting European Specifications (BS EN 12591: 2000). In most engineering applications of bitumens, their inherent temperature susceptibility is utilised to ensure that they are sufficiently fluid during application and sufficiently stiff when in use. To increase their fluidity for some applications, penetration-grade bitumens may be used as 'emulsions' which are dispersions of bitumen in water, designed to 'break' leaving a continuous film of stiff residual bitumen on application. A large range of penetration-grade bitumens can be produced to suit different circumstances and the addition of polymers can be used to produce polymer modified bitumens (PMBs) with enhanced mechanical properties.

As noted above, bitumen is an extremely temperature-dependent material whose response also depends on loading time. One of the most important qualities, in terms of bitumen performance, is the temperature–viscosity relationship. During application the bitumen must be: (1) fluid enough to coat the aggregate; (2) viscous enough not to run off the aggregate during mixing or transport; (3) fluid enough for the mixture to remain workable during compaction; and (4) viscous enough to carry the traffic. To achieve these requirements, the operating temperature must be selected to ensure that the appropriate viscosity is attained. The Bitumen Test Data Chart (BTDC) was developed for showing the effect of temperature on the mechanical behaviour of bitumens. An example from Read and Whiteoak (2003) for a dense bitumen macadam, manufactured using a 200 pen grade bitumen is shown in **Figure 1**. It can be seen from this figure that the bitumen viscosity should be 0.2 Pas for satisfactory coating and between 2 Pas and 20 Pas for optimum compaction.

Aggregates

Aggregates used in asphalt production can be divided into three main groups: (1) natural aggregates; (2) artificial or synthetic aggregates; and (3) recycled aggregates.

Natural aggregates used in asphalt production come from one of the following rock types: (1) igneous; (2) sedimentary; and (3) metamorphic. Igneous rocks were formed by volcanic or magmatic action and possess a wide range of chemical composition, grain size and texture. Examples are granites and diorites. Sedimentary rocks were typically formed as a result of weathering and subsequent deposition

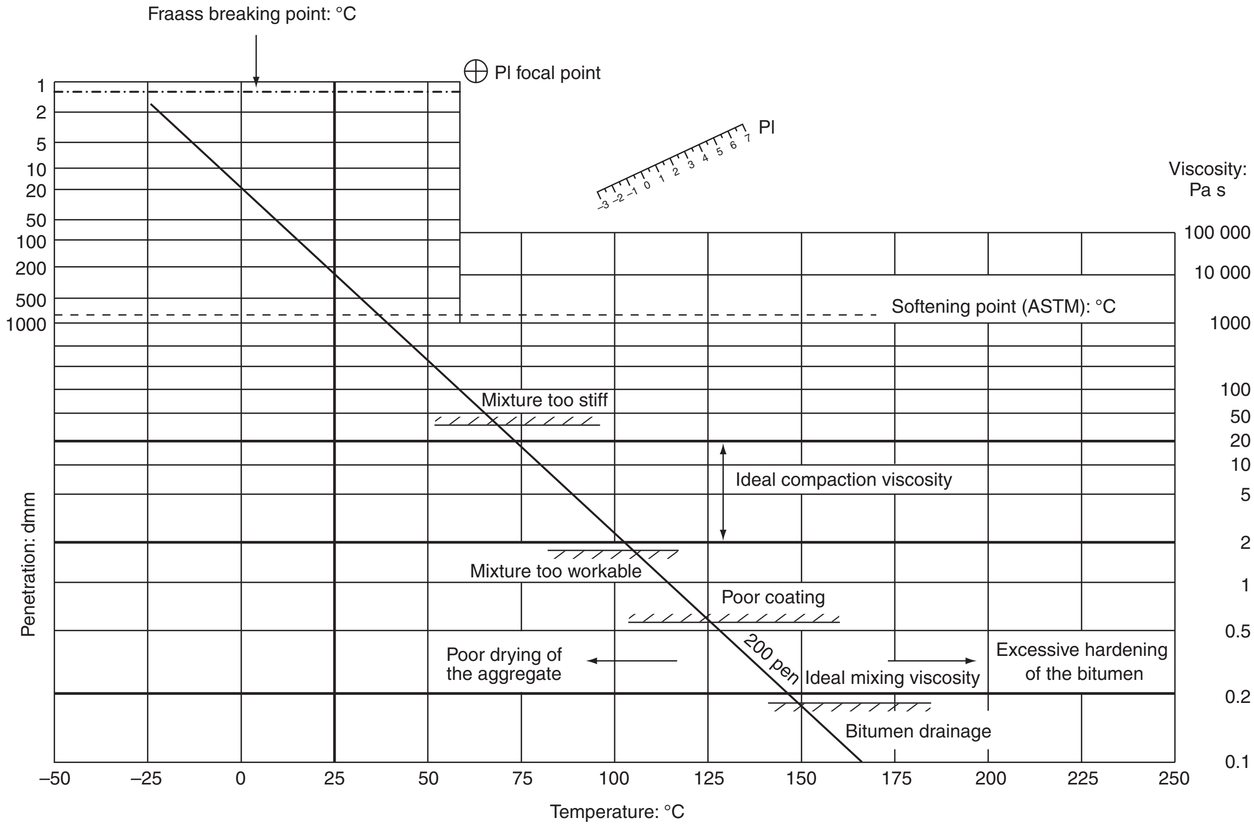

Figure 1 Bitumen test data chart (reproduced with the kind permission of Shell International Petroleum Company, part of the Royal Dutch Shell Group)

into stratified layers which were consolidated or cemented by chemical action. Examples are sandstone and gritstone. Metamorphic rock is the result of alteration of existing sedimentary or metamorphic rock by heat, pressure or chemical activity. Examples are marble and quartzite.

Artificial or synthetic aggregates are materials where there has been some human involvement. Examples include slags, which are a by-product of the steel industry and calcined bauxite which is used in high-friction surfacings. Recycled aggregates include materials such as recycled asphalt planings (RAP) and concrete and demolition waste.

There are a number of aggregate properties that are relevant to asphalt mixtures specified in the European Standard (BS EN 13043: 2002). For example, polishing resistance, abrasion resistance and resistance to fragmentation and impact are all assessed. Two of the key aggregate properties associated with skid resistance in surfacing materials are the polished stone value (PSV), which is a measure of the resistance to polishing, and the aggregate abrasion value (AAV), which is a measure of the ease with which an aggregate will abrade.

Asphalt

A wide range of asphalt mixtures can be produced depending on the gradation of the aggregate and the type and grade of the bituminous binder. The particle sizes of natural aggregates are often suitable for the production of asphalt without the need for mechanical size reduction although, in some cases, the larger sizes must be crushed. It is generally considered that crushed aggregates are more desirable as the angular particle shape contributes to mechanical strength, particularly resistance to rutting. Blastfurnace and steel slags are still used in asphalt but the decline of the steel industry means that these aggregates are not so common as in the past.

The particle size of aggregates used for asphalts can typically range from 37.5 mm down to fine dust and the grading or distribution of particle sizes is determined by carrying out a sieve analysis. The maximum aggregate size depends on the type of mixture, the application for which it is used and the thickness of the layer. Base materials, which are typically laid in lifts of 100 mm or more, may have a maximum aggregate size of 20 mm or 37.5 mm whereas surfacing mixtures may have a maximum aggregate size of 14 mm or 10 mm. Weight for weight, a fine aggregate has a greater surface area than a coarse aggregate. As it is necessary to coat all the aggregate particles in a mixture with bitumen it follows that more bitumen must be used with mixtures that contain higher percentages of fines and filler. Some of the more common types of asphalt are discussed in the following paragraphs.

Continuously graded mixtures (known as asphalt concretes) contain all the aggregate sizes from the maximum down to filler and are specified in BS EN 13108-1: 2006. This type of mixture results in a dense interlocking aggregate skeleton and relies less on the properties of the bitumen for its strength.

Gap-graded mixtures consist of a fines–bitumen mortar which, in the case of a surfacing mixture, also contains added filler. The fines are generally natural sand in contrast to the crushed material normally used in asphalt concretes. The mortar is mixed with a single-sized coarse aggregate to provide a grading in which the intermediate sizes are missing (i.e. there is a gap in the grading). In the UK such material is known as 'hot-rolled asphalt' which is specified in BS EN 13108-4: 2006. Typically, for base and binder course mixtures the coarse aggregate content is increased so that particles are in contact providing an aggregate skeleton which has structural strength. The bitumen content is higher than that of the asphalt concrete due to the higher surface area of the fines.

A major contributory factor in many wet road accidents is spray from fast-moving vehicles. A surfacing material which can dramatically reduce spray and the standing water which causes aquaplaning and skidding is porous asphalt. Porous asphalt consists of a single-sized coarse aggregate with a little fines, filler and bitumen, is laid over an impervious surface. The void content of freshly laid porous asphalt can be over 20% and water will pass through it and drain off laterally. Porous asphalt has the additional advantage that it can significantly reduce traffic noise. It is specified in BS EN 13108-7: 2006. The disadvantage of porous asphalt is that, because of the open nature of the material, the bitumen can oxidise relatively quickly and water can cause the bitumen to strip from the stone. If the aggregate can be coated with thicker films of bitumen, these problems could be reduced. Solutions include the use of bitumen modifiers and the addition of fibres to the mixture. Another problem with porous asphalt is that the voids can eventually become clogged.

Stone mastic asphalt (SMA) can be classified as gap-graded material, consisting of a coarse aggregate skeleton in which the aggregate particles are in contact. The space between the coarse aggregate particles is filled with a fines–filler–bitumen mortar. Care is taken in the design of the mixture to ensure that the mortar just fills the space between the coarse aggregate particles. SMA is a durable material with high stability (rutting and deformation resistance) due to the stone–stone interlock of its coarse aggregate skeleton. It is specified in BS EN 13108-5: 2006.

Sustainability and recycling

Asphalt is a highly sustainable product, in fact it is 100% recyclable or reusable in some way. For example, it can be recycled back into asphalt or reused as a fill or sub-base material. There are a number of potential methods for recycling asphalt back into asphalt depending on site circumstances. Examples include in situ hot or cold recycling and ex situ hot or cold recycling.

There is also a wide range of secondary materials that can be used to replace primary aggregates in asphalt. Examples include crushed glass, blastfurnace slag, steel slag, incinerator bottom ash, rubber, china clay sand and slate waste.

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Chapter 23

Bitumen properties and test methods

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Bitumen is a complex, viscoelastic material extracted from the hard-end of the hydrocarbon spectrum during the distillation of crude oil and used as the binder in flexible, asphalt road pavements. Its complexity results in a wide range of physical properties that can be measured through a combination of empirically based and fundamental rheological test methods.

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Introduction

Bitumen is a civil engineering construction material manufactured from crude oil through a series of distillation processes undertaken during the refining of petroleum. The principal use of bitumen is as a binder in the road construction industry where it is mixed with graded aggregate to produce asphalt. This asphalt material is then laid as the structural pavement layers of a road. The main function of these 'bitumen-bound' layers is to spread loads (caused by the trafficking of vehicles) evenly over the unbound pavement layers of the road and natural subgrade to prevent overstressing. In addition to providing stiffness and bearing capacity, asphalt must be able to resist two primary modes of flexible pavement distress, namely excessive permanent deformation (rutting) and fatigue cracking. As the mechanical properties of asphalt are strongly dependent upon the properties of the binder, bitumen has to fulfil certain mechanical and rheological requirements in order to ensure the integrity of the road.

Brief history

The term 'bitumen' originated in Sanskrit, where the words *jatu* meaning pitch and *jatu-krit* meaning pitch creating, referred to the pitch produced by certain resinous trees. The later Latin equivalent is claimed to be originally *gwitu-men* or *pixtu-men*, which was subsequently shortened to 'bitumen' when passing by way of French to English (Read and Whiteoak, 2003). The origins of bitumen as an engineering material date from 3800 to 3000 BC when surface seepages of 'natural' bitumen were used as mortar for masonry and waterproofing in the Euphrates and Indus Valleys (Abraham, 1945). There are also Biblical accounts of the use of bitumen. Noah used it on the ark and Moses was hidden in the bulrushes as a baby in a basket waterproofed with bitumen. In the middle of the nineteenth century attempts were made to utilise rock asphalt from European deposits (primarily the mines of the Neuchatel and Val de Travers area) for road surfacing and from this there was a slow development of the use of natural asphalt products, followed by the advent of coal tar and later of bitumen manufactured from crude oil.

Although 'natural' bitumens, such as Trinidad Lake Asphalt (TLA), are still used, most present-day applications make use of bitumen manufactured from crude oil. Crude oil consists of a complex mixture of hydrocarbons of different molecular weight produced by nature primarily from plant life and the remains of marine micro-organisms over millions of years under varied conditions of temperature and pressure. The manufacture of bitumen from crude oil involves a number of distillation processes and, if required, further treatments including air blowing and blending (**Figure 1**). Initially, crude oil is refined by atmospheric fractional distillation to separate gas, gasoline, kerosine, gas oil and long residue (heaviest fraction consisting of a complex mixture of high molecular weight hydrocarbons). The long residue is then further distilled under vacuum at 350–400°C to produce short residue, which is the feedstock used in the manufacture of different grades of bitumen. The physical properties of the short residue may be further modified by processes such as air ratification (blowing) or blending of different short residues to produce a range of bitumens with different consistencies (degrees of hardness). Bitumen manufactured directly from the short residue is known as 'straight run' bitumen.

The four principal crude oil producing areas in the world are the USA, the Middle East, the countries around the Caribbean, and Russia. Physically, crude oils range from viscous black liquids to free-flowing straw-coloured liquids. Chemically, they may be predominantly paraffinic, naphthenic or aromatic, with combinations of the first two being most common. As crude oils vary widely in the amount and nature of the hydrocarbons they contain, bitumen, as the residue left after the more volatile fractions

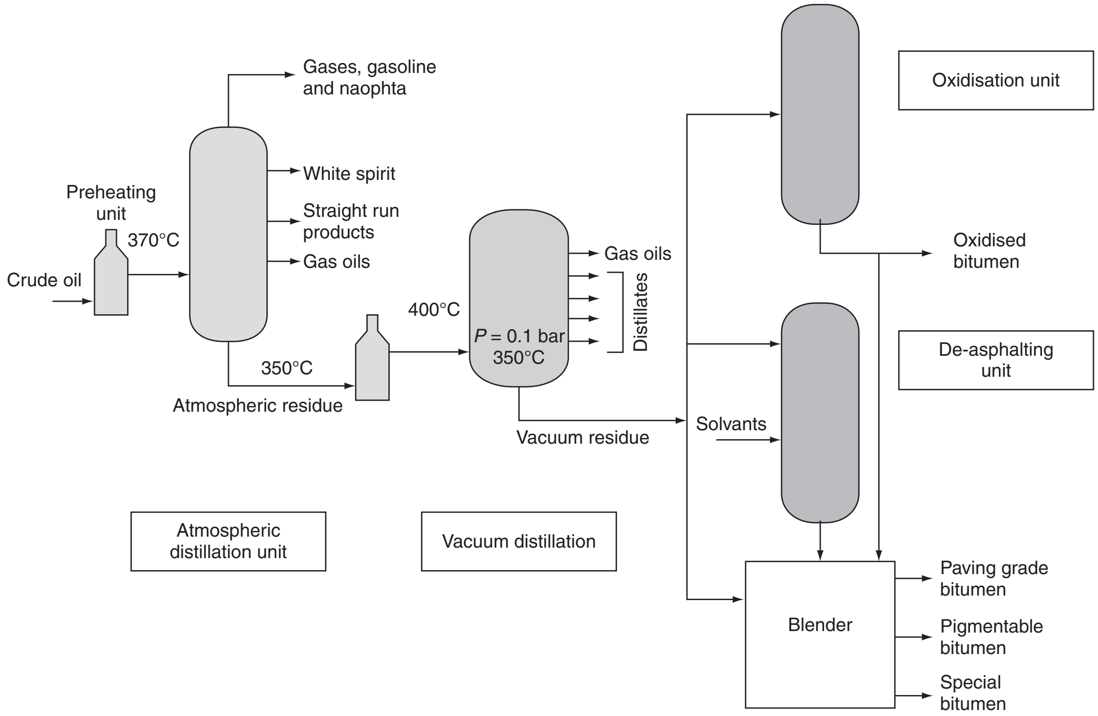

Figure 1 Schematic representation of crude oil distillation process

of the crude oil have been distilled off, varies significantly from crude to crude. The physical state of crude oil can be described by its American Petroleum Institute (API) gravity, which is directly related to the density of the crude oil. The API gravity varies from 0.0 to more than 70° API. Crude oils with a low API gravity are viscous with high density and generally contain a high percentage of bitumen. Of the 1300 types of crude oil classified worldwide, only 10% are suitable for producing bitumen capable of meeting the specifications. These crudes are known as 'bitumen' crudes and are defined as 'heavy' with an API gravity of <20° API. These crudes also tend to have a high sulphur content (>1% by weight) and are classified as 'sour' compared with the low sulphur content 'sweet' crudes used to produce distillate products (LPG (liquid petroleum gas), naphtha, kerosene and gas oil). The manufacture of bitumen is, in reality, a very sophisticated activity requiring extensive expertise and considerable financial investment to provide consistent products meeting the relevant specification requirements, such as those described in the European Standard BS EN 12591: 2000.

Bitumen chemical composition

In terms of its chemistry, bitumen is a complex mixture of organic molecules which vary widely in composition. These molecules are predominantly hydrocarbons, but also contain small amounts of structurally analogous heterocyclic species and functional groups (heteroatoms) containing sulphur, nitrogen and oxygen atoms as well as trace metals such as vanadium and nickel (Traxler, 1936; Romberg *et al.*, 1959). The organic molecules vary from non-polar, saturated hydrocarbons to highly polar, highly condensed ring systems with the three principal types of molecules being the non-polar aliphatics and naphthenics and the polar aromatics. Knowledge of the molecular structure is crucial for a fundamental understanding of how bitumen's composition affects its physical properties and chemical reactivity (Petersen, 1984).

The precise composition of bitumen depends on the crude oil source and the processes involved in its manufacture. As the chemical composition is so complex and the number of different chemical structures so astronomically large, chemists usually do not attempt to separate and identify all the molecules in bitumen (Petersen, 1984; Halstead, 1985). Instead, techniques have been developed to separate bitumen into less complex and more homogeneous fractions based on molecular size, chemical reactivity and/or polarity. Examples of chromatographic techniques used to quantify these fractions include high-performance liquid chromatography (HPLC), ion exchange chromatography (IEC) and gel permeation chromatography (GPC).

In general, bitumen can be split into two broad chemical groups, namely asphaltenes, which are insoluble in n-heptane, and maltenes. The maltenes can be further separated into saturates, aromatics and resins (Corbett, 1970).

The asphaltene content has a large effect on the rheological properties of bitumen, being related to a number of physical parameters such as the glass transition and bitumen viscosity (Corbett, 1970; Halstead, 1985). Increasing the asphaltene content produces harder bitumen with higher viscosity. Asphaltenes typically constitute 5–25% of the bitumen and are the heaviest constituents with molecular weights ranging from 600 to 300 000 (Read and Whiteoak, 2003). Resins act as dispersing agents or peptisers for the asphaltenes and their proportion to asphaltenes determines the structural character of the bitumen (Nellensteyn, 1924). They have molecular weights ranging from 500 to 50 000. Aromatics are the major dispersion medium for the peptised asphaltenes and constitute 40–65% of the bitumen and have molecular weights ranging from 300–2000. Saturates are non-polar viscous oils, consisting of 5–20% bitumen, with molecular weights ranging from 300–2000 (Read and Whiteoak, 2003).

In terms of its structure, bitumen is traditionally regarded as a colloidal system consisting of high molecular weight asphaltene micelles dispersed or dissolved in the lower molecular weight oily maltenes medium. There are therefore two distinct types of bitumen, these being the well-dispersed solution (SOL) type and the gelatinous (GEL) type (Pfeiffer and Saal, 1940) (**Figure 2**). SOL-type bitumens are those which have sufficient quantities of resins and aromatics of adequate solvating power leading to fully peptised, well-dispersed asphaltenes that do not form extensive associations. GEL-type bitumens are those in which the resin and aromatic fraction is insufficient to fully peptise the micelles and therefore the asphaltenes form large agglomerations or even continuous networks. The Index of Colloidal Instability (CI), which is defined as the ratio of the amount of asphaltenes and saturates to the amount of resins and aromatics, is sometimes used to describe the stability of the colloidal structure. The higher the CI, the more the bitumen is regarded as GEL type bitumen. The lower the CI, the more stable the colloidal structure.

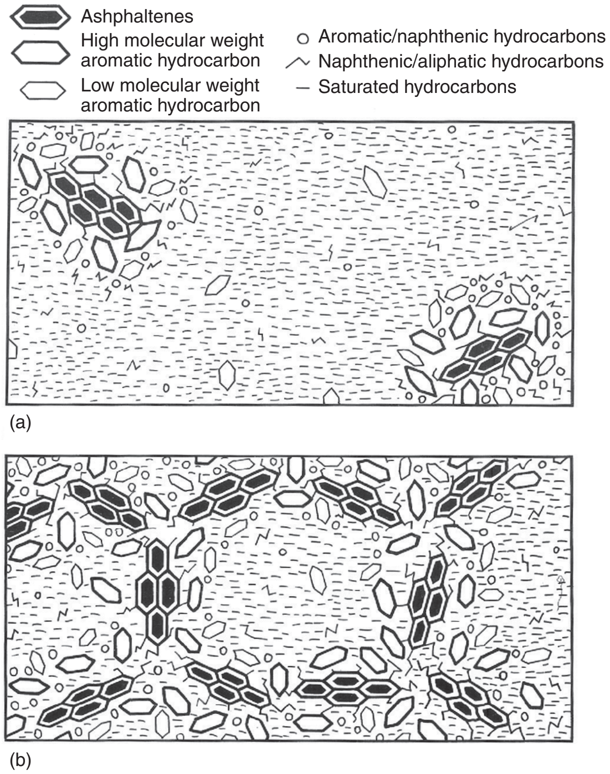

Figure 2 Schematic representation of (a) SOL and (b) GEL bitumen structure (Read and Whiteoak, 2003)

Other structural models have been proposed including a conceptual microstructural model proposed by the Strategic Highways Research Program (SHRP) researchers (Petersen *et al.*, 1994) and a thermodynamic solubility model (Redelius, 2006). Irrespective of their differences, all these models have one common goal of attempting to establish a correlation between the chemical composition of the bitumen and its physical properties. The physical, mechanical and rheological characteristics of bitumen are therefore determined and defined by both the constitution (chemical composition) and the structure (physical arrangement) of the molecules in the material (Nellensteyn, 1924; Corbett, 1970; Petersen, 1984).

Physical properties

Physically, bitumen can be classified as a thermoplastic material that shows glass-like behaviour (elastic and brittle response to loading) at low temperatures (<0°C) and fluid-like behaviour at high temperatures (>60°C). At intermediate temperatures (0–60°C), bitumen possesses both elastic and viscous properties (viscoelastic response) with the relative proportion of these two responses depending on temperature and loading rate. It is this fundamental rheological (flow) property of bitumen that defines its physical nature and makes it such a versatile binder for paving mixtures in virtually all of the habitable climates found on earth.

In the measurement of the physical properties of bitumen, primary emphasis is given to the characterisation of the rheological behaviour of bitumen. Rheology is a fundamental interdisciplinary science which is concerned with the study of the internal response of real materials to stresses. The word ‘rheology’ is derived from the Greek words $\rho\varepsilon\omega$, which translates literally as ‘to flow’ and $\lambda o\gamma o\sigma$ meaning ‘word, science’ and therefore literally means ‘the study of the flow’. Bitumen rheology can therefore broadly be defined as the fundamental measurements associated with the flow and deformation characteristics of bitumen.

The physical behaviour of bitumen is complex and to describe its properties over a wide range of operating

	Unit	Test method	Grade designation								
			20/30	30/45	35/50	40/60	50/70	70/100	100/150	160/220	250/330
Penetration @ 25°C	0.1 mm	EN 1426	20–30	30–45	35–50	40–60	50–70	70–100	100–150	160–220	250–330
Softening point	°C	EN 1427	55–63	52–60	50–58	48–56	46–54	43–51	39–47	35–43	30–38
Resistance to hardening @ 163°C		EN 12607-1 or									
– change in mass, maximum, ±	%	EN 12607-3	0.5	0.5	0.5	0.5	0.5	0.8	0.8	1.0	1.0
– retained penetration, minimum,	%		55	53	53	50	50	46	43	37	35
– softening point after hardening, minimum	°C	EN 1427	57	54	52	49	48	45	41	37	32
Flash point, minimum	°C	EN 22592 (b)	240	240	240	230	230	230	230	220	220
Solubility, minimum	% (m/m)	EN 12592	99.0	99.0	99.0	99.0	99.0	99.0	99.0	99.0	99.0

Table 1 Specifications for paving grade bitumens (adapted from EN 12591: 2000)

conditions (temperature, loading rate, stress and strain) would require a large number of tests. To avoid this and simplify the situation, the mechanical behaviour and rheological properties of bitumen have traditionally been described using empirical tests and equations. The two consistency tests required in the European Standard BS EN 12591: 2000 (**Table 1**) to characterise different bitumen grades are the needle penetration test (BS EN 1426: 2000) and the ring and ball softening point test (BS EN 1427: 2000). These tests provide an indication of the consistency (hardness) of the bitumen without completely characterising the viscoelastic response and form the basis of the bitumen specification.

Penetration test

The penetration test, shown in **Figure 3**, is used to measure the consistency of bitumen. This is done by expressing the distance, in tenths of a millimetre (decimillimetre), that a standard needle (specified dimensions) will penetrate vertically into a sample of bitumen under a load of 100 g at a fixed temperature of 25°C for a loading duration of 5 seconds. The greater the penetration of the needle, the softer the bitumen and, conversely, the lower the value of penetration, the harder the bitumen. This test is the basis upon which penetration grade bitumen is classified into standard penetration ranges as demonstrated in the specification shown in **Table 1**. Other combinations of load, temperature and loading time may also be used, although meaningful comparison of the penetration of bitumen over a range of temperatures is only possible if the same load and loading times are used consistently. The penetration test can be considered as an indirect measurement of the viscosity of the bitumen at a temperature of 25°C.

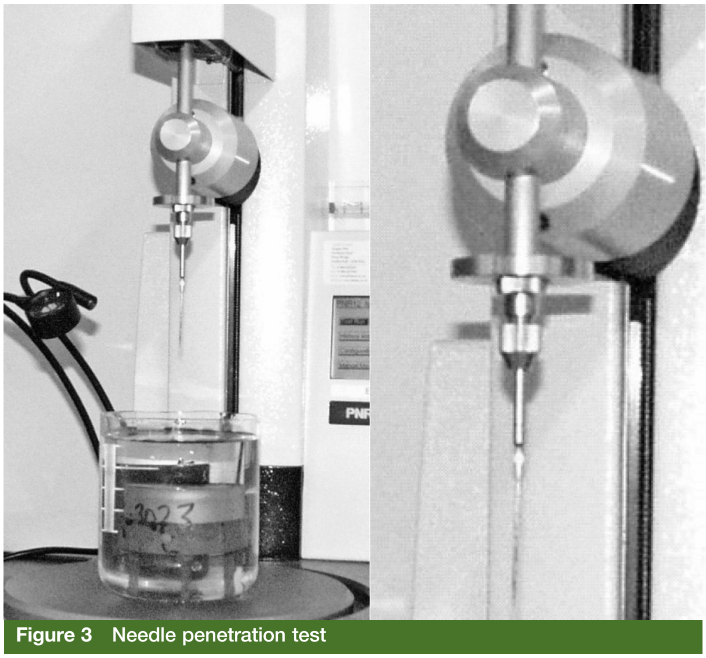

Figure 3 Needle penetration test

Softening point test

The ring and ball softening point test is an empirical test used to determine the consistency of bitumen by measuring the equiviscous temperature at which the consistency of the bitumen is between solid and liquid behaviour. Therefore, regardless of the grade of the bitumen, the consistency will be the same for different bitumens at their respective softening point temperatures. The American Society for Testing and Materials (ASTM) also specifies a softening point test but unlike the BS EN 1427: 2000 method no stirrer is used in the liquid bath and, consequently, the ASTM softening point is 1.5°C higher than the EN method.

The softening point test consists of placing a steel ball on a disc of bitumen contained within a brass ring and suspended in a water or glycerol bath, as shown in **Figure 4**. The temperature of the fluid bath is then raised at a constant rate of 5°C/min. The softening point is the temperature at which the bitumen softens enough to allow the ball enveloped in bitumen to fall a distance of 25 mm before hitting a base plate. The test is commonly referred to as the ring and ball test due to the components used in the test. Van der Poel (1954) showed that softening

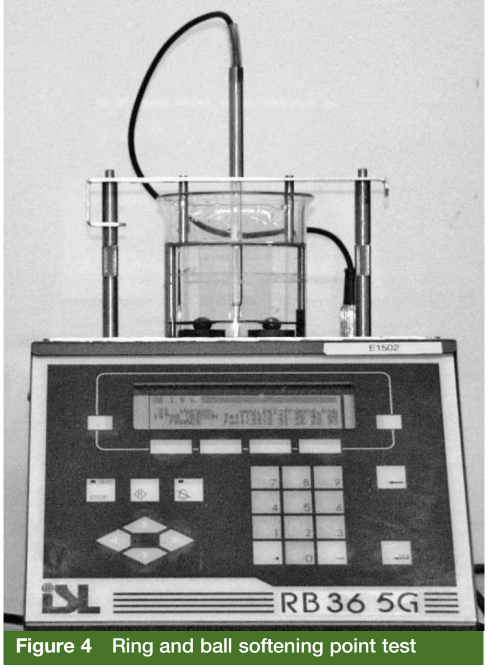

Figure 4 Ring and ball softening point test

point is approximately the temperature at which penetration is 800. Hence the notation '$T_{800\mathrm{PEN}}$' is commonly used to describe softening point temperature.

Viscosity

Viscosity is the measure of the resistance to flow of a liquid and is defined as the ratio between the applied shear stress and the rate of shear strain measured in units of pascal seconds (Pa.s). It is a fundamental characteristic of bitumen and determines how the material will behave at a given temperature and over a temperature range. In addition to absolute or dynamic viscosity, viscosity can also be measured as kinematic viscosity in units of m^2/s or more commonly mm^2/s with 1 mm^2/s being equivalent to 1 centistoke (cSt).

The viscosity of bitumen can be measured with a variety of devices in terms of its absolute and kinematic viscosities. Specifications are generally based on a measure of absolute viscosity at 60°C and a minimum kinematic viscosity at 135°C using vacuum and atmospheric capillary tube viscometers respectively. Absolute viscosity can also be measured using a fundamental method known as the sliding plate viscometer. The sliding plate test monitors force and displacement on a thin layer of bitumen contained between parallel metal plates at varying combinations of temperature and loading time.

The rotational viscometer test (ASTM D 4402-02) is presently considered to be the most practical means of determining the viscosity of bitumen. The Brookfield rotational viscometer and Thermocel system (**Figure 5**) allows the testing of bitumen over a wide range of temperatures (more so than most other viscosity measurement systems).

Figure 5 Rotational viscometer

The operation of the rotational viscometer consists of one cylinder rotating coaxially inside a second (static) cylinder containing the bitumen sample all contained in a thermostatically controlled environment. The material between the inner cylinder and the outer cylinder (chamber) is therefore analogous to the thin bitumen film found in the sliding plate viscometer. The torque on the rotating cylinder or spindle is used to measure the relative resistance to rotation of the bitumen at a particular temperature and shear rate. The torque value is then altered by means of calibration factors to yield the viscosity of the bitumen.

Fraass breaking point

The Fraass test is an empirical means of obtaining an estimate of the temperature at which a thin film of bitumen might crack. **Figure 6** illustrates the equipment and sample arrangement. The test involves the flexing of a sample of bitumen contained on a spring steel plaque at successively lower temperatures until it cracks. The temperature at which the sample cracks is termed the breaking point and represents an equi-viscous temperature. The Fraass breaking point temperature is used in bitumen specifications in many European countries and is standardised in BS EN 12593: 2000.

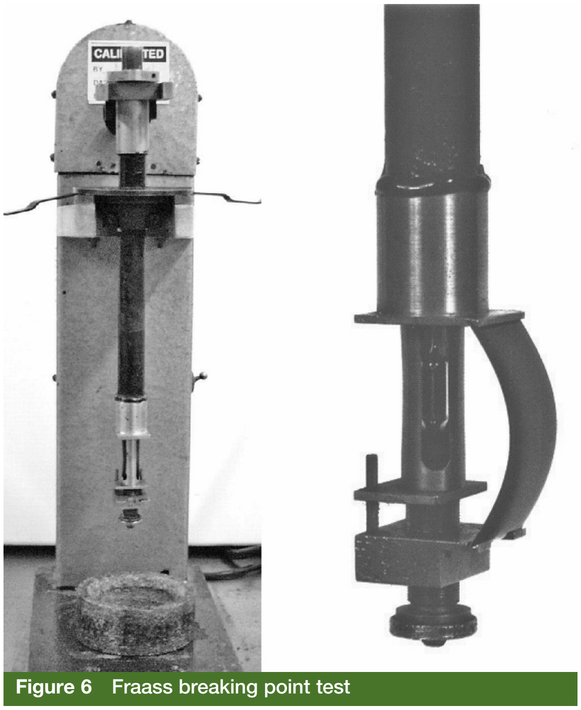

Figure 6 Fraass breaking point test

The Bitumen Test Data Chart

The Bitumen Test Data Chart (BTDC) was developed by Heukelom (1973) utilising an approximate relationship between penetration and absolute viscosity, as shown in **Figure 7**. It may be seen that there are two equiviscous lines on the chart, one occurring at a penetration of 800, representing the softening point and another at a penetration of 1.25, representing the Fraass breaking point. The BTDC provides a convenient means of plotting routine bitumen data, enabling interpolation and tentative extrapolation of data, since the majority of bitumens produce data that plot as a straight line on the chart. It is important to note that the softening point line on the BTDC is for the ASTM test conditions (no stirrer) and values from the BS EN test should be increased by 1.5°C when plotted.

Temperature susceptibility

The consistency of all bitumen varies with temperature. The rate at which this occurs depends upon the type and grade of bitumen. Several expressions exist to define this temperature susceptibility. One of the best known is the Penetration Index (PI) developed by Pfeiffer and Van Doormaal (1936).

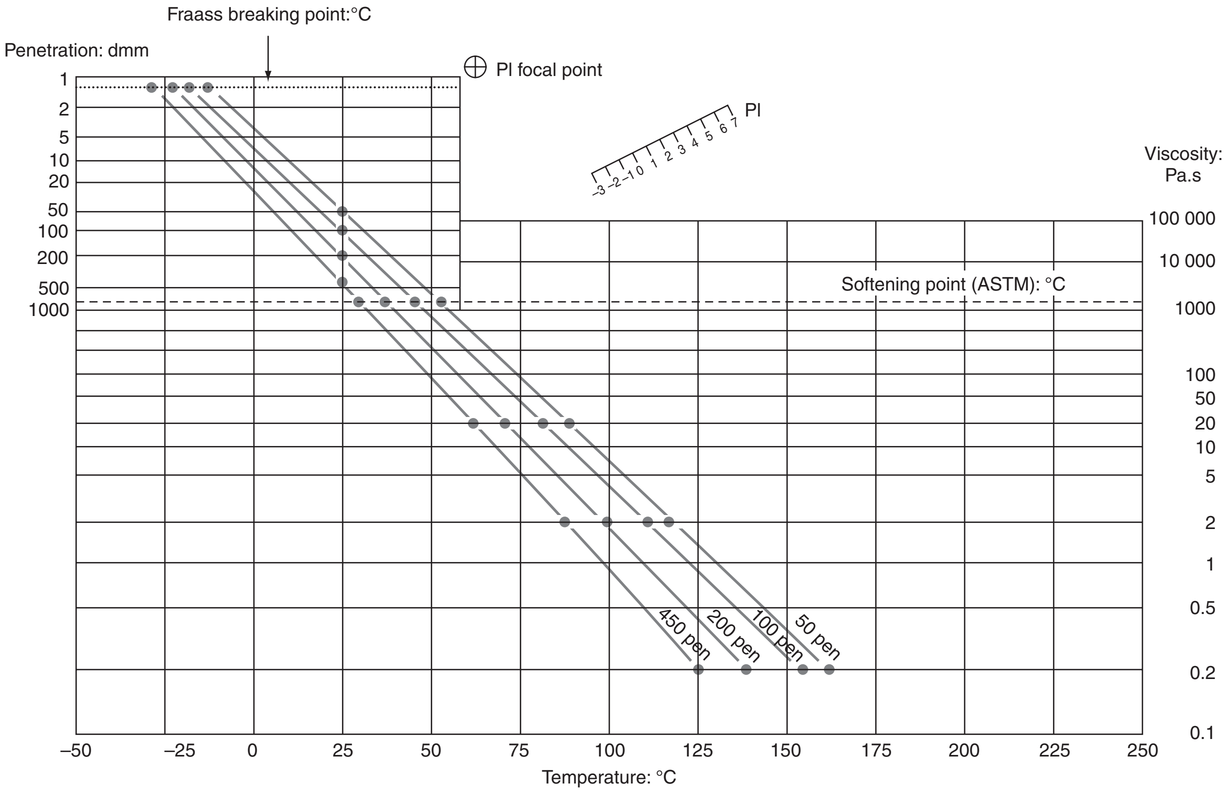

Figure 7 Bitumen test data chart comparing penetration grade bitumen manufactured from one crude source (Read and Whiteoak, 2003; Heukelom, 1973)

Using the linear relationship between the logarithm of penetration and temperature in the BTDC ($\log P = AT + K$), allows the slope of the line (A) to be related to the temperature susceptibility of the bitumen. The slope, A, can then be calculated from the penetration and the softening point temperature (equivalent penetration of 800 dmm in the BTDC) of the bitumen:

$$A = \frac{\log(800) - \log(P@25°\mathrm{C})}{\text{ASTM softening point } (°\mathrm{C}) - 25°\mathrm{C}} \qquad (1)$$

These values of A tend to be very small and inconvenient so a further equation was developed to indicate temperature susceptibility by an index figure, i.e. Penetration Index, where:

$$PI = \frac{20(1 - 25A)}{1 + 50A} \qquad (2)$$

Using this system, the PI of bitumen varies between approximately −2.5 and +7.0 with bitumens of lower and negative PI softening more readily than those with higher PI.

Advanced rheological testing

Empirical tests, such as penetration and softening point, and even the more fundamental tests such as viscosity, do not provide a complete rheological characterisation of the bitumen as they do not quantify the time-dependent response of the binder (Anderson *et al.*, 1991). This has led to the use of dynamic mechanical methods using oscillatory-type testing to fully characterise the rheological properties of bitumen. These tests are generally conducted under linear viscoelastic (LVE) conditions where the rheological response of the bitumen can be considered to be independent of stress and strain level.

These advanced rheological tests are undertaken using dynamic shear rheometres (DSRs), which apply oscillating, sinusoidal shear stresses and strains to samples of bitumen sandwiched between parallel plates, as depicted in **Figure 8** (Goodrich, 1988). The DSR tests are performed at different loading frequencies and temperatures. The sinusoidal stress and strain readings are then used to calculate various stiffness, viscosity and viscoelastic parameters that are used to build a complete picture of the rheological properties of the bitumen as a function of temperature and time of loading (or loading frequency).

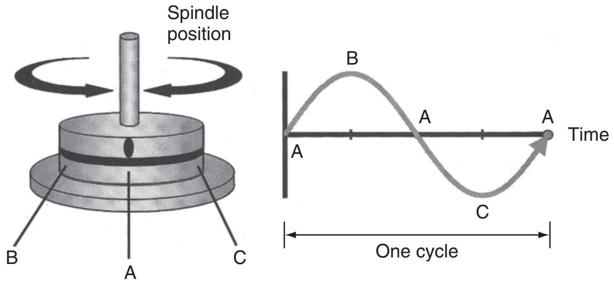

Figure 8 Dynamic shear rheometer operation

The principal viscoelastic parameters that are obtained from the DSR are the complex shear modulus, G^*, and the phase angle, δ. G^* is defined as the ratio of maximum stress to maximum strain and provides a measure of the total resistance to deformation of the bitumen when subjected to loading. It contains elastic and viscous components which are designated as the storage modulus, G', and loss modulus, G'', respectively. These two components are related to the complex modulus and to each other through the phase (or loss) angle which is the phase, or time, lag between the applied shear stress and shear strain responses during a test. The phase angle is a measure of the viscoelastic balance of the bitumen behaviour. If δ equals 90° then the bituminous material can be considered to be purely viscous in nature, whereas δ of 0 corresponds to purely elastic behaviour. Between these two extremes the material behaviour can be considered to be viscoelastic in nature with a combination of viscous and elastic responses.

DSR testing of bitumen is standardised in Clause 928 of the UK Highways Agency Specification for Highways Works and in the AASHTO T 315-06 Standard. In general, two testing geometries are used with the DSR, namely 8 mm diameter parallel plates with a 2 mm testing gap and 25 mm diameter plates with a 1 mm testing gap. The selection of the testing geometry is based on the operational conditions with the 8 mm geometry generally being used at low temperatures (−5–20°C) and the 25 mm geometry at intermediate to high temperatures (20–80°C).

The DSR generated rheological results can be presented in several forms. The most common forms consist of isochronal plots (viscoelastic parameters versus temperature at constant frequency), isothermal plots (viscoelastic parameters versus frequency at constant temperature), master curves (several isothermal plots shifted along the frequency axis to produce a smooth curve) and black diagrams (complex modulus against phase angle). The construction of master curves relies on the ability to shift rheological data through the equivalency between time and temperature (known as thermo-rheological simplicity) using a concept known as the time-temperature superposition principle (Ferry, 1980).

Bitumen ageing

In addition to its complex rheological properties, bitumen, like many other organic substances, is affected by the presence of oxygen, ultraviolet radiation and by changes in temperature. These external influences result in the phenomenon known as 'ageing' and cause changes in the chemical composition and therefore the rheological and mechanical properties of the bitumen (Petersen, 1984).

Ageing is primarily associated with the loss of volatile components and oxidation of the bitumen during asphalt mixture production (short-term ageing) and progressive oxidation of the in-place material in the road (long-term ageing). Both factors cause an increase in viscosity (or stiffness) of the bitumen and consequential stiffening of the asphalt mixture. Other factors may also contribute to ageing, such as molecular structuring over time (steric hardening) and actinic light (primarily ultraviolet radiation, particularly in desert conditions) (Traxler, 1963; Vallerga *et al*., 1957).

Ageing can have two effects: either increasing the load-bearing capacity (strength) and permanent deformation resistance of the pavement by producing a stiffer material; or reducing pavement flexibility, resulting in the formation of cracks with the possibility of total failure. Therefore, ageing at moderate levels is generally accepted and can even enhance performance. However, ageing at significant levels results in embrittlement of the bitumen, significantly affecting its adhesive characteristics and usually resulting in reduced cracking resistance of the asphalt mixture under repeated loading.

Tests related to ageing of bitumen can be broadly divided into two categories, namely tests performed on neat bitumens and tests performed on asphalt mixtures. Much of the research into the ageing of bitumen utilises thin film oven ageing to age the bitumen in an accelerated manner. Typically, these tests are used to simulate the relative hardening that occurs during the mixing and laying process (i.e. short-term ageing). To include long-term hardening in the field, thin film oven ageing is typically combined with pressure oxidative ageing.

The most commonly used short-term ageing test is the rolling thin film oven test (RTFOT), shown in **Figure 9** and standardised in BS EN 12607-1 and ASTM D2872-95. The RTFOT was developed by the California Division of Highways and involves rotating eight glass bottles each containing 35 g of bitumen in a vertically rotating shelf, while blowing hot air into each sample bottle at its lowest travel position (Hveem *et al*., 1963). During the test, the bitumen flows continuously around the inner surface of each container in relatively thin films of 1.25 mm at a temperature of 163°C for 75 minutes. The vertical circular carriage rotates at a rate of 15 revolutions/minute and the air flow is set at a rate of 4000 ml/minute. The method ensures that all the bitumen is exposed to heat and air and the continuous movement ensures that no skin develops to protect the bitumen. The conditions in the test are not identical to those found in practice but experience has shown that the amount of hardening in the RTFOT correlates reasonably well with that observed in a conventional batch mixer.

Long-term ageing of bitumen can be achieved using the pressure ageing vessel (PAV) developed to simulate the in-service oxidative ageing of bitumen in the field (Christensen and Anderson, 1992). The method involves hardening of bitumen in the RTFOT followed by oxidation of the residue in a pressurised ageing vessel. The PAV procedure (detailed in AASHTO R28-06) entails ageing 50 g of bitumen in a 140 mm diameter pan (approximately 3.2 mm binder film thickness) within the heated vessel, pressurised with air to 2.07 MPa for 20 hours at temperatures between 90 and 110°C.

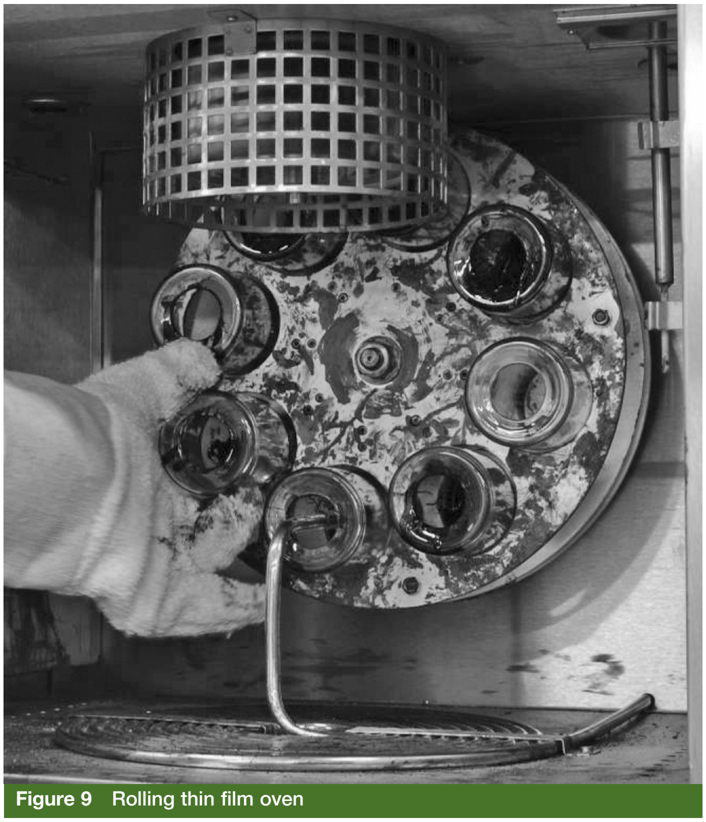

Figure 9 Rolling thin film oven

Bitumen modification

Another process, other than ageing, that alters the chemical composition and therefore the rheological properties of bitumen is polymer modification. While conventional bituminous materials tend to perform satisfactorily in most pavement applications, there are areas of the road network that require enhanced asphalt mixture performance only possible through polymer modification of bitumen. Typical polymers include styrene butadiene styrene (SBS), styrene butadiene rubber (SBR), ethylene vinyl acetate (EVA) and polyethylene among others (Isacsson and Lu, 1995). In terms of overall performance, polymer-modified bitumens (PMBs) tend to have greater elastic response, improved cohesive and fracture strength, greater ductility and are able to better resist the pavement distress mechanisms of permanent deformation and fatigue cracking.

Although polymer modification tends to be the most popular means of modifying bitumen, a wide range of

Type	Generic examples
1 Fillers	■ Mineral fillers: Crusher fines Lime Portland cement Fly ash ■ Carbon black
2 Extenders (chemical modifiers)	■ Organo-metallic compounds ■ Sulphur ■ Lignin ■ Polymers
3 Rubbers (thermoplastic elastomers) a Natural latex b Synthetic latex c Block copolymer d Reclaimed rubber	■ Natural rubber ■ Styrene butadiene rubber (SBR) ■ Polychloroprene latex ■ Styrene butadiene styrene (SBS) ■ Styrene isoprene styrene (SIS) ■ Crumb rubber modifier
4 Plastics (thermoplastic polymers)	■ Polyethylene (PE)/polypropylene (PP) ■ Ethylene acrylate copolymer ■ Ethylene vinyl acetate (EVA) ■ Polyvinyl chloride (PVC) ■ Ethylene propylene or EPDM ■ Polyolefins
5 Combinations	Blends of polymers in 3 and 4
6 Fibres	■ Natural: asbestos Rock wool ■ Man-made: polypropylene Polyester Glass fibre Mineral Cellulose
7 Oxidants	■ Manganese salts
8 Antioxidants	■ Lead compounds ■ Carbon ■ Calcium salts ■ Amines
9 Hydrocarbons	■ Recycling and rejuvenating oils ■ Hard and natural asphalts (gilsonite, TLA)
10 Antistripping agents (adhesion improvers)	■ Amides ■ Lime
11 Waste materials	■ Roofing shingles ■ Recycled tyres ■ Glass
12 Miscellaneous	■ Silicones ■ De-icing calcium chloride granules

Table 2 General classification of bitumen additives and modifiers

additives and modifiers can be used to enhance the properties of conventional bitumen as listed in **Table 2**.

In addition to altering the chemical composition of the PMB, polymers also affect the microstructure of the binder. Fluorescent microscopy is a useful technique to look at the structure (morphology) of PMBs, where, under an ultraviolet light source, a two-phase image can be produced with the polymer-rich phase appearing light and the bitumen-rich (asphaltene-rich) phase appearing dark, as shown in **Figure 10**.

Fluorescent microscopy is based on the principle that polymers swell due to absorption of some of the constituents of the base bitumen (primarily some of the light oil fractions found in the maltenes) and, subsequently, fluoresce in ultraviolet light. This fluorescence from the polymer-rich phase is due to the fluorescence of the aromatic oils that have been absorbed by the polymer during the swelling process. Using this technique, it is possible to determine the quantity of modifier in the PMB and estimate the rheological behaviour of the binder.

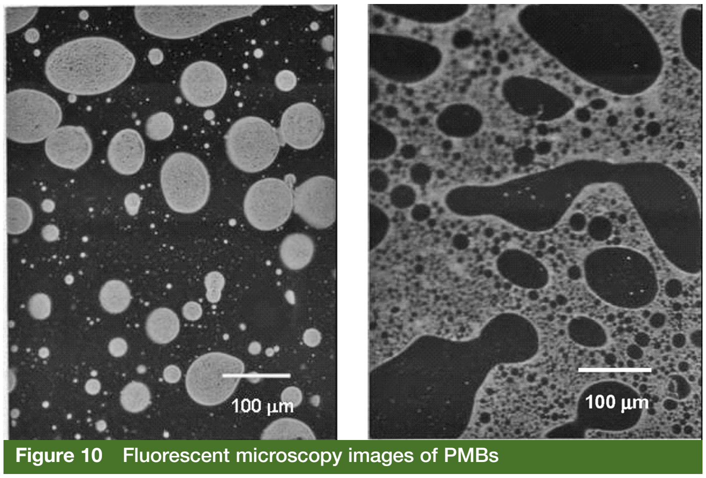

Figure 10 Fluorescent microscopy images of PMBs

Summary

Bitumen is a thermoplastic, viscoelastic hydrocarbon derived from petroleum either naturally or through distillation. Bitumen acts as a binder where it binds or ‘cements’ graded aggregate particles together to form an engineering material known as ‘asphalt’. Due to its complex mixture of chemical components, bitumen has a wide range of physical and rheological properties which are dependent not only on temperature but also on time of loading. These rheological properties can be defined through a combination of empirical tests, such as penetration and softening point, and more fundamental tests, such as viscosity and DSR tests. The rheological properties of bitumen are defined by both the source of the bitumen as well as its manufacturing process. In addition, the rheological properties can be further altered through oxidative ageing of the bitumen and modification using polymers or other additives.

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Chapter 24

Aggregate properties and test methods

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This chapter describes the properties and test methods required for aggregate when used in bituminous materials. It summarises the basic use of aggregate in the base, binder and surface course layers, then reviews the historical development of testing giving key sources of information. The development of British Standards from BS 812 to the current European Standards is discussed with examples of specification requirements. The chapter finishes by identifying future trends in the testing of aggregates for bituminous materials. Most notable will be the need to test aggregate as part of the bituminous material and not by unbound empirical methods.

CONTENTS

Introduction

A typical bituminous material consists of about 95% coarse and fine aggregate by weight. The mixture is designed and expected to contribute a wide range of engineering properties, many of which are attributable to the properties of the aggregate used. Despite this high proportion, the methods of testing aggregate are typically empirical. Most were developed many years ago and have poor direct relation to pavement performance, particularly as trafficking conditions and types of use were much different than those of today.

Aggregates are typically tested as unbound materials, so their real performance in asphalt mixtures is less clear. Some methods are difficult to carry out and suffer from poor reproducibility with specification compliance not ensuring acceptable performance. The importance now placed on sustainable highway construction has had a significant effect on how aggregate is used and expected to perform. This chapter considers the properties and test methods required for aggregates, and refers to the main test methods and specification requirements, and highlights areas that will affect its future use.

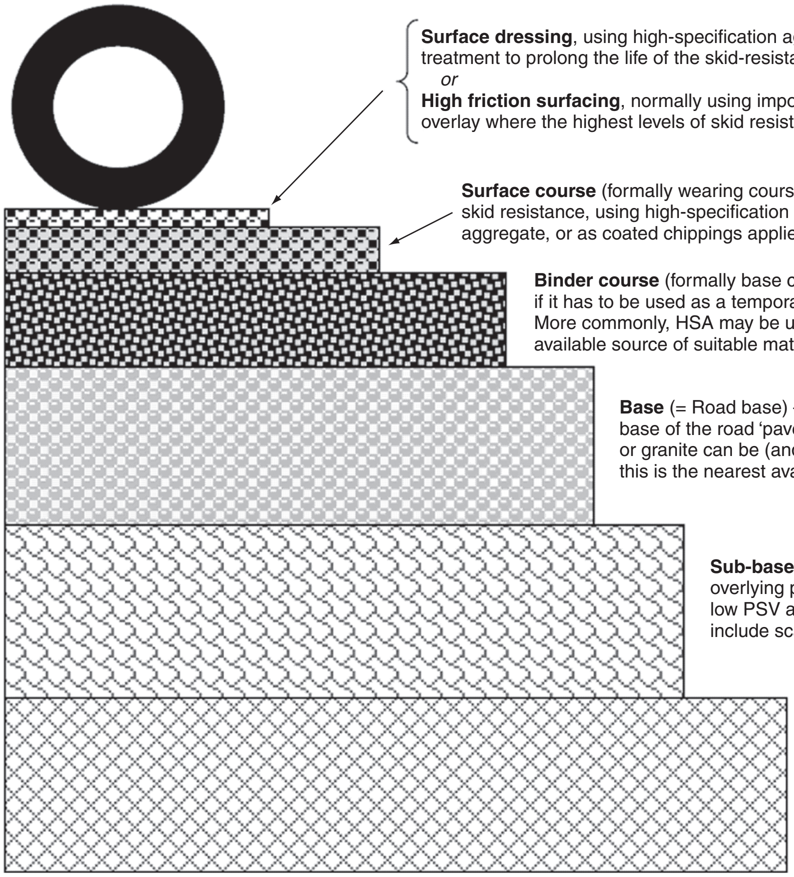
Main types of aggregate

There are three main types of aggregate that may be used to make asphalt: natural, artificial/synthetic and recycled. Their in-service performance depends on its geology. For example, the high skid resistance of gritstone relates to its sandpaper-like surface texture.

Natural aggregates may be classified as those sources where the rock is naturally occurring in the ground and obtained by conventional blasting and quarrying, excavating or dredging from land, river, estuarine or marine deposits. Aggregate from natural rock types belongs to one of four main types: igneous, sedimentary, metamorphic or sand and gravel. Igneous rocks have solidified from magma either within or above the Earth's surface and possess a very wide range of chemical composition, grain size, texture and mode of occurrence. Certain types may have formed due to slow cooling deep within the Earth's crust, i.e. coarse-grained granites. Other types have been extruded as lava flows on the Earth's surface resulting in fine-grained basalts. Sedimentary rocks are formed by weathering processes and subsequent transport by water, ice or wind. These processes cause deposition of these materials into stratified layers that are then consolidated or cemented by chemical action to form new rocks such as sandstone and gritstone. Sedimentary rocks may also be formed due to the chemical precipitation of minerals dissolved in water to form rocks such as limestone. Metamorphic rocks result due to alteration of existing igneous or sedimentary rocks by heat, pressure or chemical activity. For example, shale becomes slate with limestone becoming marble. The properties of sands and gravels are dependent on the rocks from which they were derived and their transport by water prior to deposition. Crushed gravel is typically not allowed in the manufacture of asphalt in the UK although in some other countries they are an important source of aggregate.

Artificial/synthetic aggregates have been made by man as a result of industrial processes such as the steel industry or specifically made for high-performance purposes. They include various types of slag that are a by-product of the steel industry. They also include the use of calcined bauxite in high friction surfacing. No natural aggregate can give comparable levels of wet skid resistance to calcined bauxite when used as a 3 mm aggregate.

Recycled aggregate now forms an important aggregate source in many countries due to increasing awareness of sustainable issues and the development of techniques which produce materials to meet specified requirements.



Surface dressing, using high-specification aggregate (may be applied as a remedial treatment to prolong the life of the skid-resistant surface).
or
High friction surfacing, normally using imported calcined bauxite (may be applied as an overlay where the highest levels of skid resistance are required).

Surface course (formally wearing course) – normally the main layer providing skid resistance, using high-specification aggregate – either as the only coarse aggregate, or as coated chippings applied to the surface of hot-rolled asphalt.

Binder course (formally base course) – may require high-specification aggregate if it has to be used as a temporary running surface, but this is rarely the case. More commonly, HSA may be used, despite not being required, if it is the nearest available source of suitable material.

Base (= Road base) – the principal bound structural layer, forming the base of the road 'pavement'. Low PSV aggregate, such as hard limestone or granite can be (and usually is) used, although HSA may be used where this is the nearest available source of suitable material.

Sub-base – forms the constructional platform for the overlying pavement. Usually made of graded, unbound, low PSV aggregate (or secondary aggregate, but may include scalpings etc. from HSA quarries).

Capping layer (if needed, to increase the bearing capacity of weak ground).

Figure 1 The main structural elements of a modern flexible pavement and the types of aggregate required (reprinted from Thompson *et al*., ©2004 Capita Symonds Ltd)

Some specifications now allow up to 50% of recycled asphalt to be used with virgin materials. Other materials such as crushed concrete and demolition waste are widely used in the lower unbound pavement layers.

Use of aggregate in bituminous materials

Aggregate is used in all layers of road construction. **Figure 1** shows a typical section through a pavement and the terms used to describe the different layers and types of aggregate required. The bitumen bound layers start with the base. This forms the main structural layer of the pavement. Any aggregate that meets the basic requirement of clean, hard and durable can be used to construct this layer provided it bonds well with the bitumen used and consideration is given to potential moisture-related problems.

The binder course can be constructed using the same aggregate with one exception: if it is to be used as a temporary running surface the aggregate may be required to have skid-resistant properties measured using the polished stone value (PSV) test. In the UK this is referred to as high specification aggregate (HSA). Although not common in the UK, use of the binder course as a temporary running surface is still practised in Southern Ireland.

The surface course is expected to provide a range of characteristics such as wet skid resistance, texture depth, resistance to ravelling, fatigue and permanent deformation to low noise and rolling resistance. These are achieved by careful design and choice of aggregate and bitumen type. Wet skid resistance is the main property and is provided by aggregate with suitable skid and abrasion resistance properties graded so that the effects of water are minimised within the contact patch by removal of surface water either across or within its structure.

Types, sources and properties of aggregate

The aggregate used in bituminous materials is expected to have a range of properties such as size, shape, strength and durability. However, the properties of skid and abrasion resistance, measured using the PSV and aggregate abrasion value (AAV) test methods, influence the types of aggregate used and specification properties. In simple terms, most types of clean hard and durable aggregate can be used in the lower asphalt layers with the requirement for skid resistance narrowing the types for surface course layers.

Some of the earliest research relating rocks to aggregate properties was by Lovegrove who by 1929 had data for over 400 sources of aggregate from the British Isles (Lovegrove *et al*., 1929). Researchers such as Knight (1935), Hartley (1974), Lees and Kennedy (1975), Bullas and West (1991), Thompson *et al*. (1993, 2004) and Collis *et al*. (2001) considered the properties of aggregate. Nicholls (1998) and Hunter (2000) detail the required aggregate properties and test methods required for use in modern bituminous materials. Knill (1960) identified the importance of petrology, particularly in relation to skid resistance. She showed that gritstone aggregates possessed the highest values and proposed this was due to plucking, i.e. the progressive removal of the grit-sized grains from a softer matrix so providing a continual rough surface. This type of rock subsequently became the main type for use in the surface course.

The introduction of skidding resistance standards renewed the need to identify suitable sources of aggregate to meet the limits given. Hosking (1992) summarised much of the subsequent work on natural aggregates into understanding the factors that influence aggregate PSV. For the highest levels of skidding resistance, research into synthetic aggregates led to the use of calcined bauxites and development of high friction surfacings.

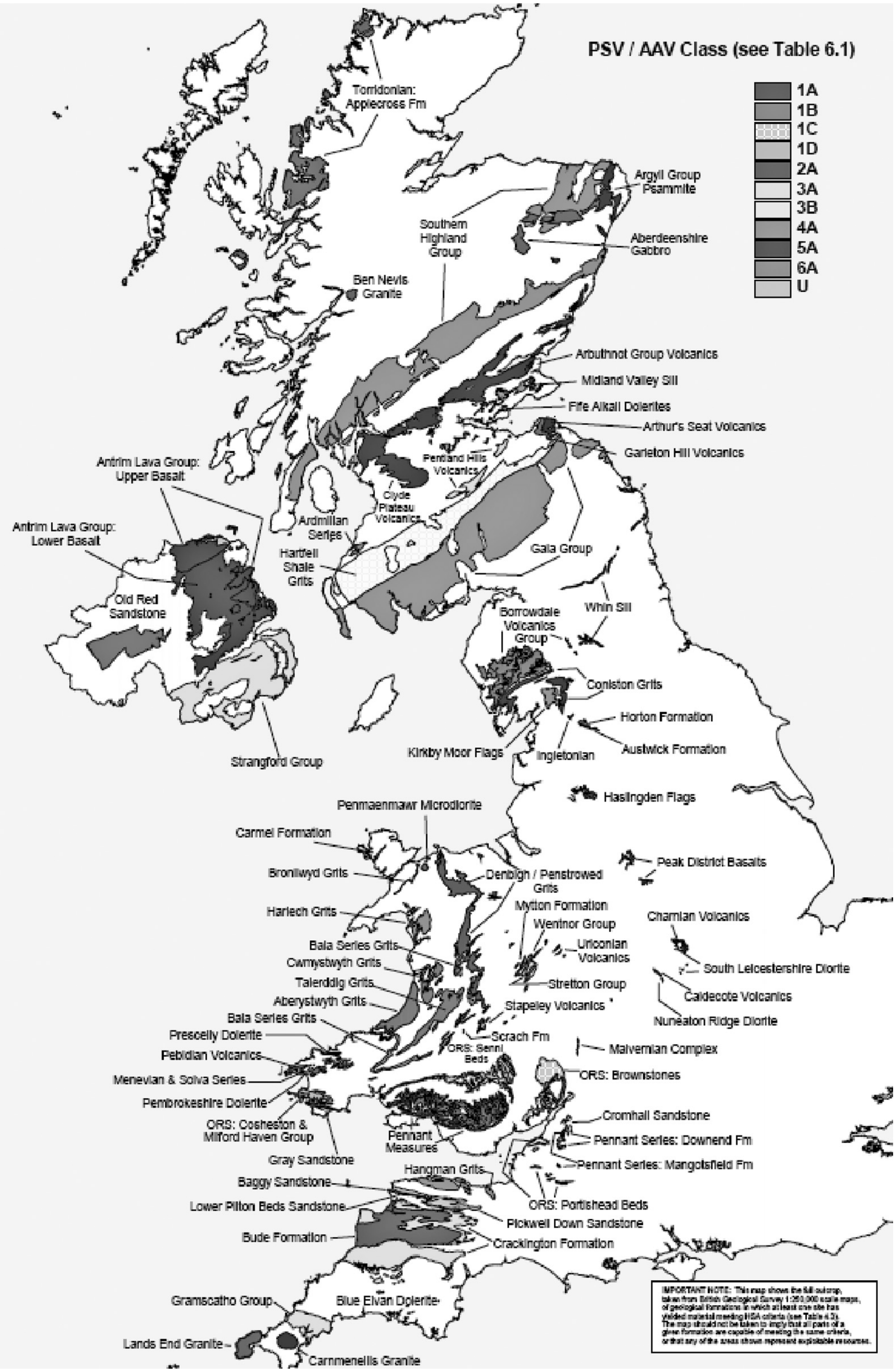
By the late 1980s, the geographical distribution and technical properties of rock types that could meet, or potentially meet, the required specification requirements became an important matter of concern. There were major problems in demonstrating the need for additional consented reserves, the consenting of new reserves and possible sterilisation of resources.

In the early 1990s the Department of Transport renewed its interest in sources of higher PSV aggregates. This led to the Travers Morgan survey in 1993 (Thompson *et al*., 1993). This collated, reviewed and enhanced the knowledge of existing and potential sources of high-specification aggregates (HSA) within the whole of the United Kingdom. It assessed the current and potential future supply of these materials in relation to existing and anticipated future levels of demand. Of the 76 sources of HSA identified, only seven were classed as being of the highest grade, with six located in Wales. This survey was subsequently revised in 2004 by Thompson *et al*. (2004) who considered the sustainable use of high-specification aggregates for skid-resistant road surfacing in England. Carried out by Capita Symonds Ltd, the report examined the use of high-specification aggregates in England for use in road surfacing applications and highlighted the sustainability issues involved.

This is the most comprehensive survey of aggregate for use in bituminous materials. Details are given of suitable rocks, location of quarries, aggregate test values and types of use. **Figure 2** shows the location of rocks suitable to provide high-specification aggregate classified in relation to PSV and AAV. Although there are localised exceptions, the majority of bituminous materials in the UK are made using aggregate from these sources. A regional variation in rock type exists. In north-west England the aggregate is quarried from Greywacke Sandstones of Silurian/Ordovician age and the Borrowdale Vocanics group. In the Midlands of England the important sources of aggregate come from igneous sources such as the Nuneaton Ridge Diorite, Caldecote Volcanics Formation and the Charian Volcanics sequence of pre-Cambrian age. Other sources in the Midlands include pre-Cambrian fluvial sandstones and indurated sandstones and mudstones of the Myton Flafs formation. In south-west England the main sources of aggregate come from carboniferous and Devonian sandstone and Blue Elvan Dolerites. The Welsh sources include some of the highest PSV sources in the UK such as the Stapely Volcanic Formation, Silurian and Ordovician Greywackes, Longmyndian Wentor Group Sandstones and Pennant Sandstones. The main sources of aggregate in Scotland are Quartz Dolerites from the Midland Valley sills and Silurian and Ordovician Greywackes. Finally, the main sources in Northern Ireland come from Silurian and Ordovician Greywackes and Tertiary Basalts.

The types of bituminous material used in the UK changed during the 1990s. Hot-rolled asphalt base and wearing course mixtures with 20 mm chippings had been used in heavily trafficked roads with macadam materials elsewhere. A combination of large-scale rutting of the trunk road network during summer months and problems with sourcing suitable sands and 20 mm chippings saw a change in aggregate use to higher stone content mixtures and thinner layers for the surface course. Variation in nominal stone size and void content allowed issues such as excessive noise and spray to be addressed and utilised all of the particles sizes generated during crushing.

The 1990s saw the rise in importance of recycled and secondary sources of aggregate and their use in bituminous materials either as coarse or fine aggregate replacement, as a bitumen modifier or as a modifier to the bituminous material. Of the 281 million tonnes of aggregate produced and landed, 67 million tonnes or 24% is currently from recycled and secondary sources. The continued use of recycled and secondary aggregates across a broad range of applications, including higher value uses such as bituminous materials, is facilitated by the European Standards for aggregates focusing on fitness for purpose rather than origin of the resource. The AggRegain website details case studies of recycled and secondary aggregates materials such as recycled asphalt, slate, steel slag, china clay, glass and plastic.



IMPORTANT NOTE: This map shows the full outcrop, taken from British Geological Survey 1:250,000 scale maps, of geological formations in which at least one site has yielded material meeting HSA criteria (see Table 4.3). The map should not be taken to imply that all parts of a given formation are capable of meeting the same criteria, or that any of the areas shown represent exploitable resources.

Figure 2 Location of high specification aggregates (reprinted from Thompson *et al*., ©2004 Capita Symonds Ltd)

Aggregate test methods

Development of British Standard test methods

The majority of aggregate test methods are empirical and were not developed with bituminous materials and the performance expectations of modern use in mind. This is highlighted in their historical development. BS 63 was published in 1913 as the first British Standard specification for road-making aggregates (BS 63: 1913). This standard simplified the classification of aggregate into 12 trade groups: granite, basalt (including basaltic whinstone), grit, gabbro, hornfels, limestone, porphyry, schist, flint, andesite, quartzite and artificial – terms that are still widely used today. Each group was deemed by the Geological Survey to give similar levels of behaviour.

BS 812 was first published in 1938 and contained many of the types of test still used today, e.g. sieve analysis, shape, surface texture, sampling, apparent specific gravity, volume weight and voids, water absorption and density, dry and wet attrition, abrasion, impact and crushing of 1 inch × 1 inch stone cylinders and amount of material <0.075 mm. In 1949 the aggregate crushing test appeared alongside the crushing strength test in preference to the American Los Angeles test. In 1951 there was complete revision with the intention that reference should be made to the standard for test methods, rather than the test methods as appendices to the individual materials specifications. The 1960 revision introduced angularity and the ten per cent fines test. The aggregate abrasion test replaced the Dorry abrasion test, the attrition test was omitted and the PSV test introduced. In 1967 major changes were made to the PSV test method. In 1975 the standard was divided into four parts. The methods were converted into SI units affecting sieve sizes, testing equipment dimensions and had a small but significant effect on test results. The crushing test was omitted due to poor precision and replaced by the aggregate crushing and ten per cent fines test methods for most contractual purposes. The last complete revision to BS 812 took place in 1990 when each test or related group of tests was produced as a separate part. The magnesium sulfate soundness value (MSSV) test was included due to recognition that certain aggregates were susceptible to types of premature failure which other test methods could not identify.

In summary, many of the common test methods found in specifications can trace their history back many years. In terms of testing aggregates there has not been any major shift in emphasis since BS 812 was first published in 1938. Test methods which originally used single pieces of stone were simply replaced by single-sized crushed rock aggregate. Until the inclusion of the MSSV test in 1991, the only other notable addition was the PSV test in 1960. Despite revisions the methods have all got particular issues associated with them. Examples include the testing of unbound single sizes when the aggregate is used in a graded bituminous mixture to the dry testing of strength and abrasion resistance when rain water is known to weaken aggregate.

Introduction of European test methods

The signing of the Construction Products Directive (CPD) by the UK in 1988 marked the start of a process to develop common European Standards across the construction sector. The aim of CPD was the removal of barriers to trade by an overall unification and simplification of existing national technical requirements. Numerous European research projects were carried out to unify and simplify the testing of aggregate for bituminous materials. For example, an extensive programme of cross-testing experiments was conducted over the period 1993 to 1997 involving tests for aggregates that were being developed as European Standard methods by CEN/TC 154. Up to 50 laboratories evaluated 20 different aggregate tests for properties such as particle size, shape, water absorption, and mechanical and weathering resistance. The findings are available at http://projects.bre.co.uk/aggregate/index.htm.

On 1 January 2004 a series of product standards and test methods was adopted, with existing British Standards withdrawn in June 2004. From 1 January all aggregate materials had to be supplied against the new standards (BS EN 13043: 2002) with all contract specifications such as the Specification for Highway Works revised. A National Guidance Document (PD 6682-2, 2003) was prepared to help material producers, specifiers and users. The main reference documents and test methods are summarised in **Table 1**. While the basic aggregate remained the same there were distinct changes in relation to terminology, product descriptions, standard sieve sizes, grading presentation and test methods. Unlike the previous British Standards, recycled aggregates were included within the scope of the product standards.

The introduction of European Standards brought important changes to the terminology used. For example, the coarse/fine aggregate split is now based on the 2 mm size for asphalt uses, fines being the fraction of an aggregate passing 0.063 mm with filler being material passing 0.063 mm that may be added to influence the properties of a mixture. The term category is the level of an aggregate property and may indicate the properties of a mixture. Product sizes are now based on the respective lower (d) and upper (D) sieve sizes expressed as d/D. For example, the previous 20 mm single size in accordance with the previous BS 63 became 14/20 or 10/20. The European product standards gave two options for the selection of sieves for grading and product descriptions. The UK elected to use the basic set plus set 2 with the recommended sequence for coarse and fine aggregates for asphalt use shown in **Table 2**. The traditional UK method of setting a

Reference	Document	Title	Year
BS EN 13043	Product standard	Aggregates for bituminous mixtures and surface treatments for roads, airfields and other trafficked areas	2002
PD 6682-2	National guidance document	Aggregates for bituminous mixtures and surface treatments for roads, airfields and other trafficked areas – guidance on the use of BS EN 123043	2003
PD 6682-9		Guidance on the use of European test method standards	2003
BS EN 932	Tests for general properties of aggregates	Part 1 Methods for sampling	1997
		Part 2 Methods for reducing laboratory samples	1999
		Part 3 Procedure and terminology for simplified petrographic description	1997
		Part 5 Common equipment and calibration	2000
		Part 6 Definitions of repeatability and reproducibility	1999
BS EN 933	Tests for geometrical properties of aggregates	Part 1 Determination of particle size distribution – sieving method	1997
		Part 2 Determination of particle size distribution – test sieves, nominal size of apertures	1996
		Part 3 Determination of particle shape – flakiness index	1997
		Part 4 Determination of particle shape – shape index	2000
		Part 5 Determination of percentage of crushed and broken surfaces in coarse aggregate particles	1998
		Part 6 Assessment of surface characteristics – flow coefficient of aggregates	2001
		Part 7 Determination of shell content – percentage of shells in coarse aggregates	1998
		Part 8 Assessment of fines – sand equivalent test	1999
		Part 9 Assessment of fines – methylene blue test	1999
		Part 10 Assessment of fines – grading of fillers (air-jet sieving)	2001
BS EN 1097	Tests for mechanical and physical properties of aggregates	Part 1 Determination of the resistance to wear (Micro-Deval)	1996
		Part 2 Methods for the determination of resistance to fragmentation	1998
		Part 3 Determination of loose bulk density and voids	1998
		Part 4 Determination of the voids of dry compacted filler	1999
		Part 5 Determination of the water content by drying in a ventilated oven	1999
		Part 6 Determination of particle density and water absorption	2000
		Part 7 Determination of the particle density of filler – pyknometer method	1999
		Part 8 Determination of polished stone value	2000
		Part 9 Determination of the resistance to wear by abrasion from studded tyres – Nordic test	1998
		Part 10 Determination of water suction height	2002
BS EN 1367	Tests for thermal and weathering properties of aggregates	Part 1 Determination of resistance to freezing and thawing	2007
		Part 2 Magnesium sulfate test	1998
		Part 3 Boiling test for Sonnenbrand Basalt	2001
		Part 4 Determination of drying shrinkage	1998
		Part 5 Determination of resistance to thermal shock	2002
BS EN 1744	Test methods – chemical properties	Part 1 Chemical analysis	1998
		Part 3 Preparation of eluates by leaching of aggregates	2002
		Part 4 Determination of susceptibility of fillers for bituminous mixtures	2005

Table 1 Summary of BS EN standards/documents for aggregates for bituminous materials

defined grading envelope was replaced by the use of a producer declared grading with given tolerances providing the control on key sieves together with the restrictions on the permitted amount of oversize and undersize material. An example of a grading classification is $G_c90/10$ where G is the grading category, $_c$ is coarse, 90 is the minimum percentage passing D (oversize control) with 10 being the maximum percentage passing d (undersize control).

Main European aggregate test methods

Each country was asked to nominate test methods and a selection of 36 was eventually chosen. In comparison to the methods found in BS 812, some were the same, some were similar whereas others were new to the UK. **Table 1** shows the test methods split into five main groups, each covered by a European Standard: BS EN 932 General Properties, BS EN 933 Geometrical Properties, BS EN 1097 Mechanical and Physical Properties, BS EN 1367 Thermal and Weathering Properties and BS EN 1744 Chemical Properties. The following summarises the main aggregate test methods used to assess aggregates for use in asphalt materials.

Resistance to fragmentation

Resistance to fragmentation is measured using the dry Los Angeles (LA) test (BS EN 1097-2: 1998). Originally developed in America in 1916, the method has been widely used around the world for many years. The method assesses duplicate 5000 g test portions of 10 to 14 mm size aggregate. These are placed in a large steel

Coarse aggregate: mm	Fine aggregate: mm
80	2
63	1
40	0.500
31.5	0.250
20	0.125
16	0.063
14	
10	
8	
6.3	
4	
2.8	

Table 2 Asphalt aggregate standard sieves as defined in PD 6682-2, 2003

cylinder along with an abrasive charge consisting of eleven 49 mm diameter steel balls. A shelf inside the steel drum causes the aggregate and steel ball bearings to tumble as it rotates 500 times at 33 rpm. The percentage of fine aggregate <1.6 mm in size caused by the resulting combination of abrasion, impact and crushing is used to calculate the Los Angeles value.

Resistance to polishing

Resistance to polishing is important for aggregate used only in the surface course layer. It relates to the surface roughness, or micro-texture of individual aggregate particles. Some aggregates are able to maintain their surface roughness whereas others lose it with their surface becoming smooth and polished. In the dry most aggregates can provide an adequate level of dry skid resistance. However, the ability to maintain adequate micro-texture is important for safety as it cuts through the water film that occurs underneath a moving tyre in wet road conditions.

Resistance to polishing is measured using the PSV test in accordance with BS EN 1097-8: 2000. Of the main European test methods, the PSV test is the only method that does not use 10 to 14 mm aggregate. Instead, it uses aggregate that passes a 10 mm sieve and is retained on a flake sorting sieve with a spacing of 7.2 mm. Here, 35 to 50 aggregate chippings are placed flat surface down in a curved mould and a resin used to hold the aggregate in place. Four test specimens are made for each aggregate being assessed.

The PSV test is in two parts. In the first part 14 test specimens are assessed using an accelerated polishing machine. These specimens consist of six pairs of specimens for different aggregates plus two control specimens. The test specimens are then subjected to simulated wet polishing conditions for 3 h using a coarse emery abrasive and a further 3 h using a fine emery abrasive. In the second part the degree of polishing is assessed using the Portable Skid Resistance Tester. The test is then repeated using the second pair of test specimens for each aggregate. The PSV of each aggregate is then calculated using the average of the four test specimens and corrected depending on the value of the control specimens.

Most trunk roads and motorways in the UK require aggregate with a PSV greater than 62. More critical sites require aggregates with PSV 68+. However, there are only a few quarries in the UK that produce such aggregate. The need to maintain adequate skid resistance over time highlights that it is very important to consider PSV in relation to other aggregate properties. For example, increases in PSV typically correspond to decreases in properties such as fragmentation, abrasion and soundness (Woodward, 1995).

Resistance to surface abrasion and wear

A road surface is designed to have a level of macro-texture, or texture depth that is necessary to facilitate the removal of water under a moving tyre. Therefore, surfacing aggregate must be resistant to abrasive wear caused by trafficking to maintain this property. Two methods are available in the European Standards to measure the long-term abrasion characteristics of road surfacing aggregate: the dry aggregate abrasion value (AAV) and the wet Micro-Deval (MDE) test methods.

The AAV test is carried out in accordance with BS EN EN 1097-8: 2000. The method is based on assessing dry resistance to mass loss resulting from abrasion by sand on a grinding lap. Single size 10 to 14 mm deflaked aggregate particles are used to make two flat resin-backed test specimens. These are weighed and subjected to 500 rotations on a horizontal steel grinding lap using sharp sand abrasive. The AAV is calculated from the resulting mass loss. For increasing numbers of commercial vehicles, the requirement for AAV increases.

The wet MDE test is carried out in accordance with BS EN 1097-1: 1996. The method is based on determining the mass loss of 500 g test portions of 10 to 14 mm size aggregate after they have been tumbled in a steel container for 2 h with 5000 g of 10 mm ball bearings and 2.5 litres of water. This very aggressive testing environment quickly highlights any weakness present with the aggregate that could affect its performance, e.g. aggregates prone to losing their edges or a source containing softer particles. When trafficked, such aggregates will become rounded and/or loose surface texture so affecting dynamic tyre interaction. Research has shown that small increases in PSV correspond to large decreases in wet MDE value, i.e. higher PSV aggregate is more prone to wet abrasion (Woodward, 1995).

Resistance to soundness

Soundness testing is used to highlight those aggregates that may be judged suitable in terms of their strength but found to fail in use, i.e. it is a measure of in-service durability. The magnesium sulfate soundness value (MS) test is used to measure this parameter in accordance with BS EN 1367-2: 1998. The standard MS test assesses test portions of 10 to 14 mm size aggregate. These are placed in a basket and subjected to five cycles of soaking in a saturated magnesium sulfate solution for about 17 h followed by oven drying for 8 h. During the soaking phase, salt solution penetrates any weakness within the aggregate. During the drying phase, the growth of salt crystals within these weaknesses results in internal stressing that induces disruption or breaking of the aggregate particles. Resistance to soundness is calculated as the percentage of remaining 10 to 14 mm aggregate particles. The presence of shale in gritstone or weathered basalt particles is highlighted using the MS test. Even though they may be present in relatively low amounts it is important that they are identified. In service, they tend to fail quite quickly and may be a contributing factor to premature failure of the asphalt layer. One of the problems with the MS test is that it takes approximately three weeks to carry out a test.

Resistance to freeze/thaw

Resistance to freeze–thaw is measured using the German freeze–thaw test in accordance with BS EN 1367-1: 2007. The method consists of subjecting test portions of 10 to 14 mm aggregate to ten freeze–thaw cycles of –20 to +20°C with the aggregate contained in water. However, the method is not widely used in the UK with the MS test used to predict the detrimental effects of freeze–thaw instead.

Specification of aggregate properties in the UK

Clauses 4 and 5 of BS EN 13043: 2002 specify aggregate properties with guidance given in PD 6682-2: 2003. Each clause starts with a sub-clause drawing attention to the necessity only to specify those properties relevant to the particular aggregate and end use of the aggregate. Tables of specified properties allow the user to choose the appropriate category for each property. For example, FI_{25} means that aggregates having a flakiness index with a maximum value of 25 are required. The guidance document includes a no requirement option, i.e. where a property has been judged not relevant or an alternative method is used to cover the end use requirement. Examples include use of AAV in preference to the micro-Deval test; and no requirement for resistance to abrasion from studded tyres (as these are not allowed on UK roads) and for the Sonnenbrand test (as degradation through mineralogical instability is not experienced in the UK). An example specification for aggregate for asphalt mixtures, other than specialist surfacings is shown in **Table 3**. This shows the main aggregate properties to be grading, fines content of both the coarse and fine aggregate, flakiness index, resistance to fragmentation, PSV, AAV and durability measured with water absorption and magnesium sulfate soundness if water absorption is >2%.

The skid and abrasion resistance properties of aggregate for surface courses are specified in HD36/06 of the *Design Manual for Roads and Bridges* (Highways Agency, 2006). The minimum PSV required is shown in **Table 4** with the maximum AAV required shown in **Table 5**. The minimum PSV and maximum AAV specification is based on number of commercial vehicles with increasing trafficking

Property	Category
Grading	To meet grading requirements appropriate for the end product
Fines content:	
– Coarse	To meet grading requirements appropriate for the end product
– Fine	To meet grading requirements appropriate for the end product
Flakiness index	FI_{35}
Resistance to fragmentation	LA_{30}
PSV	Site specific (wearing course)[a]
AAV	Site specific (wearing course)[b]
Durability:	
– Water absorption to BS EN 1097-6: 2000, Clause 7	$WA_{24}2$
– For *WA* > 2%, magnesium sulfate soundness	MS_{25}

Note: Specifying values for all other properties described is not necessary because, as specified in BS EN 13043, 4.1.1, 4.2.1, 4.3.1 and 5.1, they are not appropriate to the particular application at end use or origin of the aggregate.

[a] Insert appropriate category from BS EN 13043, Table 13.

[b] Insert appropriate category from BS EN 13043, Table 14.

Table 3 Example specification for aggregate for asphalt mixtures, other than specialist surfacings (adapted from PD 6688-2:2003)

Site category	Site description	IL	Minimum PSV required for given IL, traffic level and type of site									
			Traffic (cv/lane/day) at design life									
			0–250	251–500	501–750	751–1000	1001–2000	2001–3000	3001–4000	4001–5000	5001–6000	Over 6000
A1	Motorways where traffic is generally free-flowing on a relatively straight line	0.30	50	50	50	50	50	55	55	60	65	65
		0.35	50	50	50	50	50	60	60	60	65	65
A2	Motorways where some braking regularly occurs (e.g. on 300 m approach to a slip-off)	0.35	50	50	50	55	55	60	60	65	65	65
B1	Dual carriageway where traffic is generally free-flowing on a relatively straight line	0.30	50	50	50	50	50	55	55	60	65	65
		0.35	50	50	50	50	50	60	60	60	65	65
		0.40	50	50	50	55	60	65	65	65	65	68+
B2	Dual carriageway where some braking regularly occurs (e.g. on 300 m approach to a slip-off)	0.35	50	50	50	55	55	60	60	65	65	65
		0.40	55	60	60	65	65	68+	68+	68+	68+	68+
C	Single carriageway where traffic is generally free-flowing on a relatively straight line	0.35	50	50	50	55	55	60	60	65	65	65
		0.40	55	60	60	65	65	68+	68+	68+	68+	68+
		9.45	60	60	65	65	68+	68+	68+	68+	68+	68+
G1/G2	Gradients >5% longer than 50 m as per HD 28	0.45	55	60	60	65	65	68+	68+	68+	68+	HFS
		0.50	60	68+	68+	HFS	HFS	HFS	HFS	HFS	HFS	HFS
		0.55	68+	HFS	HFS	HFS	HFS	HFS	HFS	HFS	HFS	HFS
K	Approaches to pedestrian crossings and other high-risk situations	0.50	65	65	65	68+	68+	68+	HFS	HFS	HFS	HFS
		0.55	68+	68+	HFS	HFS	HFS	HFS	HFS	HFS	HFS	HFS
Q	Approaches to major and minor junctions on dual carriageways and single carriageways where frequent or sudden braking occurs but in a generally straight line	0.45	60	65	65	68+	68+	68+	68+	68+	68+	HFS
		0.50	65	65	65	68+	68+	68+	HFS	HFS	HFS	HFS
		0.55	68+	68+	HFS	HFS	HFS	HFS	HFS	HFS	HFS	HFS
R	Roundabout circulation areas	0.45	50	55	60	60	65	65	68+	68+	HFS	HFS
		0.50	68+	68+	68+	HFS	HFS	HFS	HFS	HFS	HFS	HFS
S1/S2	Bends (radius <500 m) on all types of road, including motorway link roads; other hazards that require combined braking and cornering	0.45	50	55	60	60	65	65	68+	68+	HFS	HFS
		0.50	68+	68+	68+	HFS	HFS	HFS	HFS	HFS	HFS	HFS
		0.55	HFS	HFS	HFS	HFS	HFS	HFS	HFS	HFS	HFS	HFS

cv, commercial vehicles

Table 4 Minimum PSV of chippings, or coarse aggregate in unchipped surfaces, for new surface courses (adapted from HD36/06)

requiring a higher quality aggregate. The PSV specification is further subdivided by site where the risk of an accident increases the PSV requirement of the aggregate. The PSV specification also includes intervention levels for the surface course below which increased monitoring of wet skid resistance or resurfacing is necessary.

Future trends

Future trends in the use of aggregate for bituminous materials will be governed by sustainable development. Underpinning this is the need for improved understanding and awareness not only of materials but also in the holistic view of a sustainable and long-lasting transport infrastructure. Recycled and secondary materials now account for 24% of the total aggregate production in the UK. However, continued sustainable development also requires increased exploitation of virgin materials. This requires careful resource planning so that future reserves are not sterilised by industrial or housing development.

Sustainable development requires better use of aggregates in materials that not only meet the performance

Traffic (cv/lane/day) at design life (see 3.15)	<250	251–1000	1001–1750	1751–2500	2501–3250	>3250
Max AAV for chippings for hot-rolled asphalt and surface dressing, and for aggregate in slurry and microsurfacing systems	14	12	12	10	10	10
Max AAV for aggregate in thin surface course systems, exposed aggregate concrete surfacing and coated macadam surface course	16	16	14	14	12	12

cv, commercial vehicles

Table 5 Maximum AAV of chippings, or coarse aggregates in unchipped surfaces, for new surface courses (adapted from HD36/06)

Test	Method	Proposed specification	
		Traffic (ESAL)	Limit
Uncompacted void content of coarse aggregate	AASHTO TP56 Method A	Less than 100 000 100 000 and above	40% minimum 45% minimum
Flat or elongated particles in coarse aggregate	ASTM D4791	All	50% minimum
Uncompacted void content of fine aggregate	ASTM C1252 Method A	Less than 500 000 500 000 and above	40% minimum 45% minimum
Micro-Deval test	AASHTO TP58	All	15% maximum
Magnesium sulfate soundness test	AASHTO T104	All	20% maximum

Table 6 Test recommendations and proposed specification limits (adapted from White *et al*., 2006)

expectation but which last longer. Apart from the cross-testing work carried out on proposed European Standard tests, relatively little research has been reported into the predictive ability of the test methods to assess aggregate performance as part of a bituminous material. The cross-testing work was simply to determine how laboratories carried out an existing method; it did not set out to develop new test methods that better predicted performance of the aggregate as part of a bituminous material. Woodward (1995) compared the BS 812 to the proposed European test methods. This considered the methods in detail and assessed the effect of test variables and rock type highlighting limitations with many of the BS 812 and proposed European methods. Correlation between data sets showed many of the properties to be related with a general conclusion being that increased PSV for any rock type was achieved at the expense of other properties.

In contrast to the UK, there has not been the same historical emphasis placed on skid resistance in the USA and so the historical role of the aggregate was of lesser importance compared to that of the bitumen. The range of aggregate test methods in America was never as extensive as those included in BS 812 or the recent European standards. With limited exceptions the role of aggregate as part of a bituminous material was not considered in the Strategic Highway Research Program that ended in the early 1990s. However, since SHRP there have been important research projects carried out to determine which methods best measure key aggregate properties related to pavement performance for evaluating and selecting aggregates or in conjunction with performance-related specifications. This is in marked difference to the process of European Standard test method selection where the chosen methods were picked by committee and not on their ability to predict performance when used in a bituminous material.

Prowell *et al*. (2005) presented a critical review of technical literature available and research in progress since the end of the SHRP in 1993 on the impact of the aggregate properties specified by the Superpave mixture design method on the performance of hot-mix asphalt. Kandhal and Parker (1998) evaluated aggregate test methods through literature study and laboratory testing identifying predictor test methods that related to pavement performance. These were evaluated in accelerated load testing and/or in-service pavement fatigue, rutting and moisture susceptibility studies by White *et al*. (2006) with the aim of recommending performance-based test methods. This highlighted some interesting misconceptions about aggregate properties and resulted in the relatively small set of aggregate tests dealing with particle shape, angularity, durability and soundness shown in **Table 6**.

This is essentially a simplified version of the example specification given in **Table 3** based on European test methods. It is interesting to note that the wet micro-Deval test was selected during this American research programme in preference to other methods when it is dismissed in the UK guidance document PD 6682-2 as not being relevant for surfacing course use. In contrast, Woodward (1995) found that the higher PSV aggregates used in the UK tended to have poorer wet micro-Deval values, suggesting that they were susceptible to greater levels of wear due to wet trafficking than predicted in the dry AAV test method.

Climate change predictions suggest greater variation in temperature and rainfall, factors which affect the properties of bituminous materials. Moisture sensitivity has always been a problem with all bitumen bound layers. The SATS test (Collop *et al*., 2004) was developed as a combined bitumen ageing and moisture sensitivity test for binder and base mixtures. The method uses expensive test equipment and needs revision to make it more adaptable to optimise bitumen and aggregate blends for all bitumen bound layers.

Ravelling of the surface course is a potential problem faced by many high stone content mixtures. Research needs to determine whether ravelling occurs at the aggregate–bitumen interface or within the aggregate, i.e. the use of higher-specification aggregates weakened by rainfall may not be able to match improvements in bitumen adhesion and increased flexibility of the mixture leading to ravelling-type failure. Therefore, the continued use of higher PSV aggregate, which has been shown to relate to lower strength and durability properties, needs to be reconsidered. This has important implications as it effects the specification of aggregate in the UK. The requirement for higher PSV aggregate may need to be relaxed in line with most other countries within Europe and the world. This implies the need for further collaborative research between the four main factors involved in road use: the vehicle, the tyre, the properties of the road surface and the road user.

In terms of the skid resistance properties of the road surface and of the aggregate used to provide those properties, the aggregate PSV test has been the main research tool in the laboratory. Recent studies have highlighted issues with the method and its ability to predict performance. The trend is now to consider the role of aggregate on the skid resistance of the bituminous material rather than simply considering the aggregate itself. Skid resistance research needs to correlate in-service performance, typically measured in the UK with SCRIM or GripTester and laboratory prediction of the mixture rather than the aggregate. Examples of current work include use of the Wehner Schulze equipment (Huschek, 2004; Allen *et al.*, 2007) as a replacement to the PSV test, use of accelerated wear equipment such as the Road Test Machine (Woodward *et al.*, 2007) to predict development of skid resistance under a full-scale tyre and the work of Vollor and Hanson (2006) in developing a simple polishing device that can be used in the laboratory or on site.

Key points

The key points regarding aggregate properties and test methods for use in bituminous materials are as follows:

- Although there are many types of rock that can be quarried, not all can produce aggregate with suitable properties for use in bituminous materials.
- The methods used to test aggregate properties were developed many years ago when the types of aggregate use and trafficking conditions were much different than those of today. British Standards have been replaced by European Standards, Specifications and National Guidance documents that have reduced the basic requirements of aggregate for bituminous materials.
- Almost all aggregate tests assess the properties of single-size unbound particles and not the graded bitumen-bound material. There is the need to research the role of aggregate as part of the mixture in the performance of bituminous materials involving development of better predictive laboratory methodologies.
- The effect of climate change needs to be addressed, e.g. the increased effect of greater temperature ranges causing thermal extremes and prolonged wetting causing weakening of the aggregate and the aggregate–bitumen bond.
- Emphasis on understanding aggregate properties and the limitations of its use will shift from being reliant on a national specification requirement to an issue that bituminous materials suppliers must acknowledge given the trend towards acceptance of risk, i.e. they cannot afford premature failure related to aggregate properties.

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Sources of web-based information

There are numerous sources of information now accessed by way of the web, relating to aggregate properties and their use in bituminous materials. The following is a summary of the main aggregate producers, professional institutions, government and research bodies.

AggRegain http://www.aggregain.org.uk
Aggregate Industries http://www.aggregate.com
British Geological Survey http://www.bgs.ac.uk
Building Research Establishment http://www.bre.co.uk
County Surveyors Society http://www.cssnet.org.uk
Highways Agency http://www.highways.gov.uk
Idiots Guide to Highway Maintenance http://www.highwaysmaintenance.com
Institute of Asphalt Technology http://www.insofasphalt.org
Institute of Quarrying http://www.quarrying.org
International Center for Aggregates Research http://www.engr.utexas.edu/icar/
Lafarge Aggregates http://
Mineral Industry Research Organisation http://www.miro.co.uk
Mineral Industry Sustainable Technology Programme http://www.mi-st.org.uk
National Stone Centre http://www.nationalstonecentre.org.uk
National Stone, Sand and Gravel Association http://www.nssga.org
Quarry Products Association http://www.qpa.org
Tarmac http://www.tarmac.co.uk/Aggregates/
The Virtualquarry http://www.virtualquarry.co.uk
Transport Research Laboratory http://www.trl.co.uk

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Chapter 25

Asphalt properties and test methods

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CONTENTS

Asphalt mixtures constitute the main component of a flexible pavement. Asphalt mixtures (also known as asphalt, bitumen-bound or bituminous materials) vary in terms of their bitumen content, bitumen grade, aggregate grading and aggregate size. Depending on the position of the asphalt in a pavement structure, it has to perform a number of functions including providing a smooth riding surface and protecting the underlying granular layers and soil (subgrade) from environment factors and external loads. To achieve this asphalt needs to be resistant to cracking, permanent deformation (rutting) and have a stiffness modulus sufficiently high to carry traffic loads.

Introduction

Asphalt comprises granular material with a binding component: bitumen. The physical properties of bitumen are extremely important to a full understanding of asphalt, and the reader is referred to Chapter 2 of this manual for more details. Equally, because of the granular structure of asphalt, knowledge about granular material behaviour is of considerable assistance when considering asphalt behaviour. Indeed, for a complete understanding of asphalt, it is useful to consider the interaction between individual aggregate particles, and this is considered in more detail below.

The major application for asphalt is in the construction of roads, airfield runways and taxiways, and this section concentrates on the properties and test methods of relevance to these applications. However, asphalt is also employed for a wide range of specialist uses, including for railway, recreational and hydraulic applications, for which similar principles apply.

The mechanics of asphalt behaviour

Asphalt consists of an aggregate skeleton – as in the case of an unbound material – and a binder, which includes filler-size particles within it. There will also be a certain percentage of air voids. If strain is to take place, the aggregate skeleton has to deform and, as in the case of an unbound material, there are two basic mechanisms:

- compression at particle contacts;
- inter-particle slip (combined with rotation and separation).

If no slip were to take place between particles, then the approximate stiffness of the material could be predicted from the stiffness modulus of the rock from which the aggregate is formed (say 100–200 GPa), a contact law and a means of calculating the additional effect of the binder. In the case of an asphalt, the stiffness of a conventional road bitumen at 20°C and traffic loading frequency is typically between 20 and 50 MPa, with bitumen-filler mortar stiffness around 2.5 times higher. This gives a predicted mixture stiffness of 5000–7000 MPa, which is approximately the expected range for undamaged materials. The implication is that the bitumen initially allows very little inter-particle slip or separation to occur; it is only as damage occurs that this situation changes.

The micromechanics of asphalt damage

In a typical dense asphalt mixture, the aggregate occupies around 85% of the total volume (of which about 5% is filler), with about 10% bitumen and 5% air voids. If the filler is considered as a binder additive, the aggregate skeleton represents about 80% of the volume and three-quarters of the remaining space is filled with bitumen-filler mortar. **Figure 1** illustrates the situation at a particle contact. If slip (or particle separation) is to take place, then this loading must impose a strain in the mortar, which becomes larger as the point of contact is approached. Mathematically, the strain right at the contact point becomes infinite – and the only mechanism capable of producing infinite strain is fracture. Damage to an asphalt, visible as a reduction in stiffness under repeated loading (for example, comparing materials from trafficked and untrafficked areas in a highway), therefore takes the form of localised fractures at particle contacts, allowing slip and separation to occur; experience suggests that such damage commences from the very first load application.

It would be logical to assume that, where slip (± rotation or separation) takes place, a fracture zone will extend out from the contact point until it reaches a region where the strain is low enough not to cause fracture, giving a measure of freedom to the two particles involved. The particles can slip (or separate) by a small amount.

Again, it would be logical to assume that these localised fracture zones cannot be static. Low temperature will lead

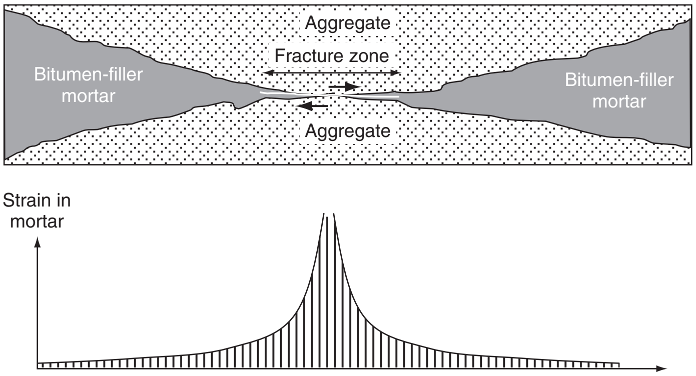

Figure 1 Slip at particle contacts (Thom, 2008)

to stiff binder and restricted inter-particle movement, resulting in additional stress being taken through the binder and therefore an increased chance of fractures occurring – and of existing fracture zones expanding. However, high temperature represents more than the mere absence of high binder stress; it also allows the phenomenon known as 'healing' to occur. At high temperature, bitumen is a liquid, fractured or not, and consequently the molecules either side of a fracture can, over time, recombine to create continuous binder phase once more. This will happen most easily when binder viscosity is at its lowest (at high temperatures), resulting in a state of dynamic flux. Winter leads to high stiffness but increased damage; summer leads to low stiffness but also healing. In theory, a state of dynamic equilibrium can be reached, depending on how much damage the traffic is doing and how quickly.

Linearity

Since unbound aggregate behaviour is known to be highly non-linear and bitumen is visco-elastic rather than elastic, one might expect the stiffness behaviour of an asphalt to be very far from linear. However, in an asphalt, the magnitude of strain taking place within the aggregate skeleton is small compared to that common in an unbound material, and in the small-strain region the stiffness of an unbound material is approximately constant. Furthermore, the loading rate is usually high enough to ensure that the bitumen is kept near the elastic end of its visco-elastic behaviour spectrum. The combination is sufficient to give asphalt an approximately linear stress–strain response. However, the actual value of stiffness modulus will vary significantly with bitumen properties – which in turn depend on temperature and loading rate.

Bitumen adhesion

The micromechanical explanation above makes one crucial assumption, namely perfect bond between bitumen and aggregate, i.e. perfect adhesion. If this is the case then there is no preferential fracture plane within the binder and the resistance to fracture at a particle contact will be governed simply by the cohesive strength of the binder itself. However, if there is any lack of adhesion then fracture will take place preferentially along one of the interfaces between an aggregate particle and the adjacent bituminous mortar, and the development of such a fracture will occur more easily than it otherwise might have.

Adhesion is therefore a very important property but, unfortunately, it is not readily measurable. The pneumatic adhesion tensile testing instrument (PATTI) – ASTM (2002) – has been used for measurement of bitumen adhesion, but is not an industry standard. The effects of lack of adhesion can be observed indirectly from mixture tests and, more dramatically, in failed asphalt pavements. Mixture tests indicate low stiffness, low fatigue resistance and high permanent deformation under multi-cyclic load, since all three properties are strongly related to the ease with which internal fracture occurs.

In terms of the cause of poor adhesion, one important parameter appears to be the acidity or alkalinity of the aggregate. Limestone aggregate generally makes for high-quality asphalt whereas many igneous rocks have proved to be less trouble-free, as have many natural gravels (although a rounded particle shape would cause poor performance even with good adhesion). Collop *et al.* (2004) are among those who report a significantly enhanced performance with limestone aggregate compared with granite, although use of an additive negated this difference. Lay (1990) provides a useful insight into the mechanisms which may be at work. Current research is concentrating on the subject of 'surface energy', a property of the aggregate surface similar in concept to the well-known 'surface tension' effect at a liquid surface. A particular issue is that most aggregates attract water (if available) in preference to bitumen – hence the need to ensure that particles are absolutely dry before mixing with bitumen.

In order to counteract acidity, additives such as hydrated lime are frequently used, although other chemical additives are marketed and have also proved effective.

Asphalt stiffness

One of the key properties of asphalt for pavement design is stiffness modulus. However, from the foregoing discussion, it is clear that an asphalt does not have a single stiffness value; it is both temperature and loading rate dependent because it depends on the efficiency with which the binder restricts inter-particle movement in the aggregate, and this in turn depends on binder stiffness. It is also dependent on the mixture proportions used.

Predicting asphalt stiffness

Many equations and procedures have been devised to predict binder stiffness, but the most widely adopted is

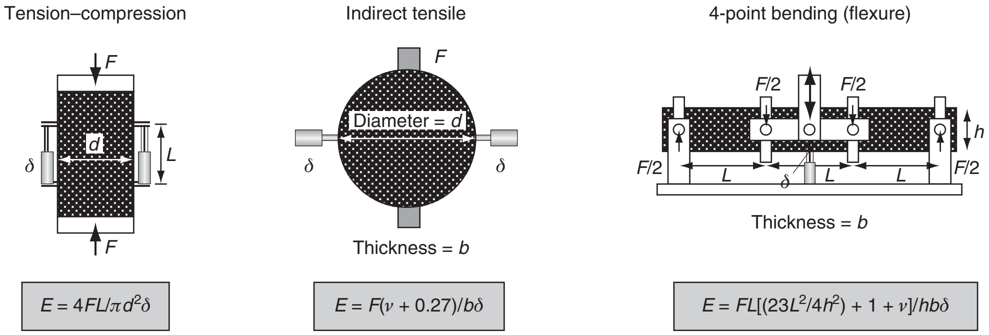

Figure 2 Typical asphalt stiffness tests (Thom, 2008)

undoubtedly that known as 'Van der Pohl's nomograph' (Van der Poel, 1954), a chart from which binder stiffness (E_{binder}) is derived from temperature (T) relative to softening point (SP), the penetration index (PI) of the binder – refer to Chapter 2 for details – and load pulse duration (t). The following approximate formula (Ullidtz, 1978) matches the predictions from the nomograph over a restricted range of input parameters:

$$E_{\text{binder}} = 1.157 \times 10^{-7} \times t^{-0.368} \times 2.718^{-PI} \times (SP - T)^5 \text{ MPa}$$

The parameter found to be the most useful in describing mixture proportions is Voids in Mixed Aggregate (VMA), essentially the percentage of a mixture which is not aggregate; in other words, the binder volume plus the air void volume. Many different equations have been proposed to predict mixture stiffness, of which the following is one (Brown, 1978):

$$E_{\text{mixture}} = E_{\text{binder}} \times \left[1 + \frac{257.5 - 2.5\text{VMA}}{n \times (\text{VMA} - 3)}\right]^n \text{ MPa}$$

where:

$n = 0.83 \times \log_{10}[4 \times 10^4 / E_{\text{binder}}]$

and:

VMA is in %; E_{binder} is in MPa.

It is worth noting that binder content is not taken into account except as a component of VMA. This is logical since it will only be the relatively small amount of binder which sits in the immediate vicinity of a particle contact which significantly influences mixture stiffness. VMA itself acts as a broad measure of the efficiency with which the aggregate particles are packed together.

In reality, such predictions can be no more than approximate. VMA is a very crude measure of packing when the infinite number of possible aggregate gradations is considered. Furthermore, since inter-particle slip is an element in asphalt strain, this means that the frictional properties at contacts should also be relevant.

Measuring asphalt stiffness

The range of laboratory test options for measuring stiffness includes bending tests (2-point bending to trapezoidal specimens, and 2-, 3- or 4-point bending to prismatic specimens), indirect tensile tests to cylindrical specimens and direct uniaxial tests (tension–compression to cylindrical specimens and direct tension to cylindrical and prismatic specimens). Some typical test set-ups are illustrated in **Figure 2**.

Of these, the tension-compression test has the least complex stress conditions and is therefore favoured by many researchers; however, it is the least practical to carry out because of the need to glue the end platens. Even if the glue is effective, there will be a concentration of stress around the perimeter at the glue–specimen interface, potentially causing premature specimen failure. The usual way to overcome this is to use a necked specimen, greatly adding to the complexity of specimen preparation. The indirect tensile test is very quick and easy to conduct and is therefore favoured by industry, but suffers from having highly complex stress conditions, particularly near to the loading points, and the interpretation is therefore less certain. The 4-point bending test is used when testing for fatigue, and it also gives a very satisfactory stiffness measurement. One of the key drivers is to be able to test a specimen which can be prepared from a core taken from the pavement – which means that the indirect tensile test scores highly.

It is worth noting that stiffness measured in any of these test modes cannot be used directly in pavement analysis and design without correction. Test temperature is, of course, important, and testing should be carried out in a temperature-controlled environment, the specimen having also been stored at test temperature for several hours prior to test. However, loading rate is also critical. A pulse of load from a vehicle on a road may typically take about 10–15 milliseconds to reach peak stress, whereas most of these tests are generally carried out at a considerably slower rate. Thus, for example, in the commonly

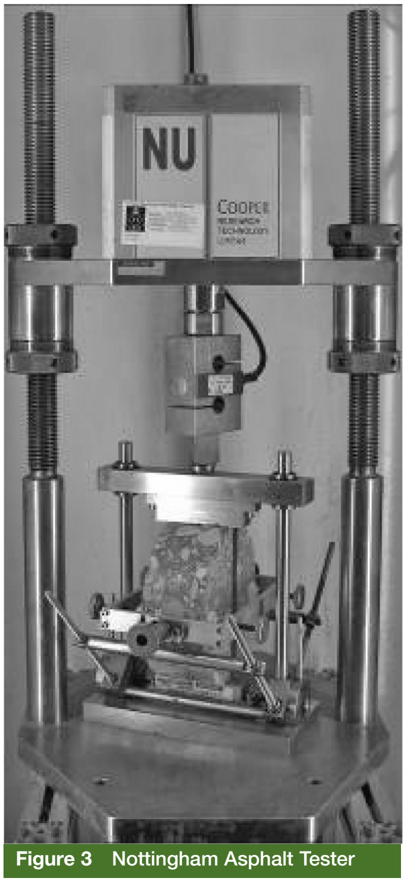

Figure 3 Nottingham Asphalt Tester

used pneumatically powered 'Nottingham Asphalt Tester', (NAT – Brown *et al.*, 1994), see **Figure 3**, with about 125 milliseconds to peak, the stiffness measured would typically be only around 70% of that applying under fast-moving traffic.

Typical stiffness values

Figure 4 presents data on different asphalt types, together with a stiffness/temperature conversion equation. The information given here is intended to give the reader a general idea of what to expect from various generic material types, but should *not* be taken as applying in every case.

One significant influencing factor is bitumen ageing, and the stiffness of a mixture will tend to increase throughout its life as the viscosity of the binder increases. For example, if a new dense asphalt base with 50 pen binder has a stiffness in the road of about 5000 MPa at 20°C at the start of its life (by which time the penetration of the bitumen has already decreased to around 35 purely as a result of mixing and laying), then (if undamaged) this may have increased to 6500 MPa after 10 years in a climate such as the UK, with much more rapid stiffness increase in hotter climates.

It is worth noting also that the stiffness of the cold-mix asphalt examples in **Figure 4** are lower – and one of them is less temperature sensitive. The reason is that the bitumen coating of the aggregate is generally less good, giving situations in which two pieces of aggregate interact without the restraint of bitumen. This immediately increases the freedom of movement, thus decreasing stiffness; all such dry contacts will also be independent of bitumen viscosity, thus decreasing temperature and loading rate sensitivity. The same effect can be seen in a partially fatigued conventional asphalt.

Finally, it should be noted that a value of 0.3 or 0.35 is commonly adopted for Poisson's ratio of asphalt.

Fracture and fatigue of asphalt

One of the key design issues for an asphalt pavement is cracking. Cracks may arise for a number of reasons, but

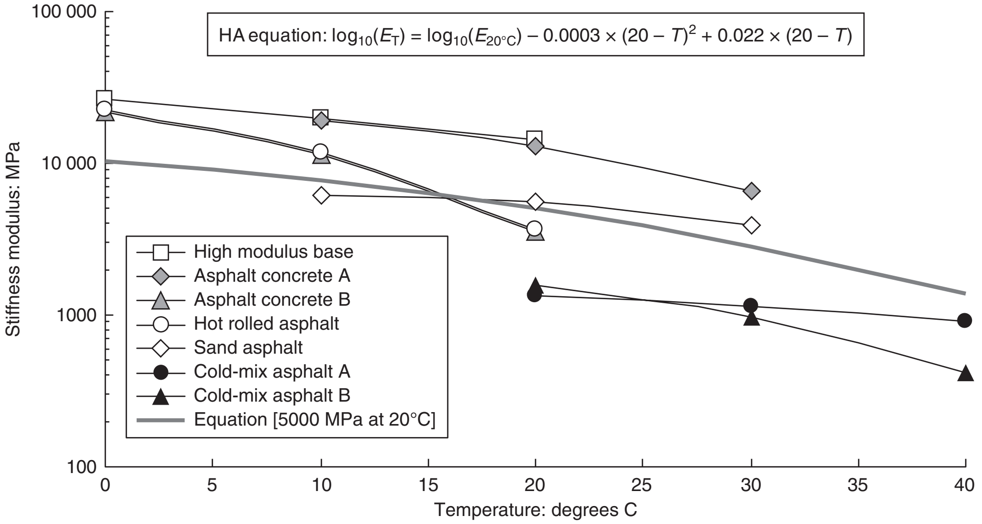

Figure 4 The effect of temperature on asphalt stiffness. The data shown were taken from Thom *et al*. (1997), Ibrahim (1998) and Sunarjono (2008). The equation shown is given in the UK Highways Agency standard HD29 (Highways Agency, 2001)

they are always a problem, leading to loss of ride quality and also allowing water to enter the pavement foundation. It is important, therefore, that the phenomenon is properly understood.

Low-temperature fracture

This is a significant problem in continental climates, including desert climates such as those of North Africa. In fact, the label 'low-temperature' is misleading, since the primary driving force is that of a large day–night temperature difference. Asphalt, like any other material, expands and contracts with temperature changes. Bitumen itself is particularly susceptible, giving asphalt a typical thermal expansion coefficient α of around 1.8×10^{-5} per degree C, significantly higher, for example, than that of hydraulically bound materials. As a result, night-time cooling imposes a significant volumetric strain. If the bitumen is soft, the asphalt can readily accommodate this by adjusting itself such that all the strain occurs either vertically or laterally rather than longitudinally (i.e. along the road). However, if the bitumen is hard, such self-adjustment induces high internal stress and, at the limit, the stress level will be high enough to cause fracture.

In climates such as those of North America, Northern Europe and Central Asia, the critical point occurs when the night-time temperature drops well below zero; in climates such as those of the deserts of Arabia and the Sahara, the night-time minimum remains the main driving force, exacerbated by rapid ageing of bitumens in desert climates, making them brittle enough to fracture.

Tests for susceptibility to low-temperature fracture are empirical. The primary tests are those already mentioned for the bitumen itself (see Chapter 2), but it is also common to carry out fracture strength tests on the mixture. For example, the Indirect Tensile Strength Test is sometimes carried out on asphalt using a suitably low temperature.

Fatigue damage

Considering fatigue in the context of metals, in which a crack first has to 'initiate' at some defect within the material and then 'propagate', provides an excellent representation of what happens in a relatively uniform material such as a metal and expressions such as the well-known Paris law have been derived to describe this behaviour. The Paris law (Paris and Erdogan, 1963) relates the rate of crack propagation to a 'stress intensity factor' at the crack tip, a measure of stress concentration which takes account of the sharpness of the crack. However, this is not the situation in an asphalt.

Asphalt consists of an aggregate skeleton surrounded by binder, with a random distribution of air voids. The key stress concentration points are at particle-to-particle contacts and, as described above, local fracture has to occur if damage is to take place. Fatigue therefore consists of the enlargement of these fracture zones at contacts and the development of new fractures. Maillard *et al.* (2004) describe tests which involve exactly this type of interparticle fracture. The effect is primarily controlled by movement within the aggregate skeleton, i.e. strain within the mixture; large movement will rapidly expand fracture zones and initiate new ones. Fatigue life is therefore primarily a function of the 'elastic' strain under load within the mixture.

Tests for fatigue of asphalt

Tension–compression, indirect tensile and flexural tests have all been successfully employed for fatigue testing, as well as more complex configurations such as torsion. Tension–compression induces the simplest stress conditions and is used extensively by researchers, but it is not as convenient as other configurations. The most popular are undoubtedly indirect tensile and flexural, with 4-point bending and trapezoidal providing alternative flexural testing options. The trapezoidal shape seems awkward, but it has the advantage that it produces a maximum tensile stress well away from any stressed boundary (typically at about one-third of the specimen height). **Figure 5** shows the trapezoidal set-up and **Figure 6** illustrates typical test modes and also presents typical test data, expressed as loss of stiffness as a function of number of load cycles (relative to the number of cycles to specimen failure).

An important distinction exists between tests which are load controlled, in which strain increases during the test, accelerating towards failure at the end of the test, and those which are displacement controlled, in which stress reduces during the test as damage occurs and stiffness reduces. These two types of test give quite different fatigue lives, even with identical stress and strain at the first load cycle. In the case of displacement control, there is often no actual failure of the specimen and no visible damage at all until the stiffness has reduced to a small fraction of its original value.

Specialists disagree as to which type of test is the most appropriate for pavements; generally displacement control is considered appropriate for thin asphalt layers, where strains are principally influenced by other layers, whereas load control is considered more appropriate for thicker layers, where the asphalt is the dominant material. In reality, neither is totally lifelike. Laboratory loading can never be realistic since it can never duplicate the sequences of multiple loads followed by at least a second or two of 'rest' induced by real vehicle loading. The result is that those who wish to apply laboratory fatigue data in practice commonly have to treat the test as an indicator and then apply a considerable adjustment factor to the data in order to arrive at a usable result.

Key practical test requirements are:

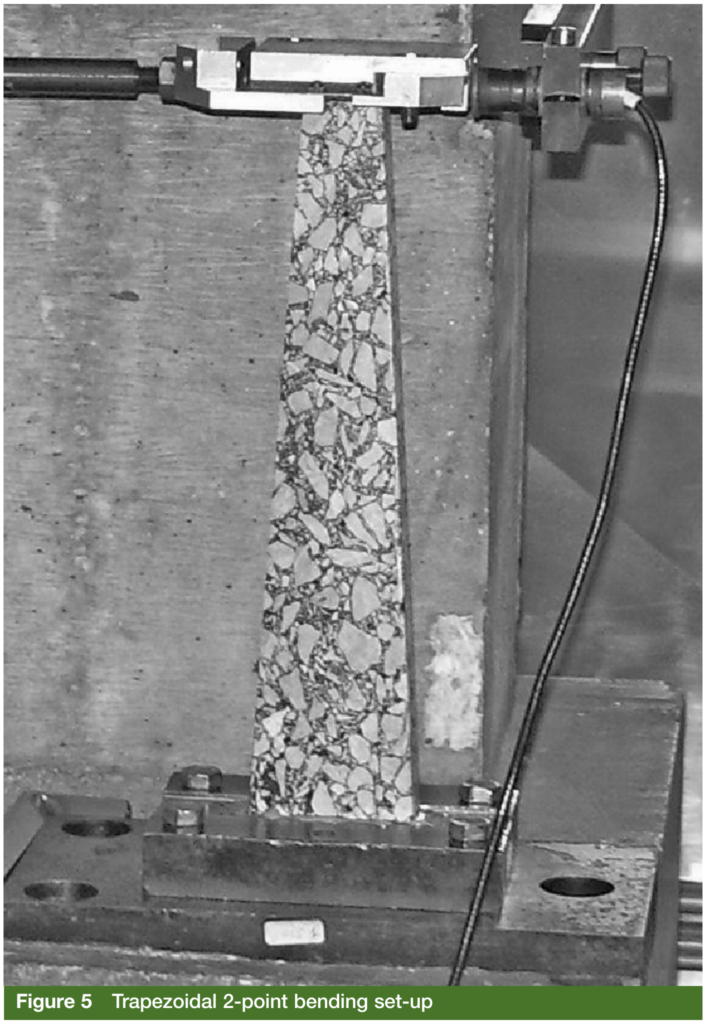

Figure 5 Trapezoidal 2-point bending set-up

- the specimen size should not be so small in any of its dimensions that the result can be greatly influenced by local aggregate orientation;
- the test should be simple enough for practical application.

It is also desirable to avoid excessive permanent deformation during a test, a significant criticism of the indirect tensile test if carried out under compressive load only. Despite the undoubted convenience of the indirect tensile fatigue test as a means of comparing the fatigue resistance of different materials, the significant permanent deformation induced means that the results should be treated with caution. For some materials, particularly low stiffness materials, the results may understate the true fatigue resistance.

Development of a fatigue characteristic

Whichever test is used, it is necessary to express the data in a meaningful way, including selection of an appropriate failure criterion. If the test is load controlled, as in the example in **Figure 6**, failure is clear; the specimen falls apart at the end of the test. However, if displacement control is used, then it is usual to work to 50% stiffness loss in place of total failure. This is an arbitrary value, but is justified as commonly representing the point where damage begins to accelerate after a relatively stable phase. Note that the data in **Figure 6** suggest that, for any given material, this assumption is only valid over a certain temperature range.

Next, a series of tests has to be carried out at different levels of load (and therefore stress) or displacement (and therefore strain), generally at least six and preferably ten. The results can then be plotted as shown in **Figure 7**. It is

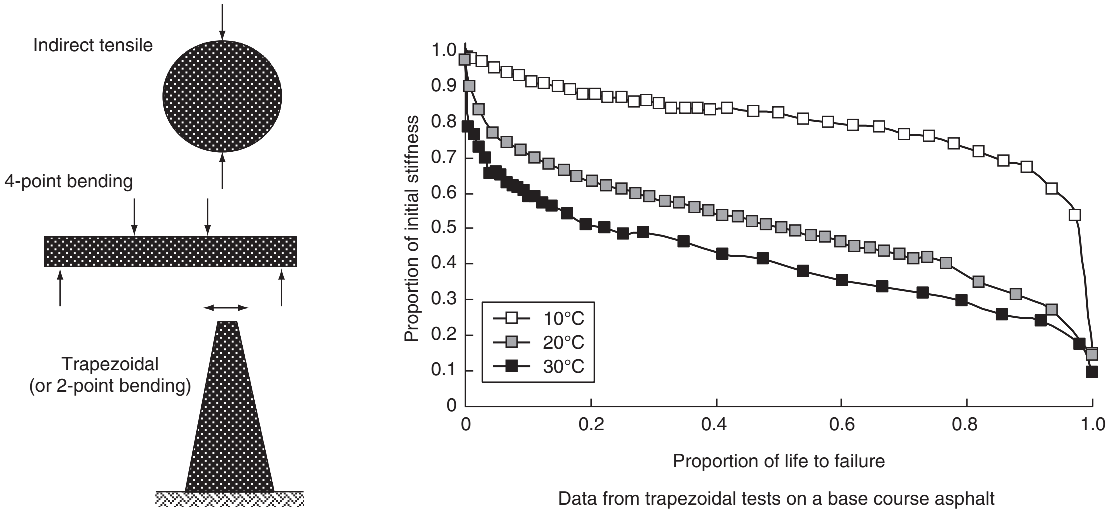

Figure 6 Typical asphalt fatigue tests. Data taken from Choi *et al*. (2005)

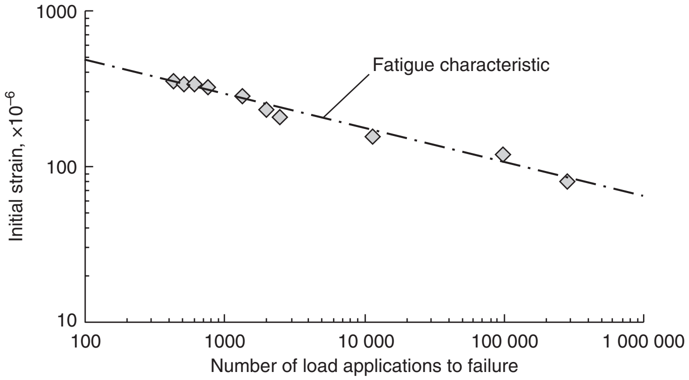

Figure 7 An asphalt fatigue characteristic (Thom, 2008)

usual to plot number of load applications to failure against strain (strain at the first load cycle in the case of a load-controlled test) rather than stress, since experience is that the resulting 'characteristic' is approximately independent of temperature and loading rate, at least within a broad range, whereas this is not the case in a plot against stress. This is not unexpected since damage is primarily related to inter-particle movement, i.e. strain.

It cannot be assumed that test temperature will never affect fatigue life so long as the number of load applications to failure are expressed against strain, and the example in **Figure 6** gives an indication of the different forms of behaviour that occur. Experience is that life will be longer, i.e. the fatigue characteristic in **Figure 7** will be further to the right, if the temperature exceeds a certain value. This value will generally, but not always, be over 30°C. This means that it is *not* automatically acceptable to adjust test temperature upwards in order to allow a target strain to be reached at a lower load level.

Healing

This phenomenon has been referred to already in relation to asphalt stiffness, the point having been made that bitumen fracture at particle contacts tends to occur in cold weather, but that binder continuity can be restored in warm weather due to viscous flow and molecular remixing. The healing effect is responsible for the improved fatigue lives found in tests at high temperature; it also means that there will be differences between tests depending on the details of load pulse frequency and any 'rest periods' allowed between load pulses. This is illustrated in **Figure 8**.

However, understanding of healing is incomplete. It is largely because of healing that it is not possible to apply a laboratory fatigue characteristic directly in pavement performance prediction, since the degree of healing in the pavement will be quite different from that in the laboratory test; a 'shift factor' will be required.

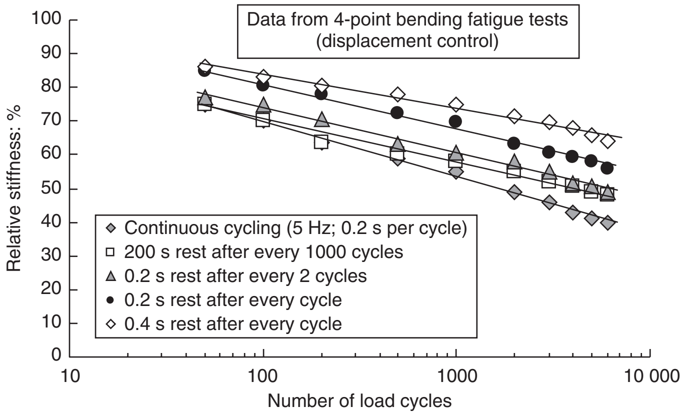

Figure 8 The influence of rest periods on fatigue behaviour. These data, taken from Oliveira (2006), were obtained on a grouted macadam mixture, with 200 pen (i.e. very soft) binder. The use of this binder will have accentuated the healing effect, but the trends shown also apply to conventional asphalt mixtures

Permanent deformation

As with stiffness and fatigue, the fundamental structure of asphalt influences permanent deformation. Typically, asphalt can only deform if the aggregate skeleton deforms; thus, the resistance of an asphalt to deformation is intimately tied up with the stability of that aggregate skeleton. Therefore, the same properties which lead to low plastic strain in an unbound material (angular particles, broad gradation, large particle size) also give stability to an asphalt. Although binder technology may help, it is difficult to produce a deformation-resistant mixture without an appropriate aggregate skeleton.

The influence of binder

Despite its secondary nature, the role of the binder is still extremely important. Plastic strain of the aggregate skeleton is greatly inhibited by the binder. In fact, throughout most of the year in a temperate climate, when pavement temperature is low or moderate, plastic strain will be negligible. This is because the binder takes much of the stress away from particle contacts. However, as the temperature increases, the stress taken across particle contacts increases, leading to increased danger of interparticle slip, some of which will be irreversible. The binder may self-heal, but the aggregate skeleton will be permanently deformed.

In practical terms, for unmodified bitumens, experience suggests that to mitigate against permanent deformation it is preferable not to allow the pavement temperature to reach the ring and ball softening point of the bitumen. A hard binder will therefore raise the temperature at which the onset of significant plastic deformation occurs.

However, despite its beneficial binding action, bitumen can also exhibit the same sort of pore pressure effect as

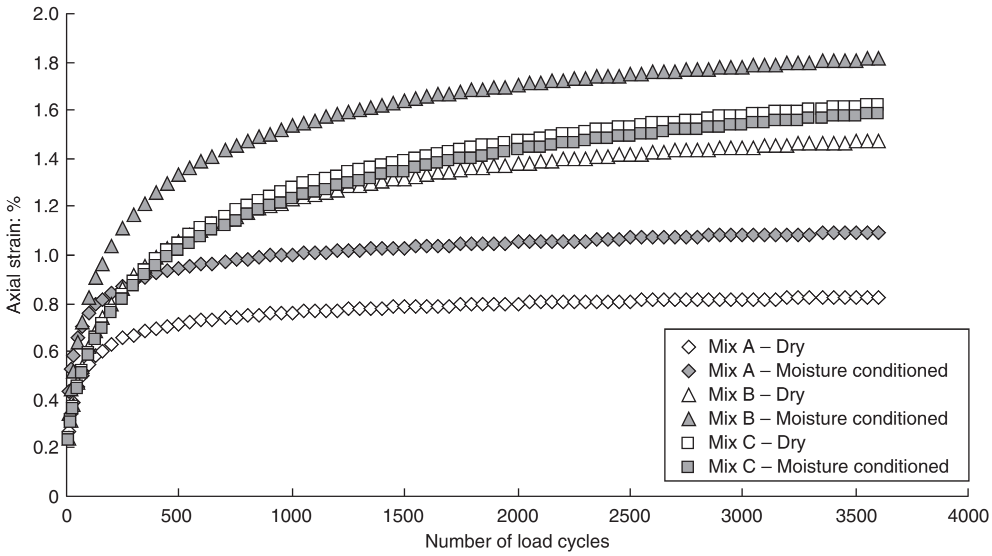

Figure 9 Repeated load axial test data – 100 kPa, 40°C (Airey *et al*., 2004)

exhibited by water in an unbound material. As the aggregate skeleton strains elastically under load, some of the intervening voids will naturally decrease in size and some will increase, purely as a result of the aggregate geometry. If there is plenty of air in the system, the air will readily expand and contract. However, a void that is full of binder cannot change its volume, which means that the binder may have to carry excessive load. Extra compressive stress in the binder will result in less normal stress at aggregate contacts, easier slip and therefore more plastic strain. Many pavements therefore develop rutting simply because there is not enough air in the system to prevent this pore pressure effect. Many specifications guard against this by insisting that a certain minimum percentage of air voids is built into the mixture; however, secondary compaction may still occur under traffic. An ultimate air void content under 1% (in the pavement) is likely to lead to deformation, i.e. rutting.

Measurement of permanent deformation

There is no single property defining resistance to permanent deformation, and testing is only indicative. The most usual forms of test are:

- a creep or repeated load axial test (RLAT), whereby a cylinder of material is loaded (repeatedly in the case of the RLAT) in compression and the resulting axial deformation measured;
- a repeated load triaxial test, specifically an axial test with the addition of confining stress; confinement can be applied by means of an external pressure within a surrounding cell, or by means of a (partial) vacuum applied to the specimen when sealed by a flexible membrane. A practical alternative to application of a confining pressure, pioneered in Sweden, is to use a top platen diameter smaller than the specimen diameter, allowing the outer part of the specimen to provide confinement to the inner part;
- a wheel-tracking test.

The axial test is the most convenient since the test arrangement is simple and the specimen can be obtained from a core through a pavement; the repeated load form of the test (RLAT) is considered to best represent the real-life situation of repeated applications of wheel loading. The test has been criticised as overly severe on base materials, where a considerable degree of confinement would in reality be present. However, it is not intended to be more than an index test, and the severe nature of the test results in intrinsically poor materials showing up very clearly indeed, producing useful information when considering whether and at what level in the pavement such materials may be used. **Figure 9** presents data sets for materials of differing quality. It is worth noting that the materials in **Figure 9** were tested both dry and following 'moisture conditioning' – see below under Durability – since many materials are found to perform worse when water is present.

The triaxial test allows a more accurate representation of the real stress regime applying in a pavement, and also in theory allows the depth at which the material is to be used to be simulated by setting the level of confinement appropriately. However, the test is more complex than the axial set-up and the 'appropriate' level of confinement is open to debate. Since it is still only an index test, it is not clear that the triaxial test has sufficient advantages to warrant its general use, although it does give a much better indication of the performance of low-stability materials such as porous asphalt.

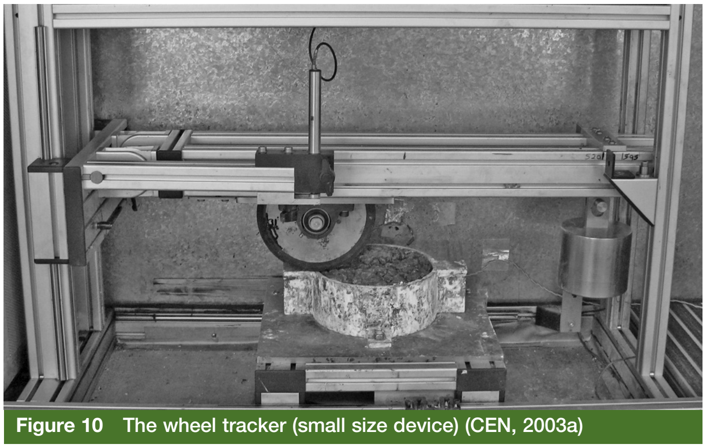

Figure 10 The wheel tracker (small size device) (CEN, 2003a)

Many specifications require a wheel-tracking test, at least for surface course materials (see **Figure 10**).

All are relatively small-scale tests such that the device can be contained within a temperature-controlled cabinet. Details of the load used and test temperature vary from specification to specification, but it is usual to use a high temperature (40–60°C) in order to give a relatively rapid result, see for example CEN (2003a). The close simulation of reality which wheel-tracking gives means that the results, though still index numbers, can be treated as directly related to deformation susceptibility in the pavement.

Durability

The term 'durability' includes several different phenomena, the common theme being that they all take time to develop. This section will consider each aspect of durability in turn. However, the complex nature of the various phenomena, their interrelation and their time dependence mean that devising realistic testing regimes for durability is an almost impossible task.

Ageing

Ageing indicates that ageing affects all bitumens. Chemical changes with time mean that bitumen becomes both stiffer (beneficial) and more brittle (generally not beneficial). Thus, a fatigue characteristic, plotted against applied strain as in **Figure 7**, should tend to shift slightly to the left as ageing occurs. Experience, however, suggests that this shift may not be significant in tests carried out under standard conditions, implying that ageing is not a problem. The difficulty is that standard tests are always rapid, producing failure in a matter of hours rather than years, and the vital component of asphalt behaviour described above, namely healing, is not properly taken into account. An aged bitumen will heal much less readily than an unaged bitumen, with the result that, in the pavement, asphalt is often observed to reach a state of ageing where distress accelerates.

It is important to understand the factors that affect ageing. These are primarily temperature, aggregate absorption (of binder) and the presence of oxygen. Ageing will therefore occur most severely in hot climates, near the road surface, in voided mixtures and where high-absorption aggregates are used. In a dense base, in a temperate climate, using a standard aggregate, ageing should be extremely slow, giving many decades of trouble-free life. This is an excellent reason to achieve good compaction during construction.

Water damage

It is well known that it is essential to dry aggregate thoroughly during asphalt production in order to ensure good bitumen adhesion, and that it is sometimes necessary to add lime or some other adhesion agent in order to facilitate its development. As discussed above, the bitumen–aggregate interface is potentially vulnerable to water. The key to good performance is high density and low permeability; if no water ever reaches these vulnerable interfaces, water damage cannot occur.

There is no agreement as yet on the most appropriate tests to determine susceptibility to water damage. Permeability can be measured directly in a high-pressure permeameter, although this is not a standard tool. Void content can, of course, also be determined, and experience suggests that a void content of about 5% or less results in sufficiently low permeability for water damage to be unlikely. Direct testing can also be conducted by soaking specimens in water, usually at an elevated temperature, and then measuring a suitable mechanical property, typically 'retained' stiffness (or strength), i.e. a percentage value compared to the unsoaked condition, for example, CEN (2003b). A value of 80% or more is usually taken to represent non-water-susceptible material. **Figure 11** illustrates some of the typical procedures.

One key element that is missing from the procedures in **Figure 11** is the dynamic movement of water within and between void spaces under the action of traffic. For this, no test exists other than for purely research purposes (for research in this area, see Gubler *et al.*, 2004). The effect is most likely at the base of a layer, where voids tend to be highest, and where water may become trapped due to high density at the top of the layer beneath. It is also likely where pavements suffer flooding. However, the subject is so complex that it is hard to envisage any direct test which could encompass all the different components of the water damage mechanisms; as with permanent deformation, testing can only be indicative. The key point is that it is highly undesirable to allow water to penetrate and accumulate within an asphalt.

Frost damage

It is commonly observed that pavements suffer significant damage immediately following severe frosts, and some of

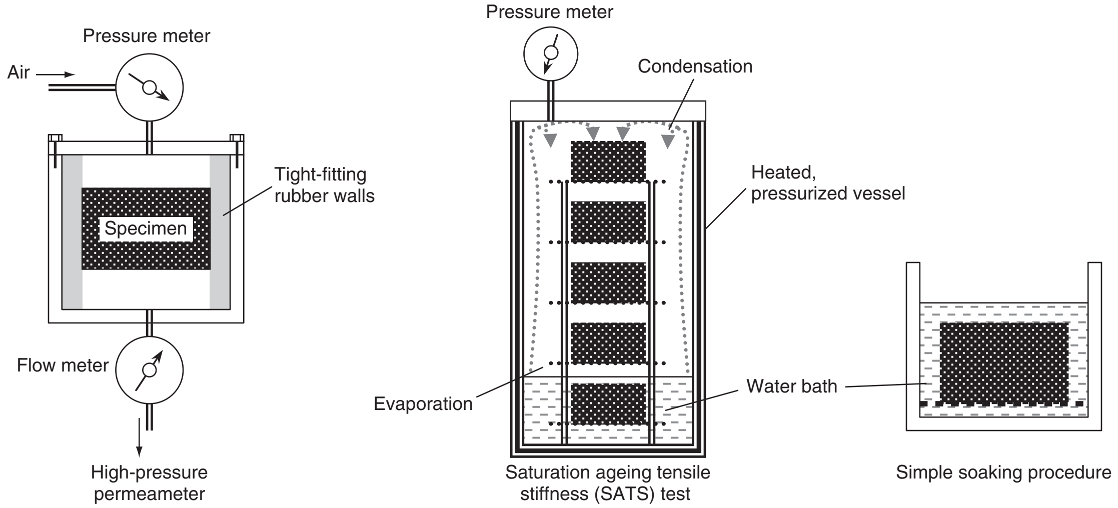

Figure 11 Approaches to water-susceptibility testing. The saturation ageing tensile stiffness (SATS) test (Collop *et al*., 2004; Highways Agency, 2007) allows simultaneous application of water and accelerated ageing due to a raised temperature and pressurised air

the mechanisms at work are as follows. Water expands to form ice so, as in concrete, this immediately causes pressures within the matrix of an asphalt, resulting in local fracture at aggregate particle contacts. The interaction between water-filled and ice-filled voids during freezing and melting may also give rise to unusually high dynamic pressures under traffic loading. The fact that the bitumen is most brittle (very small strains required to cause fracture) when it is cold exacerbates the problem.

The secret of protection against frost is more or less the same as that of protection against water. If the asphalt is dense ($\leqslant 5\%$ voids), water will not easily penetrate to allow ice formation. This is further confirmation that it is highly undesirable to allow water to penetrate and accumulate in an asphalt.

Biodegradation

It is known that certain bacteria can digest bitumen, see for example Traxler (1962) and Pendrys (1989). However, our current level of understating of these biological agents and their significance to the long-term durability of asphalt is in its infancy.

Summary

The following are some of the more important points discussed in this section:

- Bitumen is a visco-elastic liquid, even at in-service temperatures. It can ‘flow’ and permanent deformation of asphalt can therefore occur.
- The viscosity and elastic stiffness of bitumen are temperature and loading rate dependent, which means this is also true of an asphalt mixture.
- Most asphalt for pavement applications forms a skeleton of interlocking aggregate particles. Inter-particle movement is restricted by the bitumen surrounding each particle contact. Performance therefore depends on the effectiveness of the aggregate skeleton, the properties of the bitumen, and the strength of bitumen–aggregate adhesion.
- In every mixture there is a critical particle size, above which the particles form the skeleton and below which they act as binder extenders. In practice this means that filler should be seen as a binder additive, increasing both the stiffness and the fracture resistance of the mixture.
- In order to get the best out of an asphalt mixture, it is important to achieve optimum density and stiffness, as well as enough binder for fatigue and durability purposes and enough air voids to avoid ‘pore pressure’ and permanent deformation.
- It is highly undesirable to allow water to penetrate and accumulate in an asphalt.

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Chapter 26

Asphalt mixture design

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CONTENTS

Asphalt mixture design can be considered as a three-stage process. The first stage determines the choice of aggregate type and aggregate particle size distribution; in the second stage the type of bitumen is chosen based on the traffic loading and climate conditions of the pavement. In the final stage, the design bitumen content at which the mixture will fulfil the engineering properties in relation to the desired behaviour in service is chosen based on a combination of performance-related tests and theoretical volumetric calculations.

Introduction

As an engineering material, asphalt is typically designed to provide stiffness and bearing capacity, and resist repeated loading as experienced by a road under traffic. The effect of repeated loading manifests itself in two ways: permanent deformation, commonly referred to as 'rutting', and cracking. Additionally, asphalt mixtures are designed to minimise the effects of water on the system by either making the mixture dense and relatively impenetrable to moisture, for example in the case of *asphaltic concrete*, or by adding sufficient bitumen to the mixture to provide a thick coating of bitumen on the aggregates (referred to as *bitumen film thickness*) as in the case of *porous asphalt* and *thin surface course mixtures*.

Surface course materials must also fulfil requirements such as providing a skid-resistant surface under all weather conditions, an acceptable level of rolling resistance and provide a surface which, under trafficking, produces an acceptable level of tyre/road noise and acceptable ride quality.

The objective of asphalt mixture design is to produce an economical material, making full use of local resources, that meets the engineering requirements, i.e. to design for optimal service and economic performance, not necessarily the maximum achievable performance. Asphalt mixture design has taken on increasing importance over the last two decades in order to meet the current demands placed on roads today caused by increasing traffic, larger freight loads and climate change.

Historical perspective of asphalt mixture design

Recipe mixture specifications define an asphalt mixture in terms of the aggregate grading, mixture composition (a target range of bitumen content) and the method by which the mixture should be manufactured, placed and compacted. Recipe-type specifications are simple to use and are applicable to all types of asphalt mixtures. They are based on experience of known compositions which have performed successfully in practice. The supplier usually has little difficulty making the mixture to the required composition and it is relatively easy to check compliance with specification. As a result the recipe approach to mixture design is still frequently used as the basis of specifications, particularly in the UK.

Prior to 1945, the main requirements of a road surface were, in general, that it should be low-cost and durable. At this time the majority of county roads and many trunk roads were surfaced with '*Macadams*', which proved to be less and less durable in the interwar years as traffic increased. Highway engineers recognised that improved durability would be achieved using dense, impermeable mixtures. The gradual increase of the use of (BS 594: 1935) *hot rolled asphalt* for trunk roads and the much later development of *dense bitumen macadam*, which first appeared in the 1961 edition of BS 1621, reflected this growing awareness.

During the early 1970s it was realised that these dense recipe mixtures, which had up to this point given long and satisfactory performance, were deforming under increasingly demanding traffic loading. Thus, recipe formulations were found to have a number of limitations in high-stress situations. Today, recipe mixtures, while widespread in less onerous situations, are no longer used in high-stress, high-trafficked areas such as experienced by much of the UK major road network.

Whereas in recipe specifications the bitumen content is selected solely on the basis of experience, in *design mixture* methods the choice of bitumen content is based in whole or in part upon the results obtained in one or other laboratory test procedures. As early as the late nineteenth century, American engineers were starting to develop simple

Mixture type	Coarse aggregate PSD type/% >2 mm particle size	Bitumen content by mass: %	Filler content by mass: %	Air content: %
Asphaltic concrete	Continuous, 65%	4–5	2–10	3–6
Stone mastic asphalt	Discontinuous, 75%	5–6	6–8	2–6
Hot-rolled asphalt	Discontinuous, 35–50%	7–8	10–12	2–6
Porous asphalt	Single size, 90%	4–5	2–4	18–25

Table 1 The main types of asphalt mixtures used in road pavements and a summary of their components

design methodologies. Francis Hveem derived the first fully developed mixture design methodology in California in the late 1920s–1930s (Hveem, 1936) and a version of the methodology is still used today in some western US states.

Arguably the best-known mixture design procedure 'The Marshall Design Method' was developed at the Mississippi Highway Department around 1939 by Bruce Marshall. The main core of the Marshall procedure is to arrive at the optimum design binder content by considering key asphalt mixture properties such as density and stability. Later, in 1943, the US Army Corps of Engineers developed a simple portable asphalt design apparatus, to produce asphalt (based on the Marshall Design Method) that could perform well under the increasing aircraft wheel loads experienced by runways during the Second World War.

There followed a widespread adoption of the Marshall method of design and this type of approach formed the basis of the British Standard method for hot-rolled asphalt mixture design (BS 598 Part 107: 2004) and other methods, such as that adopted by the civil and military airfields in the UK (Defence Estates, 2005).

Since the 1990s, the Superpave$^{\text{TM}}$ mixture design method (National Research Council, 1994) has risen to prominence in North America, although this has not been adopted in Europe. The Superpave mixture design method is based upon a fundamental evaluation of the bitumen, and is directly linked to climate and traffic loading conditions of the site. The bitumen tests are chosen to measure performance at a given temperature relating to the climate, and the mixture design itself assesses workability in addition to the finished engineering properties of the mixture.

Today, proprietary thin surfacings, and to a lesser extent proprietary base and binder course mixtures, are established in the UK market and the choice of mixture design methodology, which is not mandated in the UK, is largely the choice of the asphalt supplier. A mixture of techniques will be used including Marshall Design, French Mixture Design Method (Delorme, 2006) and tests outlined in the Specification for Highway Works Clause 929 (Highways Agency, 2008a). Crucially, performance-related tests are commonly used to assess rutting potential, stiffness, fatigue and water sensitivity of the designed mixtures and typically today less emphasis is placed on the exact derivation of target binder content based on mixture volumetrics.

General principles of asphalt mixture design

In general, there are three stages to the design of asphalt mixtures: (i) the choice of aggregate type and aggregate grading; (ii) the choice of an appropriate bitumen grade; and (iii) the determination of a bitumen content that will optimise the engineering properties in relation to the desired behaviour in service.

The principal ingredient of an asphalt mixture is the aggregate and the basic character (and naming) of asphalt mixtures is formed by the particle size distribution of the aggregates (see **Table 1**). It is the aggregate grading that controls the basic stiffness and resistance to internal movement of an asphalt layer thus, for dense mixtures in particular, a fundamental principle for the design is that the aggregate fractions achieve maximum packing and an aggregate structure which is *fully interlocked*.

The maximum density-grading curve for the aggregates can be determined using a Fuller curve, which is of the form:

$$p = 100(d/D)^{0.45}$$

where

$D =$ maximum size of aggregate
$d =$ size of aggregate in question
$P =$ percentage of aggregate in question

Alternatively, for example, when a porous asphalt layer is required, an aggregate particle size distribution is adopted which allows for voids to be present in the mixture when fully compacted in order to allow water to pass through the layer.

Thus, the design of an asphalt mixture is initially based on the blending of aggregate fractions to form the desired particle size distribution. Several aggregate fraction sizes are available to construct an aggregate grading. Typically, aggregates greater than 4 mm in size are referred to as *coarse aggregates*, natural sands and crushed rock fines

Figure 1 Polished stone value apparatus (courtesy of Lafarge Aggregates Ltd)

smaller than 4 mm in size are referred to as *fine aggregates*, and *filler* and *fines* are the terms used for the aggregate fraction with a particle size less than 63 microns.

The types of aggregate used depend on several factors including availability at the asphalt plant, *polished stone value* (if used in surfacing layers) and thickness of the layer to be produced. The requirements for aggregates and the test methods used to characterise aggregates used in asphalt have been standardised in Europe (BS EN 13043: 2002). Aggregates commonly used in asphalt include limestone, granites, amphibolites, diorites, basalt and gneiss. Additionally, recycled aggregates such as crushed glass and secondary aggregates, such as slag from iron or steel production, are also commonly used.

Typically, all aggregates need to possess several basic properties such as hardness, toughness, a good affinity to bitumen, resistance to abrasion, and resistance to polishing for surface course aggregates to provide skidding resistance (**Figure 1**). Additionally, durability with regard to frost action and de-icing salts used on road surfaces is also required.

The addition of bitumen provides enhanced properties in the mixture and allows the aggregate structure to be formed under compaction. Bitumen provides cohesion and enhanced stiffness to an aggregate structure. The bituminous binder is contained within the interlocked aggregate structure and in most cases discrete air voids remain within the interlocked aggregate and bitumen system. Overfilling the aggregate structure can cause undesirable effects such as instability under traffic loading, so typically a minimum volume of residual air is required to remain within an asphalt mixture to ensure stability.

The selection of bitumen type results from the loading conditions of the pavement in which the asphalt is to be used, for example the type and number of loadings expected within the design life, and the climate in which the road is to be built. Traditionally, the bitumen properties considered include simple empirical tests such as needle penetration (BS EN 12697-40: 2000) (**Figure 2**), softening point (BS EN 1427: 2000) but increasingly the fundamental properties of the bitumen are examined using equipment such as a dynamic shear rheometer (BS EN 14770: 2006) (**Figure 3**).

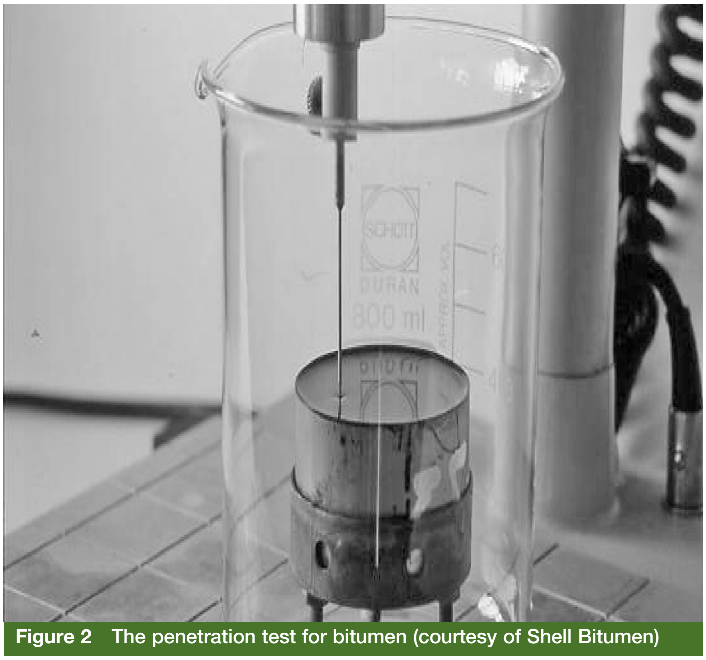

Figure 2 The penetration test for bitumen (courtesy of Shell Bitumen)

After the target aggregate particle size distribution and binder type have been chosen, the mixture design process moves on to deciding the correct quantity of bitumen to add to arrive at a mixture which fulfils the design requirements. If too much bitumen is added, gradual secondary compaction by traffic will result and excess bitumen will be squeezed to the surface, making it slippery. It may also lead to hydrostatic pressure in the body of the mixture, reducing the stability of the mixture. If insufficient bitumen is added then water and air will enter the mixture, resulting in hardening of the bitumen and possibly stripping.

The typical methodology employed is to mix asphalt in the laboratory (**Figure 4**) and manufacture several test specimens over a range of bitumen contents and measure properties of the mixtures at each bitumen content to arrive at the best performing, most economic mixture. Typically this evaluation consists of two elements: assessing the *mixture volumetrics* and/or through measurement of pertinent properties of the mixtures at (or around) the target binder content. The packing characteristics of the aggregates significantly influence the mixture properties which cannot be assessed from grading curves, as these assume uniformity of particle shape, surface roughness and specific gravity. Evaluation of compacted specimens takes into account these aggregate factors.

Figure 3 A dynamic shear rheometer (courtesy of Lafarge Aggregates Ltd)

Figure 4 Laboratory production of asphalt mixture (courtesy of Shell Bitumen)

In order to design an asphalt mixture using laboratory procedures, with the aim of producing the required density and engineering properties, it is desirable to prepare specimens which reproduce, as closely as possible, the material that will be used in practice. Ideally, specimens should be compacted to the same thickness as used in practice to a density similar to that attained under traffic during the service life. The compaction of the specimens in the laboratory can have a significant effect on the end-performance of the mixtures manufactured in the laboratory and the method of compaction is an important consideration in the mixture design procedure. Several methods of compaction (BS EN 12697-33: 2003; BS EN 12697-30: 2004; BS EN 12697-31: 2004) have been developed including *Marshall compaction* using a standard hammer (**Figure 5**), *gyratory compaction* where the asphalt is compacted using a kneading motion (**Figure 6**) and *roller compaction* (**Figure 7**) where samples are compacted in a mould using a method which closely simulates the conditions in real life where rollers are used to compact road surfaces.

While the Marshall test apparatus has been the most widespread in use to assess the mechanical properties of an asphalt mixture, other test apparatus and procedures

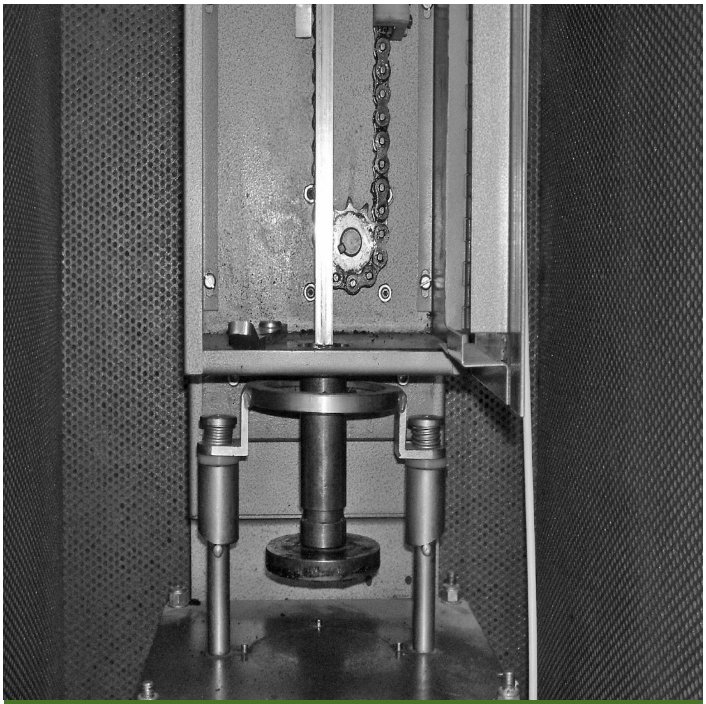

Figure 5 Standard Marshall compaction apparatus (courtesy of Shell Bitumen)

Figure 6 Gyratory compactors (courtesy of Shell Bitumen)

have also been used including various methods involving the use of triaxial or creep test apparatus. Since the 1990s the Nottingham asphalt tester (NAT) (**Figure 8**) has been used in the UK to measure stiffness and fatigue characteristics of asphalt mixtures in the UK (BS 598 Part 107: 2004). The use of the NAT is specified for base and binder course layers in the UK in the Specification for Highways Works Clause 929 (Highways Agency, 2008a).

For practical reasons, proportions by mass are typically used to specify a mixture, however the mechanical properties of the mixture are strongly dependent on the relative volumetric proportions. Hence, conversions from mass to volume and vice versa are frequently needed and a sound grasp of the important mixture parameters is required. The volume and mass components of a unit volume are shown below in (**Figure 9**) where:

G_b = specific gravity of binder

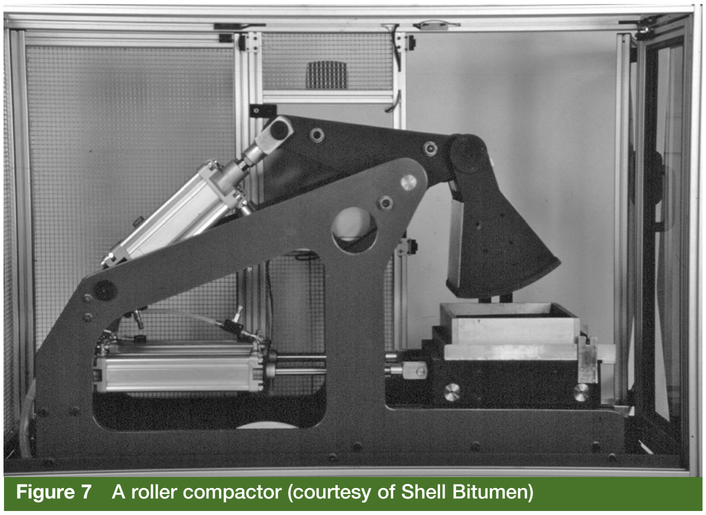

Figure 7 A roller compactor (courtesy of Shell Bitumen)

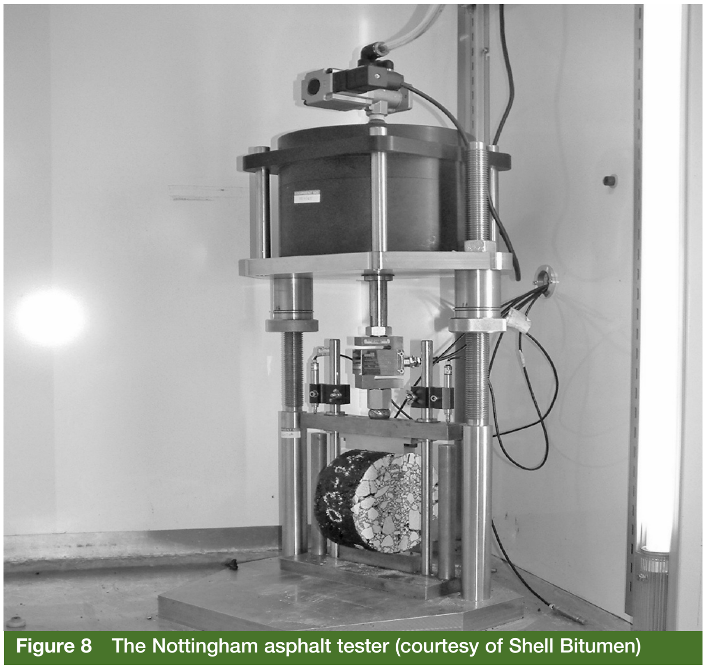

Figure 8 The Nottingham asphalt tester (courtesy of Shell Bitumen)

G_a = specific gravity of mixed aggregate

M_B = binder content per cent by mass of total mixture

M_A = aggregate content per cent by mass of total mixture

$M_A + M_B = 100\%$

M_b = mass of binder (kg)

M_a = mass of aggregate (kg)

γ_m = density of compacted mixture (kg/m^3)

γ_w = density of water (1000 kg/m^3)

V_a = volume of aggregate (m^3)

V_b = volume of binder (m^3)

V_v = volume of voids (m^3)

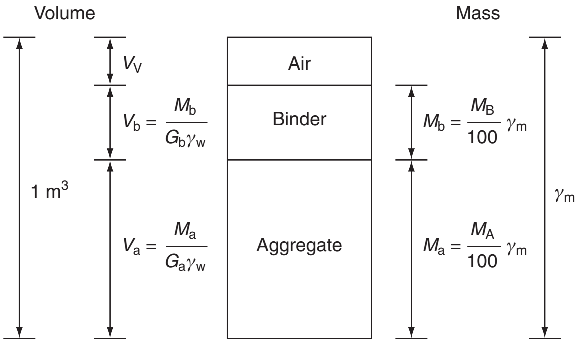

Figure 9 Volumetrics of an asphalt mixture (courtesy of Shell Bitumen)

$V_a + V_b + V_v = 1\,\mathrm{m}^3$

V_A = volume of aggregate as %

V_B = volume of binder as %

V_V = volume of air voids as %

Note that upper case subscripts imply quantities expressed as percentages.

The general approach taken to the selection of final design bitumen content has been to average the optimum bitumen contents giving peak values for certain mechanical properties together with bitumen contents giving other desired properties relating to volumetrics. Although Bruce Marshall in his original method selected the design bitumen content as that giving the peak on the unit weight versus bitumen content curve, checking other properties at this bitumen content against desired standards, the customary practice in subsequent methods based on the Marshall method has been to follow the averaging approach mentioned above.

Design methodologies

The most widely used procedure for designing continuously graded mixtures is the *Asphalt Institute Method* (Asphalt Institute, 1997). Both the Hveem and Marshall approaches are outline by the Asphalt Institute, however the Hveem method has not been adopted outside of some US states and is not discussed further in this section. The basis of the *Marshall Method* is still used today in the UK in the design of asphaltic concretes for runways, and is the underlying basis of BS 598: Part 107: 2004, the design method for hot-rolled asphalt.

The first stage in the Marshall Design procedure is to dry and grade the aggregates and blend them to achieve the desired particle size distribution. Asphalt mixtures are then produced in the laboratory at increments of 0.5% bitumen by mass on each side of the estimated optimum bitumen content. Compaction of the mixtures is carried out with the standard hammer using 35, 50 or 75 blows on each end of the specimen, according to the anticipated traffic levels.

At each bitumen content the following mixture properties are determined (based on the average of the four specimens):

- the bulk specific gravity of the compacted mixture;
- the bulk specific gravity of the compacted aggregate;
- the theoretical maximum specific gravity of the mixture;
- the voids in the mineral aggregate;
- the theoretical air voids in the mixture.

The specimens are then tested in the *Marshall apparatus* and the average *Marshall stability* and *flow* values are determined at each bitumen content. The three bitumen contents corresponding to maximum stability, maximum specific gravity and the desired value of void content are recorded, and the optimum bitumen content is taken as the average of these three values. The flow value and voids in mixed aggregate that correspond to the optimum bitumen content are determined and compared with permissible values.

For asphalt mixtures designed using the Superpave™ approach, the particle size distribution uses a 0.45 power relationship to produce the maximum packing of the aggregate. In addition, this methodology introduces a *restricted zone* which forces the grading of the mineral aggregate to deviate from the general 0.45 power line. The role of the ‘restricted zone’ is to increase particle-to-particle contacts with the aggregate which is considered to reduce rutting in the finished asphalt.

A *performance grading* (PG) system for bitumen has been developed in the US, and the bitumen is evaluated for performance based on the criteria of temperature, time of loading and aging factors. In this methodology, air temperatures are related to pavement temperatures and the properties of the bitumen before and after artificial ageing are measured and examined at the appropriate high and low temperatures relating to the climate.

In the Superpave method, the asphalt mixture is evaluated for performance under various loading and environmental field conditions. The asphalt mixture is also evaluated by calculating the *voids in mineral aggregate* (VMA) and the *voids filled with asphalt* (VFA) at a target value of 4% air voids (V_a). Many states set target values for VMA and VFA at the 4% air voids level.

Although the principle of performance grading of bitumens and pavement design based on climatic conditions have been adopted in other countries outside of the US (for example Canada), the Superpave mixture design method is used to design dense graded mixtures with limited macrotexture, and thus the procedure is not ideally suited to the design of many UK surface course mixtures.

Design of surface course mixtures in the UK

Hot-rolled asphalt (HRA) has been used for many years for surfacing major roads in the UK and traditionally these mixtures have been specified by recipe. In addition to the recipe specification, BS 594: 1973 introduced the option of using the Marshall methodology to estimate the bitumen content required to optimise the properties of the fine aggregate:filler mixture, known as the *mortar design method*. The fine aggregate and filler are blended in the ratio 6:1 by weight and Marshall specimens are prepared and tested over a range of bitumen contents, usually 8.0% to 12.0% at 0.5% increments. From the test results, three graphs are plotted – stability, compacted density of the mixture and compacted density of the aggregate – each against bitumen content. The three bitumen contents corresponding to the maximum values are determined and the mean of these three values is taken as the optimum bitumen content for that blend. The sand and filler are then used with added stone to comply with one of the standard formulations; the bitumen content for the mixture with the added stone is calculated using a mathematical formula. Limits are placed on the minimum bitumen content allowable.

BS 598: Part 107: 1985 provided an alternative for designing the mixture composition; this time the total mixture including the coarse aggregate is evaluated. Briefly, the method consists of testing three specimens at each bitumen content; a minimum of nine bitumen contents are examined at 0.5% increments. From the test results four graphs are plotted – stability, flow, mixture density and compacted aggregate density – each against bitumen content. The three bitumen contents corresponding to the maximum values of stability, mixture density and compacted aggregate density are determined and the mean of these three values is termed the optimum bitumen content. An empirical factor is added to the optimum bitumen content to determine the ‘design’ bitumen content.

The bitumen content specified for the final mixture composition is termed the target bitumen content. The target bitumen content is the design bitumen content plus (if deemed necessary) an unspecified quantity of bitumen to ‘allow for such properties as workability and durability’. This extra bitumen typically ranges from 0% to 1.0% and is 0.7% for a mixture containing 30% coarse aggregate. Thus, the actual target bitumen content may be as much as 1.7% above the optimum bitumen content determined by the Marshall design procedure.

In 1996, the Highways Agency introduced Clause 943 High Performance Hot Rolled Asphalt in which the British Standard wheeltracking device (**Figure 10**) was introduced as an additional performance test for HRA.

Continuously graded asphaltic concrete mixtures are normally used for airfield pavements and, in common with HRA, are designed using a variation of the Asphalt Institute Method. The *Defence Estates Method* which is used in the UK differs in several minor but significant respects from the BS 598 method outlined previously. In this method the maximum theoretical density is calculated and instead of voids in mixed aggregate, voids filled with bitumen are determined. The other significant difference is that the bitumen absorption is estimated using the apparent relative density, the relative density on an oven-dried basis and the water absorption value of the aggregate. If the water absorption value of the aggregate is less than 1%, all the calculations are made using the apparent relative density, i.e. assuming that the bitumen absorption is equal to the water absorption. If the water absorption is greater than 1%, the average of the apparent relative density and the relative density on an oven-dried basis is used, i.e. assuming that the bitumen absorption is only half of the water absorption.

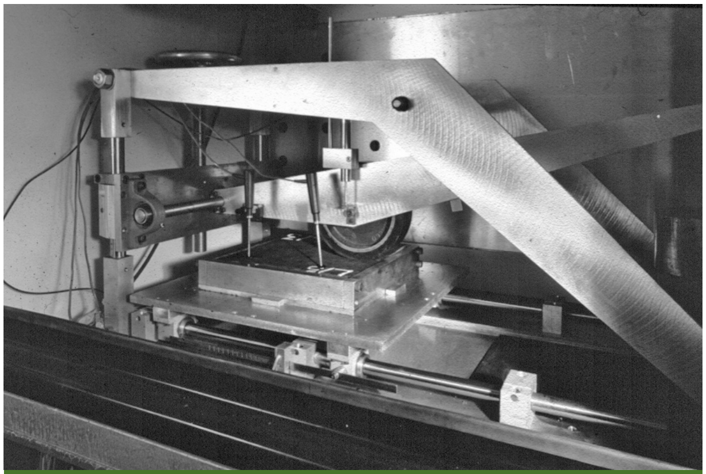

Figure 10 British Standard wheeltracking device (courtesy of Shell Bitumen)

Five graphs are plotted: stability, flow, maximum apparent relative density, voids in the mixture and voids filled with bitumen. From these graphs the four bitumen contents corresponding to maximum stability, maximum apparent relative density and the desired values of voids in the mixture and voids filled with bitumen are determined. The optimum bitumen content is taken as the average of these four values. The flow value that corresponds to this optimum value is determined from the graph and compared with the permissible values.

Since the 1990s, the types of surface course mixtures used in the UK have changed considerably with the introduction of *stone mastic asphalt* from Germany, *Béton Bitumineux Très Mince (BBTM)* from France, referred to as *thin surfacing* in the UK.

Stone mastic asphalt (SMA) is a surface course mixture originally developed in Germany to resist studded tyres. Despite a subsequent ban on the use of these tyres in 1975, stone mastic asphalt has continued to be used because of its superior performance when compared to the asphaltic concrete surfacing. Stone mastic asphalt essentially consists of discrete and almost single-sized aggregate particles forming a skeletal structure bonded together by bituminous mastic. At the bottom, and in the bulk of the layer, the voids in the coarse aggregate are almost entirely filled by mastic, while at the surface the voids are only partially filled resulting in an open surface texture. Provided texture is maintained, this provides good skidding resistance at all speeds and facilitates the drainage of surface water.

In the UK, TRL Project Report 65 (Nunn, 1994) outlines a basic mixture design methodology for SMA-type mixtures and is based principally upon the measurement of air void content and bitumen drainage (the ability of a rich asphalt mixture to retain its bitumen content during mixing, transportation and placement). A target air void content of between 2% and 4% is typical and either cellulose fibres and/or polymer-modified bitumen are used to inhibit binder drainage in SMA mixtures.

A design methodology for thin surfacings in the UK is not specified and several approaches to the mixture design can be taken. Typically, either the French LCPC Design Methodology is used or an extended TRL PR 65 approach combining the measurement of air void content and binder drainage properties with some performance-type tests such as wheeltrack testing and measurement of the water sensitivity of the mixtures.

Porous asphalt (PA) is designed to allow water to pass through its structure and hence consists of an aggregate skeleton with a high content of air voids. The aim of the mixture design is to maximise bitumen content to ensure durability of the mixture in the presence of water and air, while maintaining a high void content and low binder drainage. The target binder content determined in the binder drainage test (BS EN 12697-18: 2004) represents the maximum quantity of binder that can be safely incorporated into PA without introducing excessive binder drainage. Additionally, a relative hydraulic conductivity test (British Standards Institution, 2000) is also used on the finished mixture to ensure that sufficient interconnected voids are present to allow water to quickly pass through the material.

Design of base and binder courses in the UK

Traditionally, *road base macadam* mixtures have followed recipe specifications, which have generally resulted in materials of satisfactory performance. These recipes were empirically developed specifications, founded on the principles set by the likes of Telford and Macadam and were finally detailed in the BS 4987 (2003) series covering coated macadams. Although there was recognition that mixture composition and compaction were key features pertaining to the performance of these materials, the only checks carried out on laid materials were compositional analyses, and later, *percentage refusal density* measurements (a compaction requirement).

The shortcomings of this approach were highlighted in the early 1990s when it was observed on a site where a new bypass was being built, that construction traffic had deformed the newly laid road base. Investigations showed that although the material complied with both the recipe and compaction specifications, under the high state of compaction which had been achieved on site, the material did not contain any air voids. Hence, under loading, the material had exhibited early plastic flow because the aggregate skeleton was flooded with bitumen.

The simple way of addressing this problem was to measure the void content of the mixture at the refusal density and to set a minimum level of air voids to prevent a state of over-compaction occurring. Hence road-base mixtures began to be specified using a combination of minimum and maximum allowable air void contents and limitations on the minimum binder volume. This approach was detailed in *Clause 929 of the Specification for Highway Works* (Highways Agency, 2008a) and subsequently this clause has been further developed to include several requirements including measurement of stiffness, resistance to permanent deformation and sensitivity of the mixtures to water ingress. Although Clause 929 does not represent a true mixture design method, it does highlight the relevance of the volumetric composition of mixtures and the measurement of end performance of asphalt mixtures.

EME-2 (Enrobe A Module Elevé) asphalt is a base layer material recently introduced into the UK from France, (Sanders and Nunn, 2005). This material can form the base and binder courses of a pavement and is characterised by a high bitumen content of a hard bitumen binder combined with a carefully graded aggregate which provides a very strong asphalt with high stiffness and a high resistance to deformation. The high content of binder produces durability in terms of resistance to fatigue and cracking and it also means major benefits of workability and laying quality.

EME-2 in the UK is designed in line with the French method using the same equipment as used in France. In the early 1970s, France's Laboratoire Centrale de Ponts et Chaussées (LCPC) replaced the Marshall method of mixture design with a new *LCPC method*. The LCPC mixture design standardises the compaction effort on the road. The concept of standardising compaction effort on the road is the reverse of the Marshall approach that standardises compaction in the laboratory. In addition, the French design method makes a significant performance evaluation of the mixture using an iterative approach.

Future trends

Proprietary mixtures complying with formal approval schemes are becoming the norm in the UK for major highway contracts. As a formal mixture design methodology in the UK is not mandated, the route by which the mixtures have been designed is not always formally declared but in general the emphasis is placed on the end-performance of the mixtures. Increasingly complex end-performance tests are today used to measure the properties of asphalt mixtures.

Additionally, environmental awareness has grown significantly in recent times and adaptations to asphalt mixture design to include high quantities of recycled asphalt pavement (RAP) have been investigated and reported on (Nicholls *et al*., 2008).

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Chapter 27

Production of asphalts

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This chapter considers the types of rock used in manufacturing asphalts in the UK and examines the methods used to quarry these materials. Different asphalt production methods are then discussed and their advantages and disadvantages summarised.

CONTENTS

Raw materials

The raw materials, i.e. coarse aggregate used in asphalt production, commonly known as roadstone, must be hard, clean and durable. The term aggregate is defined in BS 6100-9:2007 as 'granular material, either processed from natural materials such as rock, gravel or sand, or manufactured such as slag'. The UK has an adequate supply of aggregates although some materials having particular properties such as high polished stone value (PSV) are becoming increasingly rare.

The roadstone industry uses crushed rocks, sand and gravel as the main ingredients for the construction of flexible pavements. It is also possible to use blastfurnace slag or steel slag but the substantial decline in the steel industry has severely limited the availability of slag.

Crushed rocks

Crushed rocks are classified according to their geological origin and these are termed igneous, sedimentary and metamorphic.

Igneous

Igneous rocks are those formed from a hot liquid, or magma, cooling either above, or beneath, the Earth's surface. These rocks have a crystalline structure. Examples include basalt, diorite, dolerite, gabbro, granite.

Sedimentary

The action of ice, wind or water on the rock's surface, or organic remains, results in the formation of smaller particles. These particles are deposited on the beds of lakes or rivers, the sea floor or depressions in the Earth's surface. This sediment accumulates, becomes buried and compaction together with cementation over the years removes the air and water to form sedimentary rocks that vary considerably in hardness. Examples include limestone and gritstone (a hard form of sandstone).

Metamorphic

Heat, pressure and/or chemical substances deep beneath the surface of the Earth act on existing igneous and sedimentary rocks to form a new rock type described as metamorphic. Examples include gneiss, hornfels, marble, quartzite and slate.

Sand and gravel

Sand and gravel are the products of mechanical and chemical weathering of rocks such as granite and sandstone and are, therefore, a type of sedimentary deposit. Weathering by heat, ice, wind and moving water transports the rock fragments; this movement results in further mechanical working and transforms the sand and gravel into rounded and smooth irregular shapes. The deposits are found along the seashore, along river valleys and on the sea floor.

Gravels are usually classified by their major rock type, e.g. flint gravel. Gravels can also contain a variety of several different rocks.

Blastfurnace slag and steel slag

Although limited in availability, BS EN 13043:2002 allows the use of these materials described in the standard as 'manufactured aggregate' in asphalts. These materials, like all aggregates in an asphalt, must comply with requirements which for the UK are set out in PD 6682-2: 2003.

Resources

The UK has a wide variety of quality aggregates with specific and different characteristics. A simplified geological survey is shown in **Figure 1**. This indicates the distribution of the major rock formations throughout the UK. The map shows that the south-east of England is predominantly sand and gravel, clay and chalk with very little hardrock.

The south-east is a highly populated region creating considerable development and therefore has a need for

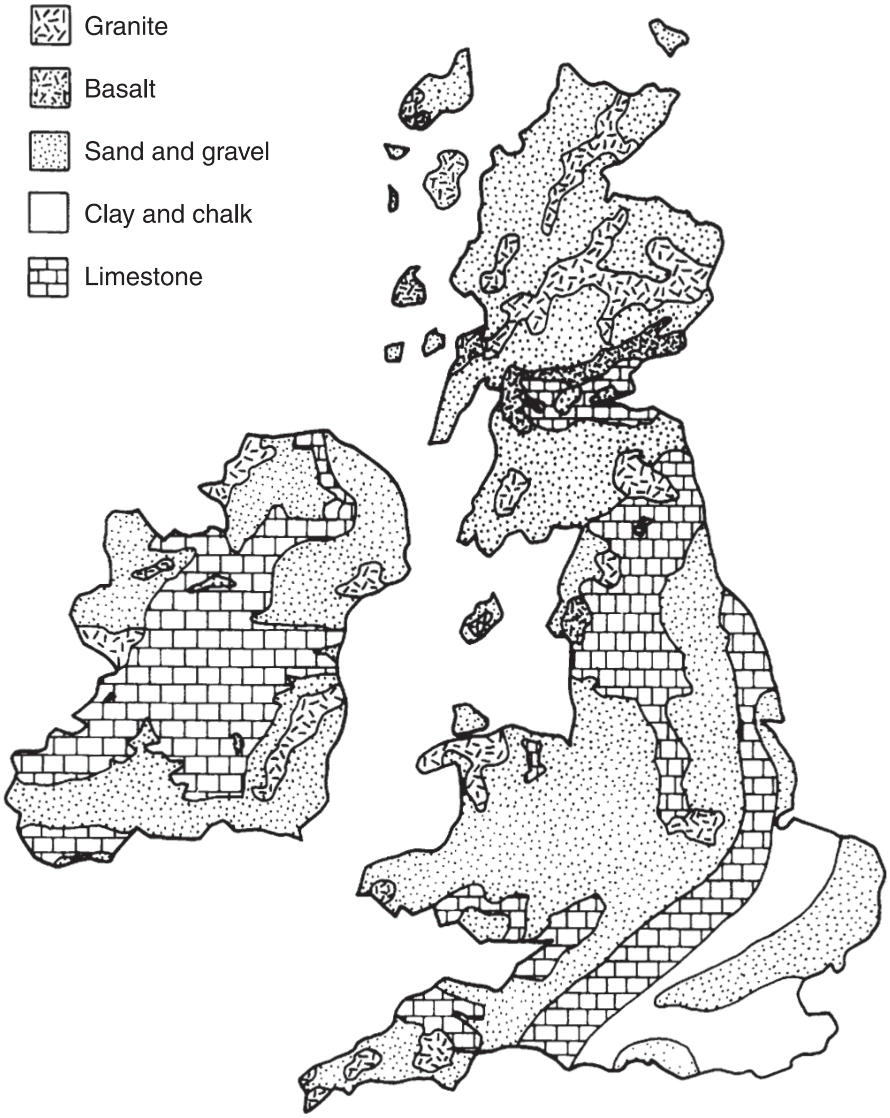

Figure 1 A simplified geological map of the UK showing major distribution of rock types used in the roadstone industry

large quantities of quality materials. Although some roadstone production can be met from local resources, the largest percentage is transported from the south-west and to a lesser degree from the Midlands and by sea from Ireland and Scotland. The major suppliers of aggregates have developed sites containing large asphalt mixing plants with capacities from 400 t/h up to 800 t/h in and around the London area. These plants have wharf offloading or rail offloading and storage facilities and they can have aggregates delivered in loads of up to 5000 t capacity by rail as well as smaller deliveries by truck.

In the UK as a whole, the majority of aggregate movement is by road transport since most mixing plants are local to the rock source. The advent of fewer but larger capacity asphalt mixing plants means that the use of both rail and sea transport is likely to increase.

Methods of producing aggregates

There are numerous types of rock making up the igneous, sedimentary and metamorphic classifications. Each rock has its own characteristics including resistance to fragmentation, surface abrasion, polishing and wear (PD 6691:2007). These inherent qualities together with local geological experience and engineers' expertise within the quarrying industry govern the type and size of equipment selected for an aggregate processing plant.

Plants manufacturing roadstone have to produce a wide range of sizes with restrictions on shape, the preference being cubical. They should have the ability to operate at varying capacities and be flexible enough to alter the range of product sizes when the need arises. Thus, the aim is to produce aggregates to the required sizes, with an

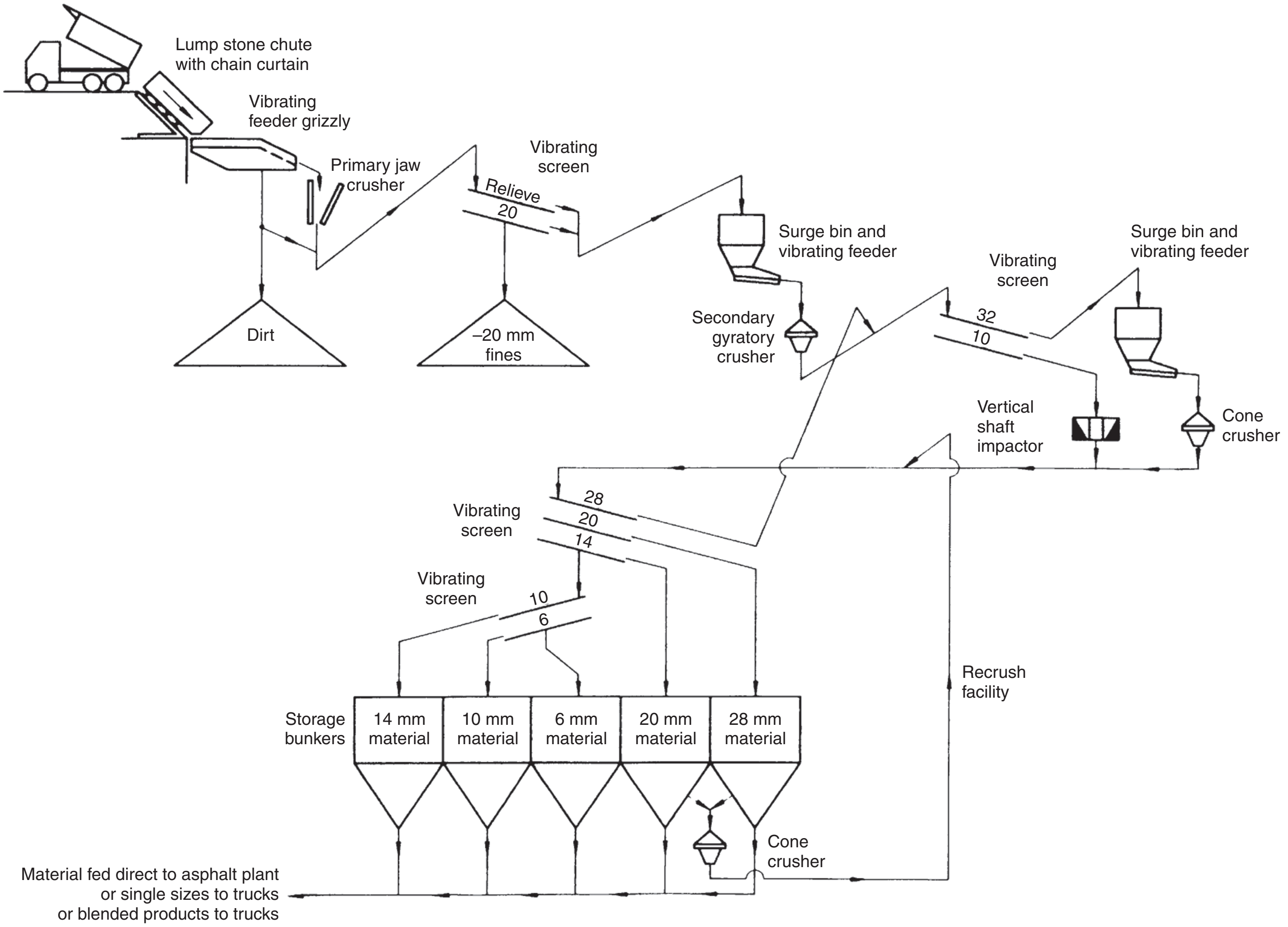

Figure 2 Flow diagram of a hardrock crushing and screening plant

optimum shape, at the capacity needed and in an economical manner. Crushed rocks used in roadstone production have a maximum Flakiness Index of 45 (50 for uncrushed gravel) whereas rolled in precoated chippings have a maximum allowed Flakiness Index of 25. The Flakiness Index is an indication of the shape of an aggregate: the lower the value the more cubical the product.

The stone is reduced in size by a crusher. There are four types of crusher: jaw, gyratory, impact and cone. To produce the required size and optimum shape, the rock is reduced in stages and plants can have one, two, three or more stages depending on the application. The shape is controlled by the type of crusher employed, the reduction ratio, the crusher setting, the rate of feed and the physical characteristics of the stone itself. The best cubical shape is achieved at, or around, the crusher setting and with a choke feed on the cone, jaw and gyratory machines.

Plants are usually rated at an operational efficiency of 80% to allow for blockages within the crushers or breakdowns. Therefore, if 240 t/h of material are required, the plant would have to be rated at a minimum of 300 t/h in order to meet this demand.

Aggregate processing plants can be roughly divided into three categories: hardrock, limestone, and sand and gravel plants. The industry loosely uses the term limestone crushing and screening plants to describe plants for soft and medium hard limestones where the silica content is less than 5%. Limestones can be hard with much higher silica contents.

Hardrock crushing and screening plant

An example of the flow diagram for a hardrock crushing and screening operation is shown in **Figure 2**.

In preparation to process rock, the overburden, consisting of topsoil, subsoil, clay and small loose rocks, is removed and holes are drilled at predetermined distances for the blast charge. Blasting generally takes place daily; some of the larger quarries can blast up to 30 000 t or more at a time. Great care and skill are needed when blasting, not only to ensure that the blast itself is safe and contained but to produce a stable quarry face afterwards. Quarries tend to be deep and therefore, in the interests of safety, the face is worked in levels, or benches. Typically these benches can each be 15 m deep but this depends on the rock characteristics and the depth of the rock seam. Many workings go below the water table and, in these instances, a pumping system is used.

The maximum size feed material to the plant illustrated is an 800 mm cube. Cost-effective primary blasting means there will be a small percentage of rock which is too large for the primary crusher. Secondary breaking is the term used for reducing these oversize rocks. Examples of this process are further drilling and blasting, the drop ball, the hydraulic hammer, the impact breaker and the rock breaker. The blasted rock is loaded and hauled from the face to the plant by large dump trucks which can have a capacity of up to 100 t.

The rock is fed into a lump stone chute equipped with a chain curtain to control the flow. The material discharges onto a reciprocating tray feeder with a built-in grizzly section. A *grizzly* is a term used by crushing plant manufacturers to describe a vibrating grid with bars in one direction only. The grizzly bars are set at 50 mm spacing, allowing the natural fines to bypass the crusher or go to a dirt stockpile if the quality is poor. The 1200 mm × 1000 mm primary jaw crusher operates at 150 mm closed side setting and therefore produces a maximum size of approximately 230 mm. The crushed rock and the '*good*' natural fines are passed over a scalping screen where the material that is smaller than 20 mm (described as minus 20 mm) is removed.

Plant capacities in excess of 600 t/h use a primary gyratory crusher instead of a jaw crusher. In such applications, the gyratory crusher does not need a feed station since the dump trucks can tip directly into the machine. Although a good cubical shape is produced, the primary gyratory is relatively expensive and requires a large headroom for installation. However, at high capacities the cost per tonne of material crushed is lower than with other crushers. **Figure 3** shows material being fed into a primary gyratory crusher.

The plus 20 mm material from the scalping screen is transferred to a surge hopper. This facility enables the secondary crusher to have a controlled choke feed. An electric vibrating feeder delivers the material to a secondary gyratory crusher that operates at 40 mm closed side setting. A gyratory crusher is used here as a secondary machine to take advantage of the high reduction ratio available and the good product shape. The crushed material is conveyed to a two-deck vibrating screen.

The plus 32 mm material is fed to a surge bin, electric vibrating feeder and a tertiary cone crusher operating with a 19 mm closed side setting. A cone crusher with this type of non-segregated feed grading has a good reduction ratio and performs effectively with hard abrasive materials. It is economical and crushing results in a cubical product near to the closed side setting providing a choke feed is maintained.

Figure 3 Direct feed of stone into a gyrator crusher (courtesy of QMJ Publishing Ltd)

The minus 32 mm plus 10 mm aggregate is directed into a vertical shaft impactor that operates on the crushing principle of throwing stone by centrifugal force against captive stone. Vertical shaft impactors are useful third- or fourth-stage crushers and produce a good cubical shape. On occasion, these machines are used to perform a shaping duty on poor-quality materials.

The crushed materials, together with the minus 10 mm aggregate from the two-deck screen, are conveyed to the final sizing screens and the single-sized materials are stored in bunkers. The plus 28 mm material is recirculated to remove the oversize stone.

Most overseas crushing plants store the materials in ground stockpiles instead of bunkers and many transportable and mobile plants in the UK also adopt this approach. This layout reduces the cost considerably but can lead to contamination and it exposes the crushed aggregates to adverse weather conditions. A material with higher moisture content will be more expensive to dry at a later stage in the roadstone production process. Storage of the crushed products also enables a recrush facility to be more easily incorporated. A cone crusher fed from the 28 mm and 20 mm aggregate bins provides a more versatile arrangement should market requirements change.

It is not unusual for hardrock plants to have extensive storage facilities including larger and smaller-sized products. The materials from the bunkers can be fed

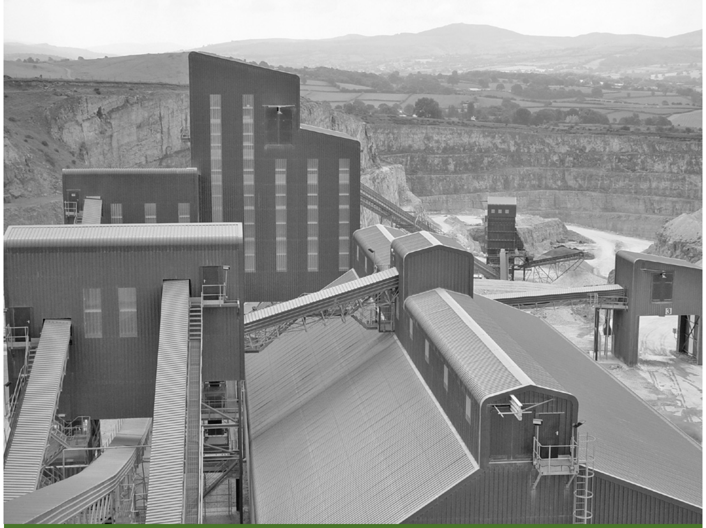

Figure 4 Limestone crushing and screening plant (courtesy of QMJ Publishing Ltd)

directly to an asphalt plant or single sizes or blended products to trucks.

Limestone crushing and screening plant

The extraction from the ground of a softer, less abrasive material such as limestone, with a low silica content, is almost identical to the hardrock process. The overburden is removed, the rock is blasted, secondary breaking takes place where necessary and the stones are transported from the quarry face to the plant in large dump trucks. A limestone crushing and screening plant is shown in **Figure 4**. In the flow diagram, shown in **Figure 5**, the trucks discharge into a lump stone chute. The largest feed size to the plant is a 900 mm cube and a chain curtain is used to control the flow. A vibrating feeder grizzly removes the minus 50 mm material to either bypass the crusher or feed to a dirt stockpile. The plus 50 mm material is discharged into a primary horizontal impact breaker.

The impact breaker works on the principle of reducing the material size by the impact of the rock against hammers which are attached to a high-speed rotor. The broken rock then strikes breaker bars located on the inside of the machine, resulting in more reduction and the stone falls again onto the revolving hammers for further reduction. The product size is determined by the position of the breaker bars relative to the rotor hammers and the rotor speed can be adjusted to cater for different feed materials.

A high reduction ratio can be achieved with this machine. A maximum feed size of 900 mm results in a product size of minus 100 mm and a good cubical shape. This machine can achieve the equivalent of two stages of crushing in a hardrock plant. The feed material must be relatively soft, not too abrasive and have a silica content of less than 5%. The crushed aggregates are conveyed to a surge bin which provides the remainder of the process with a controlled feed by eliminating the inconsistent dump truck discharges experienced at the primary crushing stage. An electrical vibrating feeder discharges the material from the surge bin to a conveyor and two-deck vibrating screen.

The plus 20 mm material is fed to a secondary horizontal impact breaker; the minus 20 mm bypasses the crusher. The secondary impactor operates on a similar principle to the primary impactor. However, the product size is better controlled and some machines incorporate grids at the discharge outlet. Although the reduction ratio is high compared with a jaw crusher, it is less than with the primary impactor and, again, a cubical shape is obtained throughout the product sizes. The crusher discharge and the bypass material are conveyed to the final sizing screens. Material sizes of 20 mm, 14 mm, 10 mm, 6 mm and 3 mm are stored in bunkers. The plus 20 mm is returned for further reduction.

As with the hardrock crushing and screening plants, a low-cost option is for the material to be stored in ground stockpiles but adverse weather can affect the finished products. Contamination can also result and there is the cost of additional drying at the asphalt plant.

A cone crusher is situated between the 20 mm and 14 mm bins to produce a recrush option should market needs change. This is not necessarily considered to be a further crushing stage but a design feature to make the plant's production more flexible.

As the setting of an impact breaker and the reduction ratio are not as significant in influencing the product shape with these low silica content materials, the process described only needs two crushing stages. If the feed material could be supplied cost-effectively at 100 mm down then only a single-stage crushing operation would be required.

Sand and gravel processing plants

Sand and gravel can be dredged from the sea or dug out of the ground. Marine-dredged gravels are only used for roadstone production when hardrock and limestone are in short supply or uneconomic. Care must be taken with marine gravels to ensure that the chlorides are removed before bitumen is added otherwise stripping of the bitumen will occur.

Land-based sand and gravel is found in two forms, a wet pit or a dry pit. A wet pit is where the majority of the deposit is under water; this material can be won by a suction dredger or a drag line excavator. In a dry pit, the sand and gravel can be worked using a loading shovel, a drag line excavator or a scraper together with a tractor. The water can be pumped out of a wet pit and the deposit worked

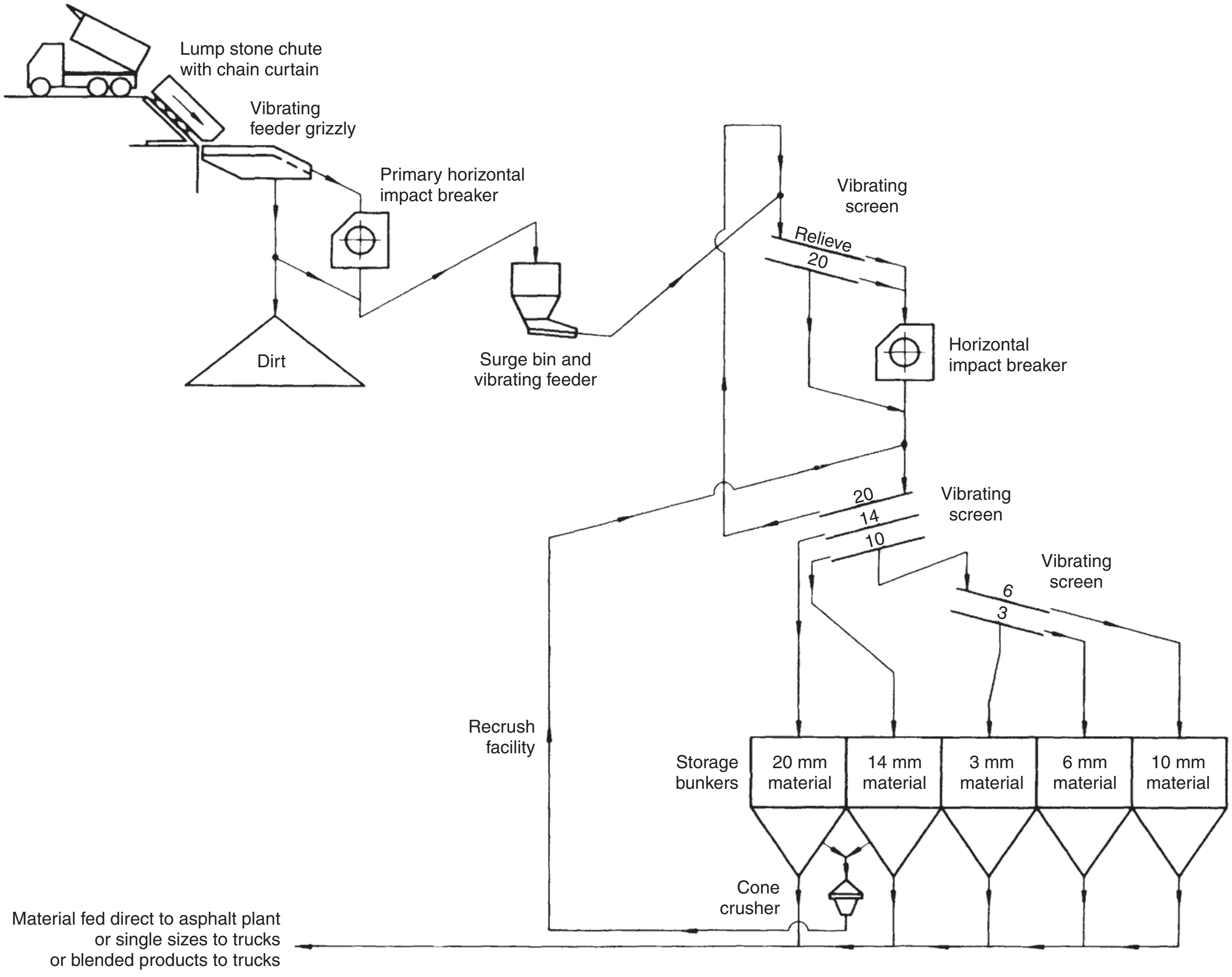

Figure 5 Flow diagram of a limestone crushing and screening plant

as a dry pit. Deposits can be up to, say, 8 m thick but they usually average about 5 m. The overburden is generally shallow, typically from 0–2 m thick.

Many boreholes are taken when excavating for sand and gravel and the samples are then tested in the laboratory. The ratio of sand to gravel is of prime interest along with the geological composition of the gravel since these factors govern the design of the processing plant. Dump trucks can be used to transport the sand and gravel from the workings to the plant or a field conveyor system can be installed. Such a plant is shown in **Figure 6**.

The sand and gravel processing plant flow diagram shown in **Figure 7** includes a field conveyor from the deposit to the plant. The as-dug material is conveyed to a vibrating grizzly with the bars set at 50 mm spacing. The plus 50 mm is stockpiled and the minus 50 mm material passes to a large surge hopper.

A fixed-speed belt feeder extracts the material from the hopper by way of a conveyor to a de-sanding screen equipped with water sprays. The aggregate is discharged into a rotating washer barrel where water is added. The washed aggregate is then passed over a reject vibrating screen, again fitted with water sprays.

Figure 6 Sand and gravel plant (courtesy of QMJ Publishing Ltd)

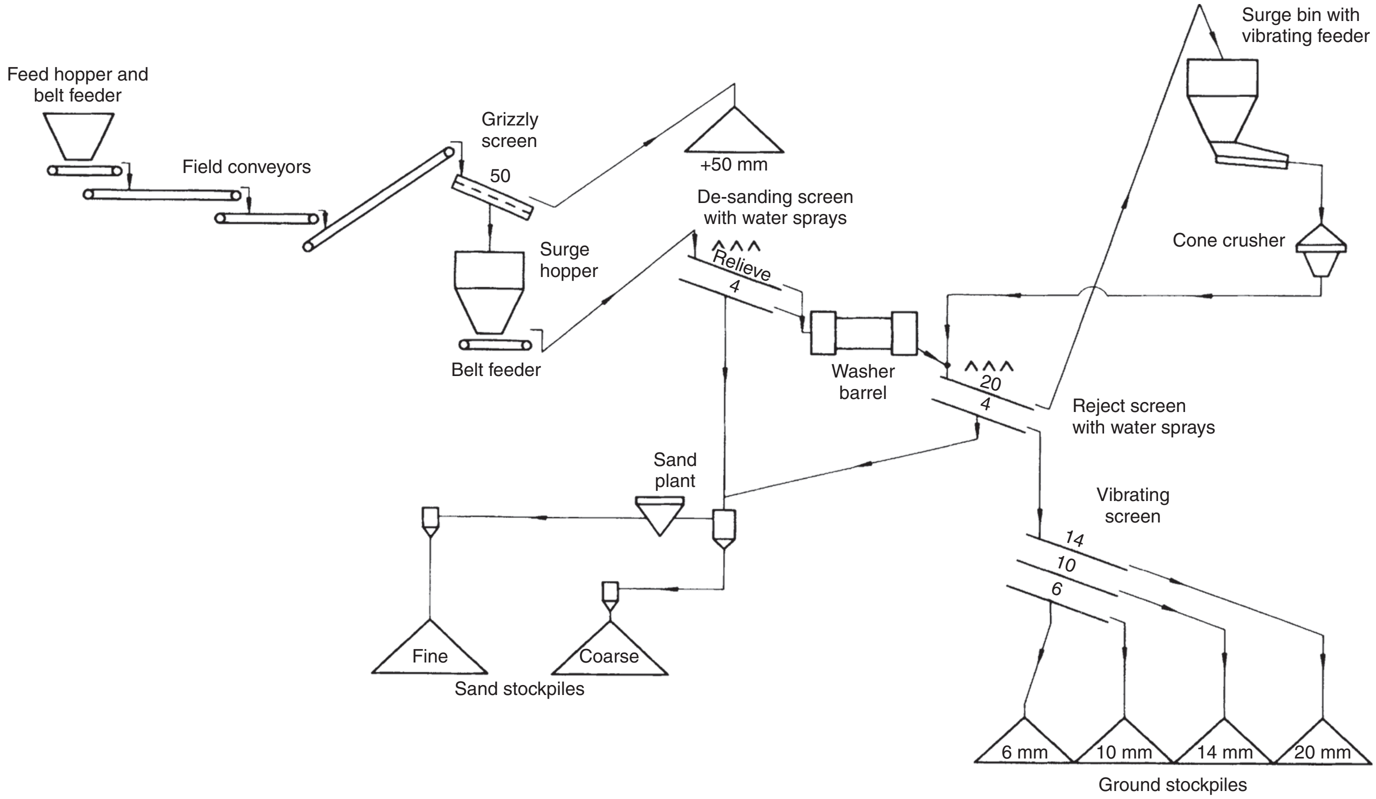

Figure 7 Flow diagram of a sand and gravel plant

The sand and water are flumed to a sump and separated into coarse and fine fractions. The dirty water is pumped to lagoons where the silt settles out prior to the water being recirculated to the plant. The plus 20 mm material from the reject screen is transferred to a surge bin. An electric vibrating feeder discharges the material to a cone crusher and the crushed gravel is returned to the reject screen. The minus 20 mm plus 4 mm material is conveyed to a grading screen separating out 20 mm, 14 mm, 10 mm and 6 mm products to ground stockpiles. These single sizes can be stored in bins and discharged, individually or blended, to trucks. A stockpile is a good means of allowing the gravel to de-water before further processing.

Methods of producing asphalts

The selection of the type of plant for producing asphalts is never easy since the circumstances related to each purchase are different. The plant could be required for a quarry where the aggregates can be controlled and are known to be in plentiful supply, or a depot site where the aggregate source is likely to change. The type of plant chosen is influenced by the markets in which the materials are to be sold since they determine the product requirements. The duty required will not only be directly related to the market potential but also to the number and type of plants in the surrounding area. The budget will play a major role in determining the equipment selected, as will the running costs. Finally, the type of plant will be influenced by local site restrictions such as noise, availability of space, height and even emission regulations.

Asphalt plants operating in Europe are designed to meet the European safety Standard BS EN 536:1999.

There are two basic categories of production facilities: batch plants and continuous plants. Batch plants are conventional asphalt batch plants and batch heaters. Continuous plants can be categorised into drum mixers and counterflow drum mixers.

Conventional asphalt batch plants

A conventional asphalt batch plant is shown in **Figure 8**.

The aggregates specified in the final mix are loaded into the feed hoppers; a different hopper is used for each ingredient. The feeders control the flow of material from the hoppers and provide a proportional feed. Each feeder is set to give the required percentage in the finished mix. A controlled and accurate feed rate is needed otherwise energy will be wasted on unused materials. The feed materials are conveyed to the dryer, consisting of a rotating cylinder with a burner mounted at the discharge end. The cold and wet aggregates are dried and then heated within the cylinder. Inside the dryer is a series of lifters which

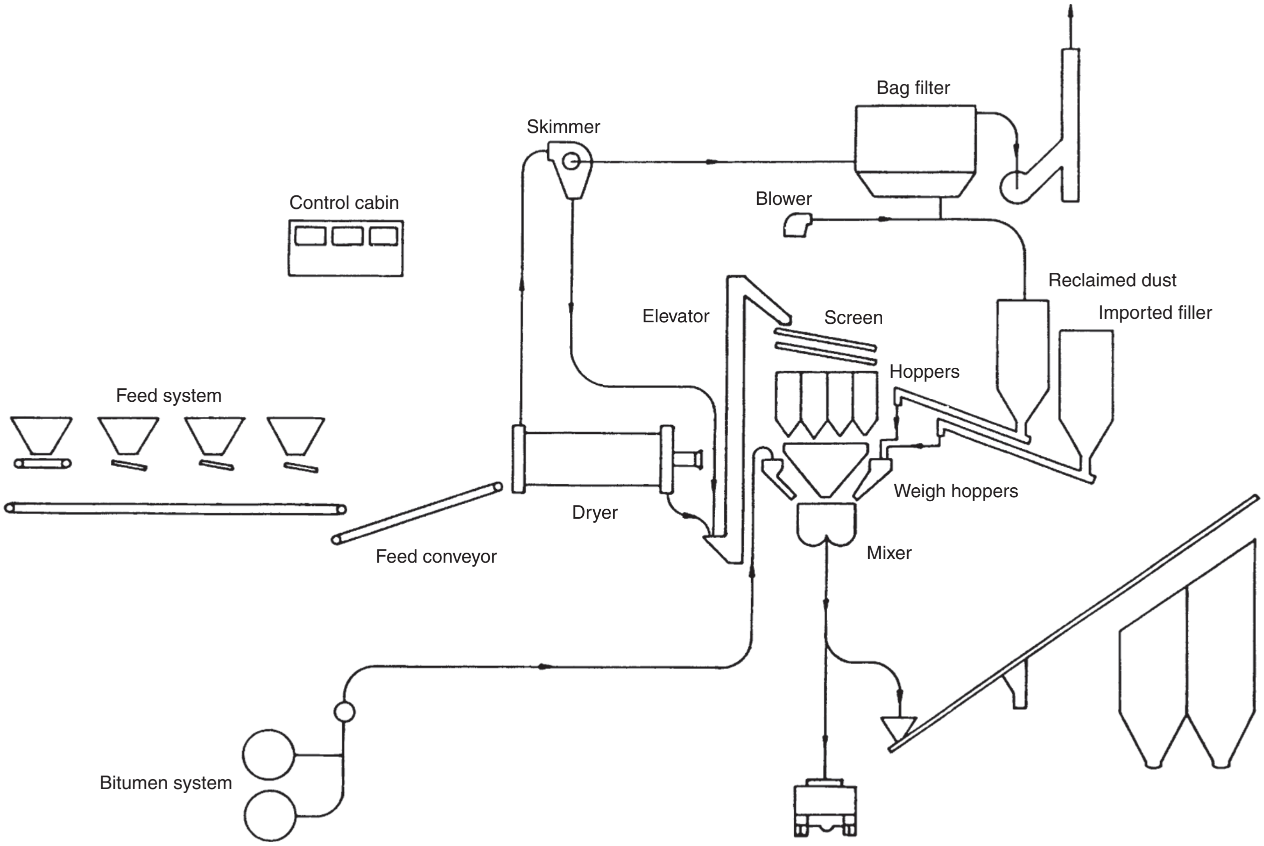

Figure 8 Flow diagram of a conventional asphalt batch plant

are designed to cascade material across the diameter of the drum. The lifters at the feed end produce a dense material curtain to expose the aggregate and sand to heat from the burner. At the discharge end, the lifters hold the material around the circumference of the dryer enabling the flame to develop and complete combustion to occur. When combustion takes place within the dryer the gases expand. The products of combustion and the water vapour produced in the drying process are removed by an exhaust system. **Figure 9** shows a Titan 3000 conventional asphalt batch plant operating in China.

A contraflow air system operates on the majority of stone dryers, where the hot exhaust gases travel in the opposite direction to the material flow. This arrangement is an efficient process since the exhaust gases pre-heat the incoming material. The exhaust system creates an air speed within the dryer so that fine particles of dust are also picked up. **Figure 8** illustrates a dry collection system, the coarse particles, approximately +75 μm, are removed by the skimmer and returned to the plant via the hot stone elevator. The fine particles of dust are removed from the exhaust air by a bag filter. Emission levels of less than 20 $\mathrm{mg/m^3}$ are achieved using this method and all of the dust is reclaimed by the process. The material discharges from the dryer at temperatures ranging from 120–200°C and is fed into the hot stone elevator. The elevator transfers the material to the screen on the mixing section. Asphalt plant screens used in the British market grade four sizes plus rejects or

Figure 9 Conventional asphalt batch plant (courtesy of ACP Titan Overseas Ltd)

alternatively six sizes plus rejects. The material discharges from the screen into the hot stone bins; internal chutes enable the bin compartments to overflow to a single point.

The mix recipe selects the required aggregates to be weighed in the batch hopper and simultaneously the bitumen and filler are weighed off. The ingredients are then emptied into the paddle mixer until the contents are fully coated. Typically, mixing cycle times of 45 seconds and 60 seconds are possible, depending on the specification. The mixer can load directly to trucks or to a skip for transfer to mixed material storage.

Ancillary equipment includes heated bitumen storage with a ring main to the bitumen weigh hopper, imported and reclaimed filler systems consisting of a silo, rotary valve and screw conveyor to the filler weigh hopper.

Table 1 lists the advantages and disadvantages of conventional asphalt batch plants.

Advantages	Disadvantages
Ability to manufacture all materials to PD 6691:2007	Heat wasted on rejected and overflow material
Emission to atmosphere within acceptable limits	Relatively high maintenance costs
Inconsistent feed materials tolerated	High production costs compared with drum mixer process
Small tonnages possible	Relatively high capital cost
Production of materials at all temperatures	Capacity restricted to mixer size and mixing cycle
Mixed material storage is not essential	
High percentage of reclaimed material can be added	

Table 1 Advantages and disadvantages of conventional asphalt batch plants

Batch heater plant

A flow diagram for a batch heater plant is shown in **Figure 10**. This process is different from the conventional asphalt plant since each batch is manufactured individually. Pre-graded aggregates are fed to the plant in a single batch by time-controlled feeders; the time is directly related to the proportion set in the mix recipe. The materials are conveyed to a check weigh hopper where the wet feed weight is determined. An allowance is made for the moisture content in the feed, the control system then calculates the bitumen and filler contents and proceeds to weigh these ingredients.

Drying and heating of the materials takes place in a short rotary drum and the discharge temperature is controlled by the time the materials are kept in the heater. All materials are discharged into a paddle mixer where a homogeneous mix is produced. A typical mixing cycle time is 2 minutes but this depends upon the moisture content in the feed and the material specification. The mixer can discharge directly to trucks or, alternatively, to mixed material storage. Some plants have an additional weigh vessel between the batch heater and the paddle mixer to determine the dry aggregate weight. The bitumen content is then

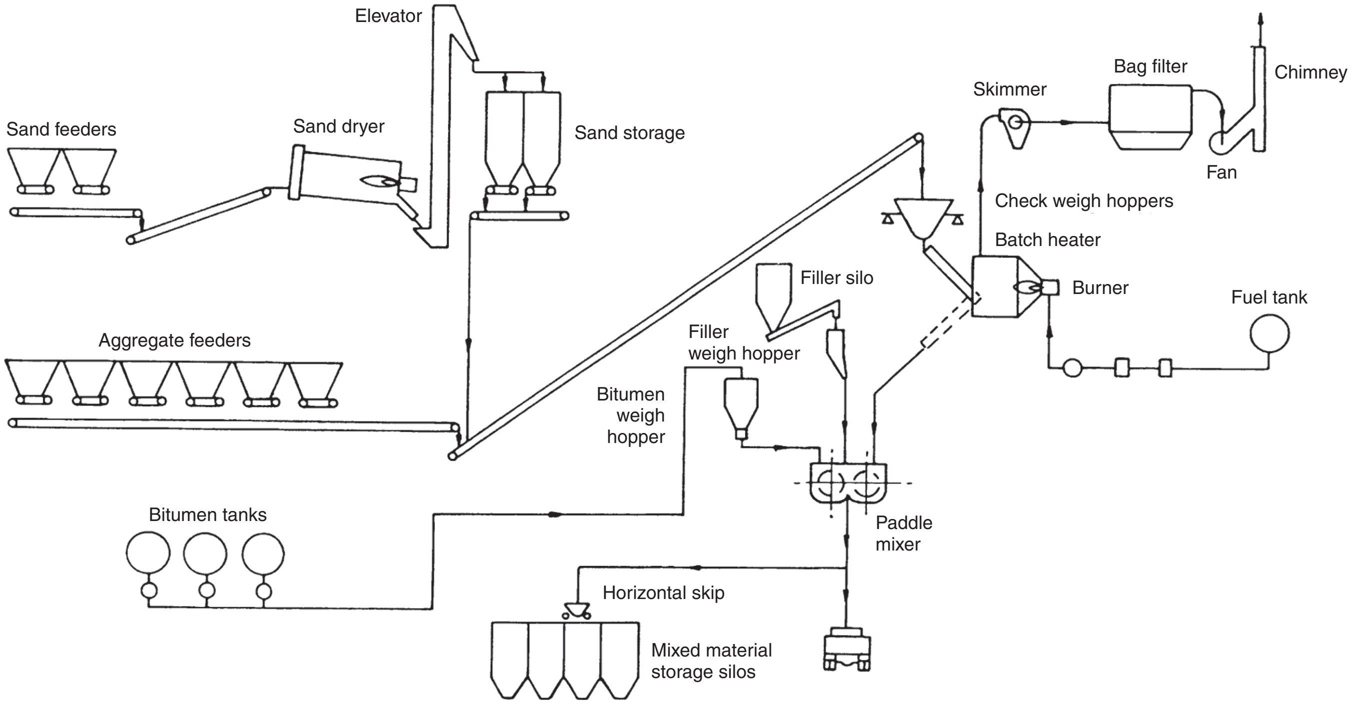

Figure 10 Flow diagram of a batch heater

Figure 11 Batch heater plant Hanson UK (courtesy of Hanson UK)

calculated from this dry batch weight. The skimmer can be arranged to discharge on a discrete basis into the paddle mixer.

The bitumen storage and ring main, the filler storage and the bag filter exhaust system are generally as described previously. If moisture contents greater than 2% overall are experienced then a pre-dryer is used for the sand, as shown in **Figure 10**. The sand is then called up from the dry storage.

Figure 11 shows a batch heater plant. **Table 2** lists the advantages and disadvantages of batch heater plants.

Drum mixer

A flow diagram of a drum mixer is shown in **Figure 12**.

The feed section is one of the most important aspects of the drum mixer process. It is essential for the success of the finished product that pre-graded aggregates are used. The feeders control the flow of material to the plant at a rate proportional to the quantities called for in the mix. Volumetric feeders are generally employed although an option that is sometimes preferred is the use of weigh feeders especially for sand and dust. Wet sand and dust can be difficult to handle and they do not always flow evenly or readily. The use of a volumetric feeder cannot therefore be relied on and in these instances a weigh feeder is used.

A scalping screen can be incorporated into the feed system to remove unwanted oversize material when the quality of the aggregates cannot be guaranteed. For example, this would be appropriate if the plant is fed from ground stockpiles.

The materials pass over a belt weigher before entering the drum. The mass passing over the weigher supplies the master signal to the ratio controller which determines the flow rates for the other mix ingredients making allowance for the moisture content in the feed.

The drum mixer consists of a rotating cylinder with the burner mounted at the feed end. Lifters around the circumference of the drum keep the material away from the flame, followed by a lifter pattern which forms a dense cascading material curtain. After drying and heating, the lifters are designed to mix the aggregates with bitumen inside the drum. The binder is usually added from the discharge end to prevent degradation which can occur if it is too close to the burner flame. Imported or reclaimed filler can also be added to the mixing zone.

One of the most accurate methods of providing a continuous flow of filler is to employ a ‘loss-in-weight’ system. Special attention is given to conditioning the filler and it is continuously reverse weighed against a known calibration rate. A pneumatic conveyor transfers the filler to the drum mixer. An exhaust system removes the products of combustion and a filter prevents the inherent dust particles being emitted into the atmosphere. Due to the nature of the process the dust collected is considerably less than that which is recovered from a conventional dryer.

As the process is continuous, large tonnages are possible in a relatively short timescale; a mixed material system is

Advantages	Disadvantages
Small batches economically produced	Single-sized feed materials
No material wastage	Maximum 2% moisture in feed materials
Ability to manufacture all materials to PD 6691:2007	Pre-drying required if moisture content greater than 2%
Quick recipe change	Relatively high capital cost
Mixed material storage is not essential	Relatively high maintenance costs
Number of feed materials is only limited by quantity of feeders	
Reclaimed material can be added	

Table 2 Advantages and disadvantages of batch heater plants

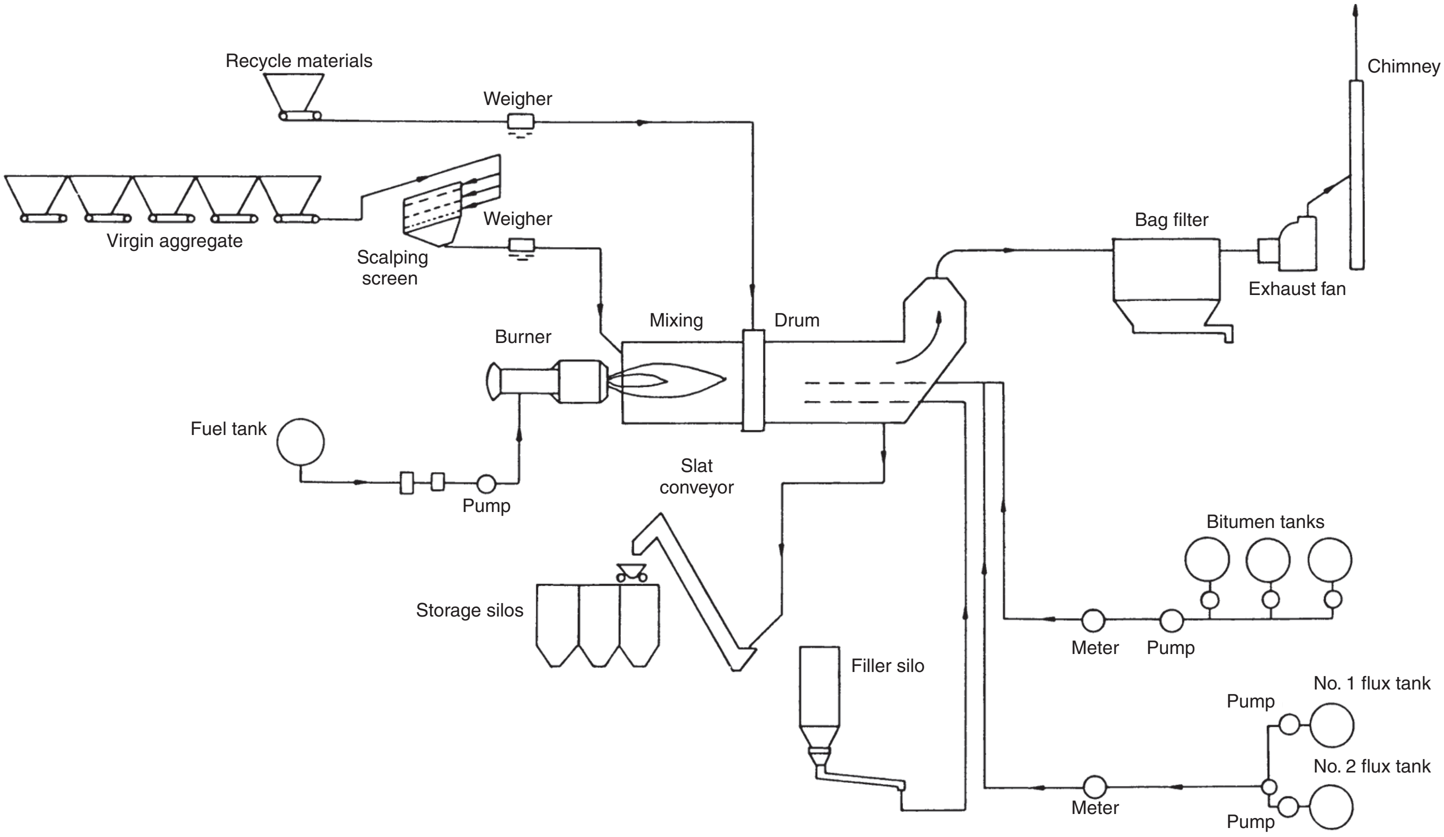

Figure 12 Flow diagram of a drum mixer with recycling facility

therefore needed. Belt conveyors, travelling skips and slat conveyors have all been used for these storage schemes.

Figure 13 shows a Cedarapids Standard Havens drum mixer plant operating in San Francisco. **Table 3** lists the advantages and disadvantages of drum mixers.

Counterflow drum mixer

Figure 14 shows the layout of a counterflow drum mixer.

There are two major differences between the counterflow drum mixer and the drum mixer. Firstly, an extended burner is used and this is mounted at the discharge end of the drum and, secondly, the mixing chamber is completely separated from the drying and heating zones.

The drying section of the counterflow drum mixer is identical to a conventional dryer, pre-heating of the feed materials results giving a more efficient drying method. The burner flame cannot be in direct contact with the bitumen since the mixing zone is isolated. As a result, the possibility of fumes is reduced considerably. A scavenger fan removes the hot gases and any fumes present from the mixing chamber are incinerated by the burner flame. Bitumen, filler and reclaimed material are added to the mixing zone as called for by the specification. The drying and heating section can be used purely as an aggregate dryer if required.

The principle of operation and associated equipment such as feeders, scalping screen, weigh conveyor, exhaust system, bitumen system, filler storage and mixed material are as described for the drum mixer. This process evolved from the drum mixer to meet the need for a continuous plant with improved emission levels and a reduced fume content when mixing at higher temperatures or using different bitumens.

Table 4 lists the advantages and disadvantages of counterflow drum mixers.

Summary of main features of asphalt production plants

Table 5 summarises the main features of asphalt production plants.

RAP and fibres for thin surface course systems

RAP (reclaimed asphalt pavement)

RAP can be added to each of the different types of asphalt plant, but it must be loose and less than 50 mm in size. The RAP may need to be processed by passing through a breaker and vibrating screen prior to its addition to the

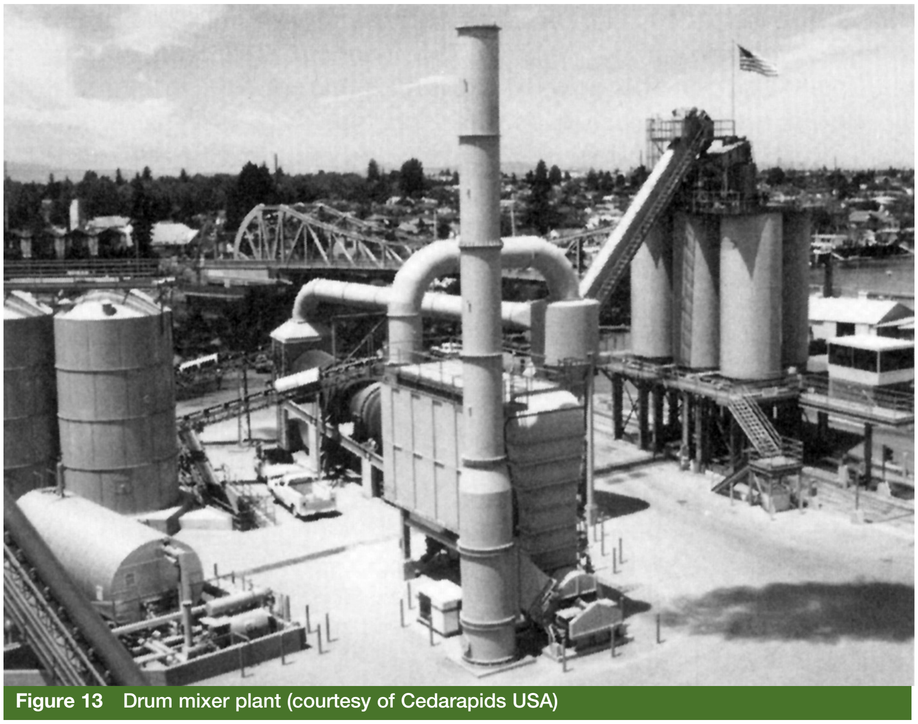

Figure 13 Drum mixer plant (courtesy of Cedarapids USA)

asphalt plant; however, this will depend upon its handling and storage after being removed from the road surface. Cancelled orders or waste from the asphalt plant can also be classed as RAP; these too may be recycled in the asphalt plant.

Up to 20% ambient RAP can be added to a conventional asphalt batch plant, the percentage to be added depending upon the moisture content in the RAP feed material. There are two basic methods of adding ambient RAP to the mix, the most common being to add the reclaimed material direct into the paddle mixer. The virgin aggregates are superheated; heat transfer takes place in the paddle mixer with the RAP to remove the moisture and to enable the final mix to reach the specified temperature. Adequate venting of the paddle mixer is important as the moisture removal creates a large expansion of gases. The equipment consists of a feed hopper, belt feeder and a conveyor fitted with a belt weigher. A flow diagram of the process is shown in **Figure 15**.

Advantages	Disadvantages
Economical plant for long production runs of one material specification	Unable to manufacture all PD 6691: 2007 materials
Number of feed materials is only limited by quantity of feeders	Single-sized feed materials
Dust removed from process is minimal	Mixed material system essential
High capacities readily achieved	Small batches are uneconomical
Easily adapted to mobile design	Possibility of fume emission with high temperature mixes
Relatively low maintenance costs	Wastage at beginning and end of production
Reclaimed material can be added	

Table 3 Advantages and disadvantages of drum mixers

An alternative method of adding ambient RAP to a conventional asphalt batch plant is by means of a feed hopper, belt feeder and conveyor discharging directly into the aggregate weigh hopper. In this arrangement the RAP is considered as an additional ingredient in the mix. A flow diagram of the process is shown in **Figure 16**. Again, the virgin aggregates are superheated and adequate venting is necessary to cope with the expanded gases. The only disadvantage with this method is that the aggregate weigh hopper can become contaminated with bitumen.

Up to 50% of heated RAP can be added to the mix on a conventional asphalt batch plant using the parallel drum method. The RAP material is heated to between 80 and 120°C to remove the moisture. It is then weighed in a separate vessel and transferred to the paddle mixer; a flow diagram of the process is shown in **Figure 17**. The exhaust from the reclaimed, or black dryer, is directed into the discharge end box of the virgin drum where the fumes are incinerated.

RAP can be added to the batch heater plant by means of a feed hopper, belt feeder and conveyor fitted with a belt weigher. The RAP material discharges directly into the paddle mixer, the virgin aggregates are superheated to remove the moisture in the RAP and to enable the final mix to achieve the specified temperature in the paddle mixer. Venting of the paddle mixer is essential to

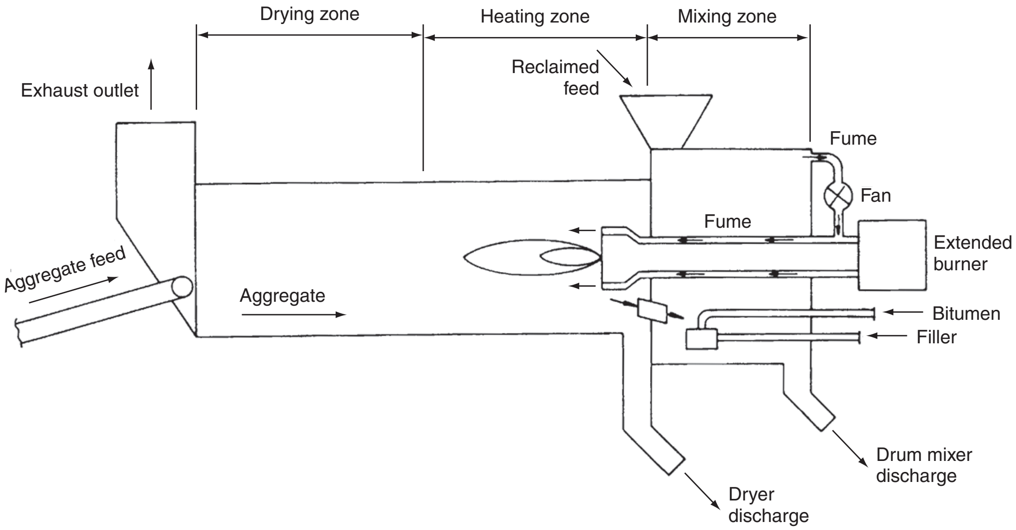

Figure 14 Flow diagram of a counterflow drum mixer

remove the expanded gases. The flow diagram for the RAP addition to the batch heater process is similar to that shown in **Figure 15**.

As much as 15% ambient RAP can be added to the drum mixer process by means of a feed hopper, variable speed belt feeder and conveyor fitted with a belt weigher. The flow diagram is shown in **Figure 12**. The variable speed belt feeder is controlled from the master signal generated by the virgin aggregate conveyor belt weigher. The RAP is added to the process through a collar mounted on the drum. The percentage of RAP is limited because excessive heating of the virgin aggregates to remove the RAP moisture and raise the product temperature causes fuming of the RAP and bitumen inside of the drum mixer. The fumes from the RAP and bitumen result in the bag filter becoming blocked and eventually plant capacities and emissions suffer.

The counterflow drum mixer can accept up to 50% ambient RAP added to the process. The RAP is added by a feed hopper, variable speed belt feeder and conveyor fitted with a belt weigher – exactly the same equipment and method as used on the drum mixer plant. The RAP is added to the process through a collar attached to the mixing zone as shown in **Figure 14**. As the drying/heating zone and the mixing zone are completely separate, the virgin aggregates can be superheated to a high enough temperature to remove the moisture from the RAP and to

Advantages	Disadvantages
Most economical plant for long production runs of one material specification	Wastage at beginning and end of production
Possibility of fume emission is reduced when producing high-temperature mixes	Small batches are uneconomical
Ability to manufacture all materials to PD 6691: 2007	Single-sized feed materials
Number of feed materials is only limited by quantity of feeders	Mixed material system essential
High capacities readily achieved	
Relative low maintenance costs	
Reclaimed material can be added	

Table 4 Advantages and disadvantages of counterflow drum mixers

Type of plant	Uses	Remarks
Conventional asphalt batch plant	The most common type of mixing plant in the UK used for small batch production and up to the medium capacity range of 240–300 t/h	Highly flexible production and capable of manufacturing all European Standard specifications
Batch heater	Ideal for small production requirements with the facility to change specifications quickly	Maximum allowable moisture content of 2%; a pre-dryer is needed if moisture is higher
Drum mixer	Economical plant for the production of large quantities of one material specification	
Counterflow drum mixer	Selected for the continuous production of large quantities of one material specification when the control of fume emission is critical	

Table 5 Summary of asphalt production plants

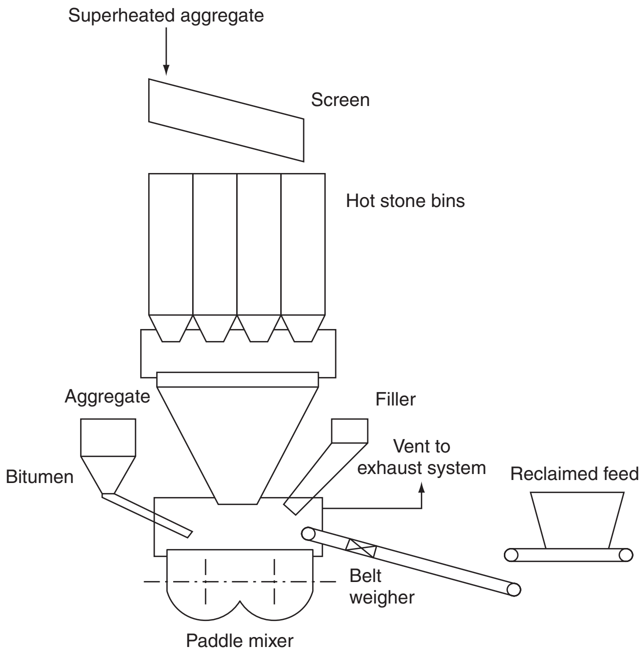

Figure 15 Reclaimed material feed to mixer

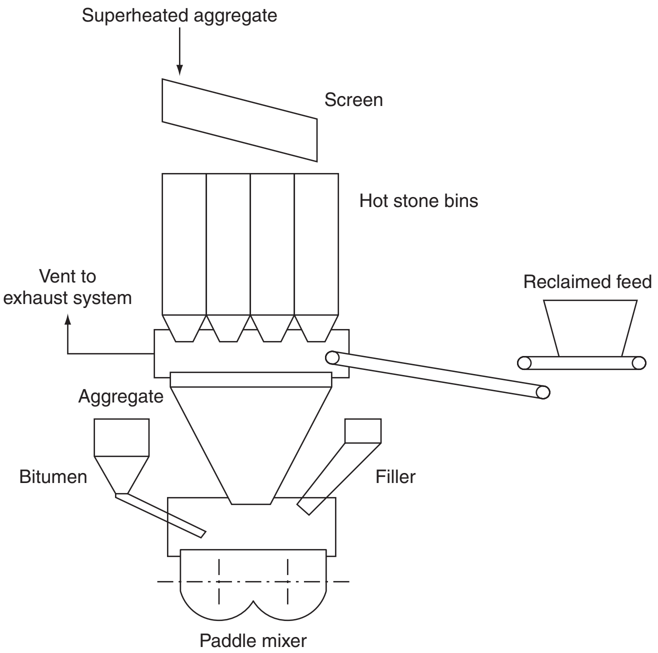

Figure 16 Reclaimed material feed to aggregate weigh hopper

enable the final mix to be at the specified temperature. Any fumes and the expanded gases in the mixing zone are removed by the scavenger fan and incinerated by the long nose burner. The exhaust gases from the counterflow drum mixer are therefore fume free and able to be processed by the bag filter without any detriment to its operation.

The addition of RAP to each of the different types of asphalt plant involves superheating of the virgin aggregates and heat transfer in the paddle mixer/drum mixer/counterflow drum mixer. Consequently, the higher discharge temperatures required from the dryer and the extended mixing times to enable the heat transfer to take place result in reduced overall plant capacities when RAP is added to the mix.

Fibres for thin surface course systems

First introduced in the early 1990s, thin surface course systems (often erroneously described as stone mastic asphalts) offer unparalleled surface regularity and other significant advantages over other materials when properly

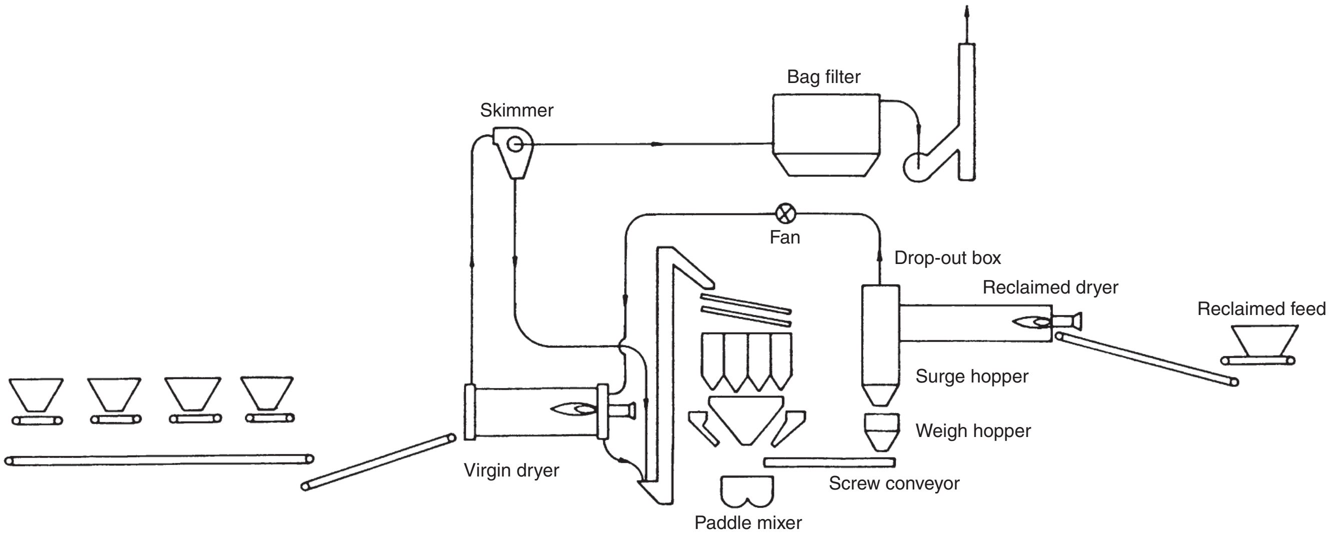

Figure 17 Flow diagram of a conventional asphalt batch plant with dryer for reclaimed material

designed and laid. These proprietary asphalts are the default surface course in England and are used in significant quantities in other areas of the UK. A thin surface course made using penetration grade bitumen has fibres added to the ingredients to hold the required quantity of bitumen in the mix. No fibres are required where the mixture includes a polymer-modified bitumen.

Cellulose or mineral fibres are added to the mix in loose fibre '*strings*' or in pellet form. The fibres originate from paper. They are added to the process either volumetrically, or by weight from a free-standing unit, usually mounted on the ground and transferred directly to the paddle mixer by pneumatic conveyor or screw conveyor. An alternative is to incorporate a separate weigh hopper as an integral part of the asphalt plant specifically for the fibres. Conventional asphalt batch plants and batch heaters are normally used for producing thin surface courses with fibres; it is difficult to get an accurate and consistent flow of fibres for feeding the continuous asphalt plants.

The amount of fibre added to the mix in the specification is relatively small – typically 0.3% by weight of the total batch size would be added as fibre. Therefore, if the batch size is 3000 kg then the fibre content would typically be 15 kg per batch. The production of thin surface courses results in reduced plant capacities as the mixing times are extended to ensure homogeneous distribution of the fibres throughout the mix.

Controlling manufacture

Aggregate processing plants

Control of the feed material in an aggregate processing plant is important for several reasons. Jaw, gyratory and cone crushers should be choke fed to optimise plant performance and obtain good product shape. Maximum feed sizes allowed to enter the crushers should be observed in order to provide continuity of production as a blockage within a crusher, or chute, can cause considerable delays. Unwanted materials such as soil, clay and very fine rocks should be prevented from entering the plant since these reduce overall throughputs.

Tramp iron (unwanted iron or steel in the aggregates, e.g. an excavator tooth) should be avoided in the feed since this can permanently damage the crusher wear parts. A metal detector is therefore usually fitted before the secondary crushing stage. An audible warning can be given to detect the metal and/or a magnet used to remove it. The majority of crusher wear parts are made from manganese steel and as this material is non-magnetic, electronic detectors have to be used to identify any of these loose objects within the process.

A limestone quarry is shown in **Figure 18**.

Larger aggregate plants are controlled by microprocessors and these eliminate the need for costly, extensive electrical relays and their associated wiring. This method of

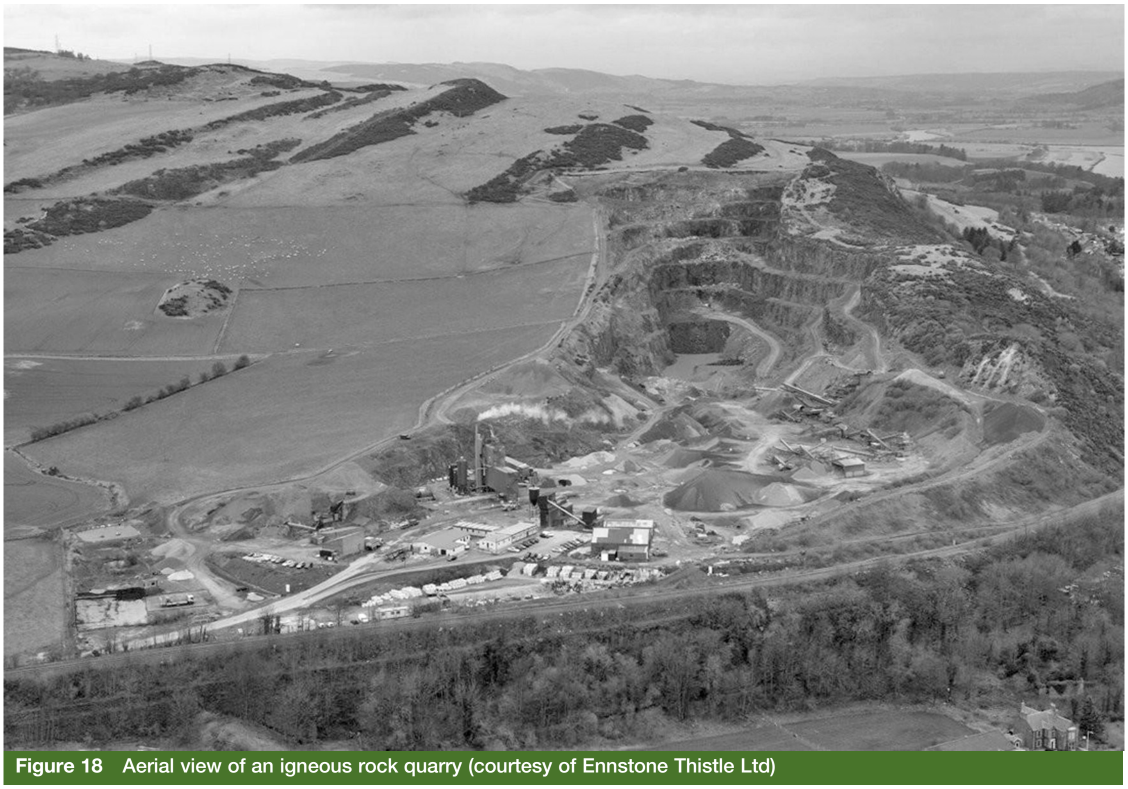

Figure 18 Aerial view of an igneous rock quarry (courtesy of Ennstone Thistle Ltd)

control also enables the status of the plant such as material flow, motors on/off, doors open/closed or water supply on/off to be monitored and alarmed if a fault condition occurs. Individual crushers can be microprocessor controlled to automatically adjust, or maintain, the discharge setting, e.g. in a cone crusher. This is a useful feature since a granite with a high silica content can noticeably wear the liners of a cone crusher in a very short time, say 4 or 5 hours, and such crushers require regular resetting to maintain a quality product.

Automatic overload protection is available on jaw, cone and gyratory crushers by means of a hydraulic system. Tramp iron and other uncrushable materials are allowed to pass through the crushers without costly damage to the liners and the original setting is then adopted. This would be in addition to the metal detector discussed earlier. The correct screen mesh sizes need to be fitted to exercise full control over the product sizes. The meshes should be inspected regularly for proper tensioning and holes caused by wear. The individual units of new aggregate plants are usually sheeted which makes the plant more aesthetically acceptable as well as reducing noise and dust emissions. The nuisance dust from the conveyor transfer points and housings is collected in a dry bag filter and the '*cleaned air*' emitted to atmosphere.

A recrush facility enables the plant to be more versatile with the sizes being manufactured. This feature incorporated in the design is useful when the market trends change. The larger-sized materials, 40 mm, 28 mm and 20 mm, are reduced to the more popular smaller sizes, 14 mm, 10 mm and 6 mm.

The final products are either stored in stockpiles on the ground, or in storage bunkers. In either case, contamination of the aggregates should be prevented. Stockpiles should be on hardstanding such as the quarry floor, or concrete to prevent contamination from below and separated to avoid overspill. Storage bunkers should be fitted with high-level indicators or overflow chutes. Having invested substantial amounts of time and money to ensure that the product size and shape are correct, degradation of the material should also be prevented. Rock ladders are used within stockpiles and storage bunkers to control the flow of large aggregates from a height and overcome unwanted breakages.

Laboratory inspection of the materials, both during and after production, is an important aspect of controlling manufacture and quality. The results of the samples from within the process indicate whether the plant is functioning correctly and enable adjustments to be made where necessary. The results from the final material samples determine whether the product is acceptable for sale. Single-sized products will be checked against PD 6682-2: 2003. The sample must be representative of the bulk material and the personnel undertaking the testing must be skilled operatives. Methods of sampling and testing aggregates are given in PD 6682-9: 2003.

Asphalt plants

Storage of raw materials

Asphalt plants can either be located adjacent to the aggregate plant in the quarry or remote at a depot site. If the plant is remote then the aggregates will be transported by truck or rail for larger operations. The quantity of materials delivered by rail necessitates an automatic offloading scheme incorporating underground discharge hoppers, tripper conveyor and covered storage bays. The material is recovered from storage by an underground feed system to the asphalt plant. Both truck and rail delivered aggregates should be stored on hardstanding and separated to avoid contamination. This method of storage along with a quality feed material provides a consistent feed to the asphalt plant, reducing overflow, wastage and fuel costs.

The ideal situation is for an asphalt plant to be fed directly from the aggregate plant bunkers. The product sizes can be more easily controlled by matching the screen mesh sizes on both plants and, since the material has not been exposed to the weather since entering the quarry plant, the moisture content will be lower. This layout can reduce the overall feed moisture content by up to 2% and is therefore more economical.

Under certain weather conditions, the filler silos containing imported and reclaimed materials may develop condensation on the inside walls. As filler is a hygroscopic material, it has a tendency to absorb moisture and the contents of the silos can become solid if left for prolonged periods. It is good practice to order only the quantity of imported filler needed for planned production runs and leave the silos empty when the plants are not operational for extended periods.

Bitumen needed for production is stored in the asphalt plant. While some plants only have the capability to store a single grade of bitumen others have the facilities to store two or more grades and some allow blending to achieve the required penetration. The bitumen is usually circulated in a ring main system up to the mixing plant before usage to reduce heat loss in the pipework, even though the pipes are heated and insulated. To prevent the bitumen hardening through oxidation, the material should not be stored at high temperatures for long periods, especially if no new bitumen is added. Prolonged storage with no usage should be at a temperature of around 25°C above the softening point of the bitumen and recirculation stopped. Bulk reheating of the bitumen should be carried out intermittently to prevent local overheating around the heating member. Bitumen can ignite instantaneously if temperatures above 230°C are reached, so such temperatures must be avoided and are in excess of the maximum permitted temperatures even for hard grades.

Plant capacities

There are two basic constraints on the output of a hot-rolled asphalt and asphalt concrete plant: the drying capacity and the mixing capacity. The drying process is complex. Some of the factors that directly affect the capacity are:

- altitude
- air temperature
- type of feed material and grading
- feed material temperature
- moisture content in feed material
- moisture content in discharge material (usually less than 0.5%)
- dryer discharge temperature
- exhaust temperature
- dryer angle
- calorific value of the fuel
- exhaust air volume available
- lifter pattern in dryer, and
- efficiency of burner.

The mixing process is also complicated and some of the factors affecting the capacity and whether the mix is homogeneous are:

- mix recipe
- mixer live zone
- paddle tip speed
- size of paddle tips
- mixer arm configuration
- method of feeding ingredients
- temperature of materials, and
- mixing time (some mixes have a minimum mixing time).

One of these processes will determine the overall plant capacity. In a conventional asphalt batch plant and a batch heater, the two processes are separate and can therefore be easily identified and considered. In a drum mixer and counterflow drum mixer, the drying and mixing are combined and require more detailed consideration. Basically, the feed material must be completely dried and heated before the bitumen is added. The drying constraints are therefore similar to the batch plant dryers. The mixing time in a continuous plant is controlled by adjusting the throughput, altering the drum angle, adjusting the position of the bitumen injection pipe within the drum, altering the design of the mixer lifters and altering the speed of the drum. In practice, once these variables are set during commissioning, they are not usually altered.

To determine the equipment sizes for an asphalt and bituminous macadam plant the manufacturer requires the following information:

- maximum plant capacity
- type of feed material
- average moisture content of feed material
- type of mixes to be produced
- temperature of mixes, and
- special constraints, e.g. minimum mixing time.

Plant control

With the exception of the batch heater, which has fixed-speed time-controlled feeders, most plants have variable speed feeders with a 20:1 turn down ratio (the feeders operate from 5 to 100% of the belt speed or capacity). Volumetric belt feeders generally have a repeatability accuracy of ±0.5% and vibrating feeders are around ±2%. This repeatability and turn-down ratio enables an accurate plant feed to be obtained and, providing the feed materials are within the specified size limits, this also ensures an economic control of the materials since heat will not be wasted on unused aggregates. Low-level indicators can be fitted to the feed hoppers with elevated lights above the hoppers to warn the shovel driver of an impending shortage. The feeders can be equipped with no-flow indicators and after an initial warning and a preset time delay, the plant will shut down if prescribed.

The burner mounted on the dryer, or drum mixer, can have turn-down ratios of up to 10:1. Different plant throughputs and different dryer discharge temperatures can therefore be easily accommodated with an economic use of the fuel. The burner is automatically controlled by a temperature sensing device in the dryer discharge chute. A contactless infrared pyrometer is normally used to record the temperature as the response times are a few seconds compared with a thermocouple probe which can take up to 15 seconds.

To sustain combustion, an exhaust system is connected to the dryer or drum mixer. As air is drawn through the drum, small particles of dust are picked up. This must be removed before discharge to the atmosphere. A dry bag filter dust collector is employed to remove the dust and plants are currently being guaranteed to meet particulate emission levels of 20 mg/m^3. The dust collector also pulls from nuisance points on the plant and controls the overall emission level. The air volume is controlled by a variable damper in the system, usually fitted prior to the fan on the clean side of the filter. The damper is controlled by a pressure reading taken at the burner end of the dryer.

Continuous level indicators can be installed in the hot stone bins of conventional asphalt batch plants. These give an indication of the material level within each compartment and provide a means of trending. From this information, the feeders can be set up during commissioning and adjusted when, and if, the feed aggregates become inconsistent. Again, this enables the plant to operate more economically with reduced overflow.

Temperatures of the sand in the hot stone bins and the bitumen in the ring main are taken by thermocouple probes and recorded. These readings confirm that the temperatures are correct before mixing commences.

Although the European Standard (BS EN 13108-21: 2006) allows tolerances on the amount of bitumen in the mix (typically ±0.5% or ±0.6%), plants are expected to perform with an accuracy of ±0.3%. Bitumen is the most expensive ingredient and therefore economics dictate the importance of this requirement. Batch plants are equipped with load cell weigh systems with repeatable accuracies of ±0.3% for the bitumen and filler hoppers. Continuous plants use a load cell in the form of a belt weigher and accuracies of ±1% are guaranteed.

Fully automatic microprocessor control systems are used on all types of asphalt plants. On batch plants, the microprocessor provides accuracy by calculating the bitumen content as an actual percentage of the aggregate and filler content achieved. Outputs are maintained at an optimum by weighing off a large percentage of the bitumen at the same time as the aggregate. The required bitumen is then calculated relative to the actual aggregate and filler content. Self-learning in-flights (to provide accurate weighing) for the hot stone bin discharges and alternative bin selection are also features incorporated in conventional asphalt plants which help throughput while maintaining accuracy.

The use of microprocessor control allows the operator to adopt a passive role during production since the plant is operated from the input data. Providing these data are correct and no faults are recorded, the finished mix will be correct. The operator is relieved to a certain extent of the responsibility and therefore the quality of the product is not operator dependent. The monitoring of process faults such as sticking doors, tare (weight) faults, stopped motors, broken chains, or sticking valves reduces down time and improves the quality of the products. In addition, the microprocessor can stop the plant from mixing if acceptable preset limits are not met. These faults are logged and recorded on the system, again providing quality control provided they are not ignored or overridden by the operator or QA staff.

An infrared pyrometer can be positioned at the paddle mixer discharge and the mixed material bin outlets to read and record the final discharge temperature. Microprocessor control can prevent contamination of different mixes within the mixed material system by remembering the type and quantity of mix in each bin. Storage silos mounted on load cells can accurately weigh out mixed material to trucks, avoiding wastage and saving time.

Full microprocessor control systems offer VDU screens with plant mimic diagrams, the ability to store over five hundred recipes, production data, storage data, a daily job queue, printout of mix results, data, time, etc. The printout and storage data are important elements of quality assurance since each batch and truck load can be traced; confirmation of a manual or automatic mix is also recorded. The controls information can be limited to the laboratory, the weighbridge and the main office accounting system, allowing information to pass in both directions.

Production and operating conditions

To obtain optimum fuel usage, a constant and uniform feed to the plant is needed. The aggregates should preferably be taken from large covered storage bins or the same place on the stockpile each time, avoiding the lower stock near to the ground and surface water from entering the loading shovel. This procedure will ensure that the minimum amount of water enters with the feed aggregates and provide uniform drying conditions. The dryer, burner and exhaust system operate more efficiently under stable conditions.

The dryer has a minimum capacity directly related to the effectiveness of the lifter pattern, the turn-down ratio of the burner and the maximum inlet temperature permitted to the bag filter. The majority of filters in the UK are equipped with Nomex material for the bags. This material has a maximum operating temperature of approximately 200°C, although higher peak temperatures can be accommodated. With a small aggregate feed to the dryer, the burner on minimum flame and a discharge temperature of, say, 190°C, there will be occasions when too much heat is available and the inlet temperature probes to the filter will automatically switch off the burner fuel supply. The dryer angle can be lowered to try to overcome this by increasing the drum loading but this will not resolve the problem when a small production rate is required. As a general rule, the minimum drying capacity for a plant equipped with a bag filter is approximately 50% of the capacities quoted for hot-rolled asphalts. This constraint does not apply to plants fitted with wet collection exhaust systems.

As the heated aggregates pass from the dryer to the mixer, there is an initial temperature loss of between 10°C and 20°C depending upon the required mix temperature. During long production runs, this temperature difference can be reduced to between 0°C and 5°C.

When drying and heating gravels or marine gravels for use in asphalt production, it is important not to overheat the aggregate. Overheating can cause *sweating* of the aggregate later in the process and this will eventually lead to stripping of the binder. For this reason, and as an aid to adhesion, since crushed gravels still tend to be slightly rounded, the addition of 2% Portland cement or hydrated lime at the mixing stage is required by some specifications, e.g. paragraphs B.4 and C.4 of PD 6691:2007.

The efficiency of the vibrating screen on a conventional batch plant changes with different flow rates. Consistent feed gradings and constant flow rates are, therefore, preferred so that the aggregate separation into each hot stone bin compartment can be predicted. The continuous level indicators in each compartment are a useful check since, in practice, a consistent feed grading is difficult to achieve.

During production, care is taken not to contaminate hardrock mixed with limestone. However, the reverse situation can usually be accommodated unless surface course mixtures are being manufactured. In a conventional asphalt plant, the hot stone bins are completely purged when the type of feed material changes. Purging can lead to considerable delays when changing specification. As each batch is manufactured individually with a batch heater, this situation does not arise. Continuous mixing plants can change specifications during production providing the same type of feed material is used, otherwise the plants are purged.

The ideal production schedule is to manufacture the lower-temperature materials first and progress to the higher-temperature specifications. In reality this is not always possible and care has to be taken not to overheat a low-temperature material that is required in the middle of a high-temperature production run. Overheating can lead to oxidation and hardening of the binder and this may well change the mix characteristics which can have an adverse effect on the life of the pavement. Maximum mixing temperatures for asphalts and grades of binder are given in PD 6691:2007.

The paddle mixer has a recommended minimum capacity of 50% of the nominal capacity. Mixers do not function satisfactorily below this level as a homogeneous mix does not result and oxidation of the bitumen can occur. Prolonged mixing of a full or partial batch does not necessarily mean a more homogeneous mix. Extended mixing times can lead to binder oxidation and subsequent hardening.

To maintain the quality of the product, the equipment should be recalibrated on a regular basis. The feeders, the burner, the temperature measuring equipment and the load cell weighgear should all be inspected within a planned maintenance schedule to attain the required quality assurance standards.

Mixed material storage

Mixed material silos are manufactured in both square and circular forms with bin capacities ranging from 30–200 t each. The top, the top doors, the sides, the conical section and the discharge doors are all insulated with material which can be up to 150 mm thick. Heaters are usually attached to the discharge doors and the conical section and although the bin sides can be heated, it is not common practice. The heating can be electric or indirect by a hot oil heater and the heating system usually allows the temperature of each individual silo to be controlled separately. The heat is applied purely to maintain the temperature by replacing lost heat and not as additional heat for a mix with a low temperature. Overheating of the material in the silo can cause the bitumen to drain from the aggregate.

Mixed material silos are available with grease-sealed or oil-sealed doors at the inlet and discharge, with the facility to add an inert gas. This design feature provides extended storage times by preventing the mix coming into contact with the atmosphere and oxidising.

Even without the sealing and the addition of an inert gas, extended storage is possible. To achieve the best results, the silo is completely filled. Dense materials store better over a longer period. For a 12-hour period the heat loss is minimal, say a maximum of 3°C and, providing the outlets are large enough, the silos will discharge freely. If the material is to be stored for two to three days then a crust forms on the top and around the discharge outlet. This can be overcome by discharging one tonne of material at regular intervals, say every nine hours. Again, the temperature loss over these few days will be minimal, probably a maximum of 20°C due to the effectiveness of the heating and insulation. Hot-rolled asphalts and open-textured materials including thin surface course systems are not recommended for extended storage because of the difficulty in discharging from the silo outlets.

Sampling

Production must be monitored to ensure that the aggregate grading and bitumen content comply with the specification but not every batch can be tested. Samples are therefore taken at predetermined intervals and these samples must be representative of the bulk material. Minimum sample frequencies are given in BS EN 13108-21:2006 but must be sufficient to demonstrate that the plant is under control. Methods of sampling and achieving representative samples are described in PD 6692:2006 for asphalts and PD 6682:2003 for aggregates, sands and fillers. Sampling should only be performed by skilled personnel using well-maintained and calibrated equipment.

Samples should be taken of all materials delivered to the plant, i.e. aggregate, bitumen and filler. If the asphalt plant is located in the quarry with the aggregate plant then part of this requirement will have been fulfilled.

The aggregate samples from the hot stone bins are taken either from special sampling devices on the bins or directly through the paddle mixer. This tests the gradings within each bin and gives an indication of the efficiency of the screen. The results from these samples determine the amount to be weighed off from each bin during the weighing cycle. This exercise should be carried out at regular intervals to ensure aggregate grading control.

Samples of the mixed materials can be taken from the paddle mixer or drum mixer discharge and the mixed material silo discharge. The mixer and drum mixer discharge sample determines whether a homogeneous mix is produced and the silo discharge sample confirms that the material has stored favourably or indicates whether segregation or other deterioration has occurred. During production, samples are usually taken from the truck load of mixed material. The paddle mixer, drum mixer and silo discharge samples are taken during commissioning and calibration and subsequently only if problems occur.

Tests on the samples take time and a large quantity of material may have been manufactured and laid before the results are available. However, sampling is essential for demonstration of compliance with European Standards for asphalts (BS EN 12697:2006), for calibration purposes and good quality control during production. It also gives confidence in the performance of the plant and, therefore, has to be undertaken on a regular basis.

Summary

This chapter has dealt with the production of asphalts and its principal constituent, aggregate. It has considered the availability of aggregate sources in the UK and how these basic materials are processed using different types of crushing and screening plants. The various categories of asphalt production plant were examined. Finally, different factors affecting the control of manufacture were discussed.

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Chapter 28

Cold and warm asphalt mixtures

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Cold asphalt mixtures are produced through the use of bitumen emulsions or foamed bitumen, in contrast to hot-mix asphalt mixtures that are produced using conventional bitumen. Warm asphalt mixtures are closer to conventional hot-mix asphalt mixtures but are produced, through a combination of different production methods and bitumen additives, at lower temperatures. Cold and warm asphalt mixtures therefore tend to be more environmentally friendly than hot-mix asphalt mixtures in terms of both their production processes and their ability to include recycled asphalt pavement.

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Introduction

Excellent though bitumen is as a binder of aggregate particles, one of its drawbacks is that it will only adhere to aggregate properly if that aggregate is heated sufficiently to drive off all moisture, and this is a costly and energy-intensive process. Thus, there are strong economic and environmental drivers towards so-called ‘cold-mix’ technology, utilisng a form of bitumen which is workable at ambient temperatures and which can be mixed with cold, wet aggregate. Reliance on hot-mix asphalt also restricts the choice of aggregate to those which are able to withstand heat without either being damaged or else giving off excessive noxious fumes. Cold-mix technology opens the door to a range of alternatives, including in-situ recycling.

Warm mixtures are quite different in terms of the technology used; they represent a compromise between hot and cold. They aim to retain the properties of a conventional hot mixture while allowing a substantial reduction in the temperature required during production, paving and compacting, with associated environmental benefits as well as an extended workability period. Warm mixtures have no in-situ recycling applications but they can be used to increase the percentage of recycled asphalt pavement (RAP) in the mixture.

This chapter concentrates largely on cold mixtures and will discuss warm mixture technologies at the end.

Bitumen emulsion

To allow bitumen to mix with aggregate at a temperature where it would normally have very low fluidity, it is clearly necessary to radically alter the form of the binder, and bitumen emulsion is one way to achieve this. An emulsion is a suspension of one liquid in another, in this case of bitumen in water. It is manufactured by putting hot bitumen through a high shear colloid mill, which reduces it to tiny droplets, typically 2–20 μm across, at the same time as feeding in an emulsifying agent and water – see **Figure 1**. The emulsifying agent is a chemical with charge at one end, either positive or negative, and a long polymer tail with strong affinity to bitumen. As the emulsifier ions attach themselves to the bitumen droplets these are converted into charged particles, each repelling its neighbour; the droplets therefore ‘float’ in the water, prevented from coalescing by their charge. The specific gravities of bitumen and water are very similar (1.02–1.03 for bitumen) so there is little tendency for the droplets to sink under gravity and the result is a very stable product. Occasional light stirring is sufficient to ensure a long ‘shelf life’. See Read and Whiteoak (2003) for a fuller explanation.

Since the water phase is continuous, the fluidity of bitumen emulsion is not much reduced from that of water, despite the bitumen comprising between 40 and 70% (typically 60% for cold asphalt mixtures) of the total volume. This means it can readily be piped, sprayed and mixed with aggregate.

Emulsions are classed as either anionic (negatively charged) or cationic (positively charged). In theory the choice of charge should be governed by the surface chemistry of the aggregate with which the emulsion is to be mixed; in practice the large majority of emulsions used for cold asphalt production are cationic. Another variable is the size of bitumen droplets, which depends on the details of the colloid mill used, notably speed, and this too can affect the performance of an emulsion as a binder. Apart from the need to produce a stable suspension of bitumen in water, the issue that continues to drive research is how to ensure that the bitumen droplets adhere strongly to the aggregate, while also ensuring that this does not occur until placement and compaction have been completed, and this requires the right choice of emulsifier as well as, in some cases, additional chemicals either to delay or accelerate the process. The transition from an emulsion to the state where the bitumen begins to adhere to the aggregate

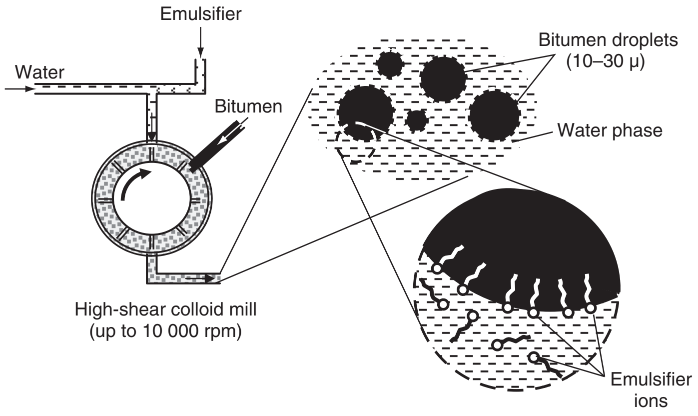

Figure 1 Bitumen emulsion (Thom, 2008)

is termed 'breaking'. Emulsifier chemistry is a science that has seen steady advance in recent years and several companies now produce emulsions that are tailored to specific types of mixture, requiring either rapid, medium or delayed breaking (e.g. Redelius, 1993).

Foamed bitumen

The alternative form of cold-mix asphalt utilises bitumen in the form of a foam. **Figure 2** illustrates the method of production. The foam is produced by the very rapid transformation of water into steam as it comes into contact with hot (typically 160–180°C) bitumen, aided by the addition of air. The water content within the foam is generally between 2 and 5% of the bitumen volume but, once transformed into steam, the foam expands to many times the volume of the bitumen itself, and in this state it can readily be mixed into an aggregate. The ratio of peak foam volume to original bitumen volume is known as the 'expansion ratio', while the time until this peak volume is halved is known as the 'half life'. The expansion ratio is typically between 5 and 20; the half life is typically between 10 and 40 s. See Jenkins (2000), Saleh (2006) and Sunarjono (2008) for detailed studies. **Figure 3** illustrates the appearance of foamed bitumen at the end of the foaming process, in which state it still contains bubbles and is more fluid than non-foamed bitumen.

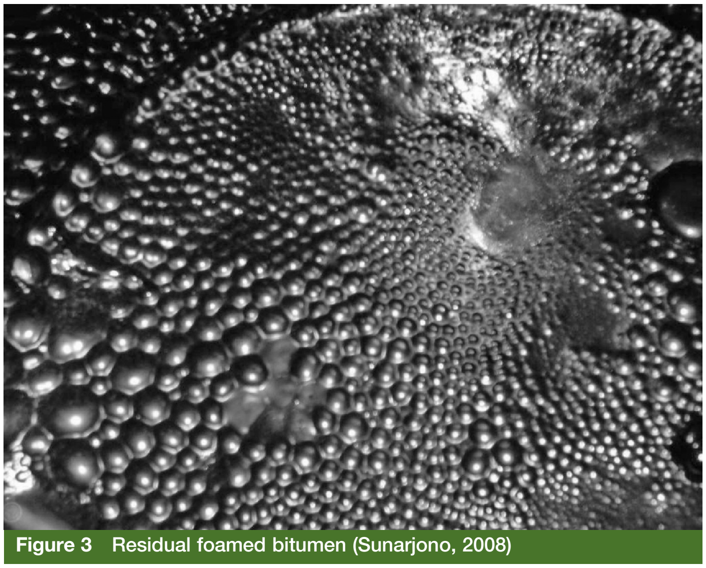

Figure 3 Residual foamed bitumen (Sunarjono, 2008)

Foamed bitumen therefore contrasts with emulsion. Emulsions can be stored for months before use, whereas foamed bitumen has to be used within a few tens of seconds of production. However, the advantage is that foamed bitumen requires less water and, as will be explained in relation to mixture properties, water presents the principal challenge to the successful use of both forms of cold mixture.

Cold mixture production

In both emulsion and foamed bitumen the binder is temporarily rendered fluid (by the action of water or a foaming process), in which state it is mixed with an

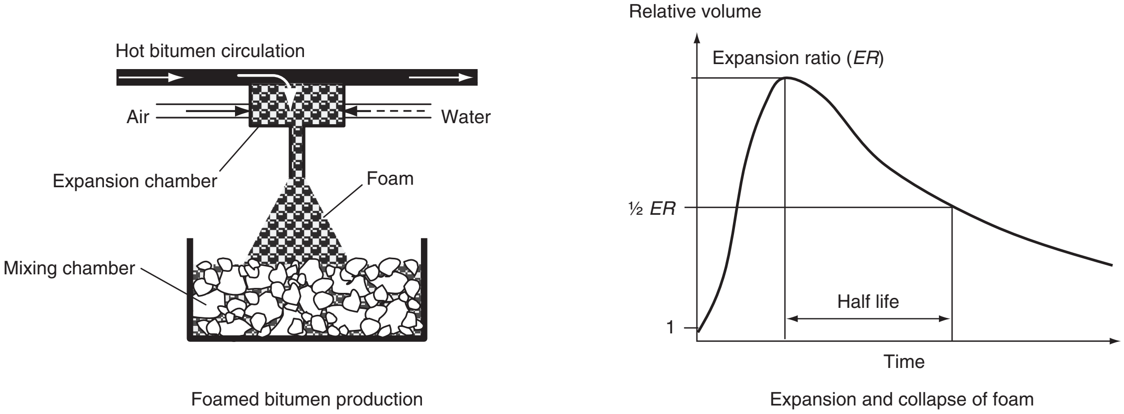

Figure 2 The foaming process (Thom, 2008)

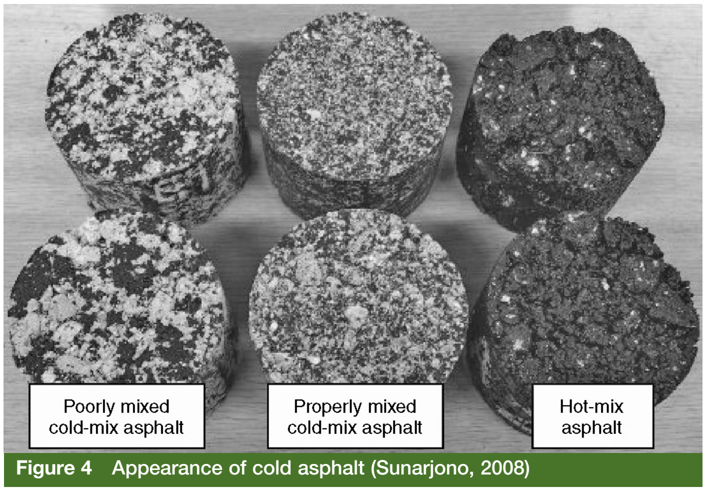

Figure 4 Appearance of cold asphalt (Sunarjono, 2008)

aggregate. The result is that the bitumen is dispersed throughout the mixture in the form of very small droplets, and at some stage these droplets are expected to adhere to individual aggregate particles. The following paragraphs discuss each stage in turn.

The mixing process

Mixing efficiency is, if anything, more critical for cold mixtures than for hot, particularly in the case of foamed bitumen mixtures where the mixing time window is limited by the collapse of the foam. **Figure 4** illustrates the appearance of poorly mixed and well-mixed materials. However, as **Figure 4** shows, even a well-mixed material looks quite different from a hot mix. This is because, in both emulsions and foamed bitumens, the bitumen is in the form of small elements rather than continuous coating; it finds its way wherever there is water in the mixture, which means that it has a preference for zones of fine material where surface tension holds water most effectively.

This last point is important; it means that the water content is a very important part of the mixture. An overriding requirement of any cold mixture is that it has to have enough water to form a thin film around each aggregate particle, a film into which the emulsion or foamed bitumen droplets can disperse during the mixing process. The greater the water content, the easier the binder will find this dispersal process. Experience suggests that a water content of about 3% is a realistic minimum, although this will depend on the gradation of the aggregate. A fines-rich material will require a relatively higher water content to ensure full binder dispersal. In general, emulsion requires a slightly lower aggregate water content than foamed bitumen since emulsion already contains significant water within it. The fact that cold mixtures rely on a different mechanism for dispersal of binder in comparison to hot mixtures can also be seen in the resulting void structure. The low viscosity of water allows penetration of all void spaces. As shown on the X-ray scans in **Figure 5**, the result, in this case for a foamed bitumen mixture, is that no large voids occur, only small voids dispersed amongst the fine aggregate and bitumen mortar.

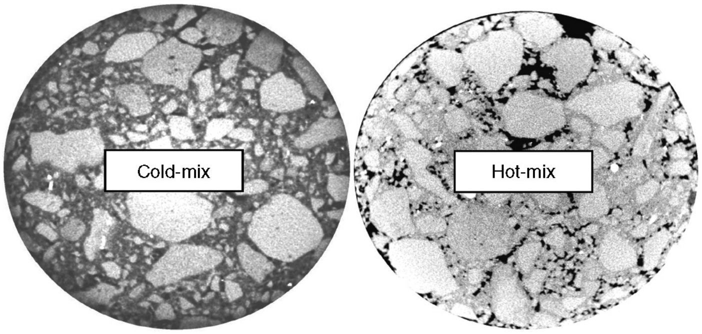

Figure 5 X-ray scans of hot and cold asphalt mixtures (Sunarjono, 2008). Black areas in hot-mix indicate relatively large voids

However good the efficiency of mixing, bitumen coverage will always be far from continuous and, of necessity, the quality of adhesion between bitumen and aggregate is initially poor due to the water film surrounding aggregate particles. In this state, the mixture is still workable; it can be stored loose for several weeks before use – another attractive property. In fact in bagged form, where water loss is prevented, it is claimed to have a shelf life of up to two years before sufficient adhesion develops to prevent compaction.

Compaction

A cold-mix asphalt requires a certain fluid (i.e. water + bitumen) content in order to achieve maximum density in the same way as an unbound material or a hot-mix asphalt. The fluid 'lubricates' the mixture, facilitating the movement of one particle relative to another. Optimum fluid content depends on the grading of the mixture, typically being about 5–6% by mass for a broadly graded material. If the fluid content is less than optimum (unlikely in the case of a cold-mix asphalt) there will be insufficient lubricant and maximum density will never be achieved; if it is above optimum there is insufficient space in the aggregate matrix for all the fluid, even when almost all the air voids have been expelled. Both scenarios result in a below-maximum density, which in turn results in reduced stiffness and permanent deformation resistance.

The problem for designers of cold asphalt mixtures is that at least 3% fluid will usually be present within the aggregate, which greatly restricts the binder content that can be added to the mixture. There are two clear dangers to bear in mind:

1. If the compacted mixture is nearly saturated, subsequent water evaporation is much more difficult than if air voids were present. This may slow the curing process significantly.

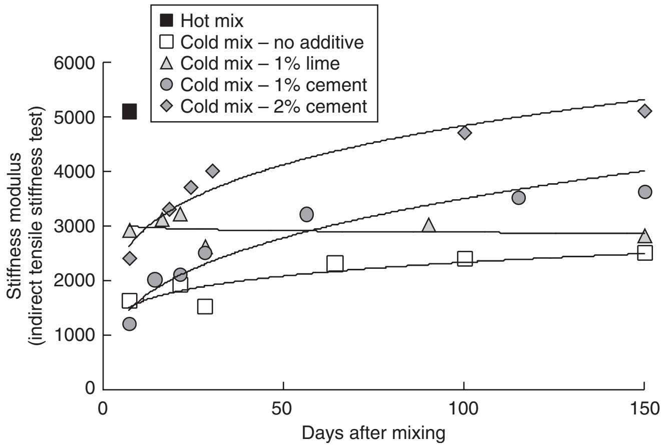

Figure 6 The curing process (Jitareekul, 2009)

2 Laboratory preparation of samples, for example in a gyratory compactor, may permit excess fluid to drain out through weep-holes, allowing good compaction to be achieved. This may not be reflected in experience on site.

Curing

Once compacted into the pavement, the process of strength gain known as 'curing' commences. Depending on the ambient conditions, the water content may decrease rapidly due to evaporation, but this in itself is insufficient to generate good binder–aggregate adhesion. However, over a period of time, commonly between one and six months, the barrier to adhesion due to the remaining water film around the aggregate particles steadily breaks down and binder–aggregate bond improves. Logically the droplets of binder must also work their way under gravity to particle contact locations, where they are retained due to surface tension effects, and the result is a steady increase in strength and stiffness. Serfass *et al.* (2003) discuss the curing process. **Figure 6** presents examples of stiffness gain data for foamed bitumen bound materials.

Mixture design

A key difference in designs for cold asphalt mixtures compared to hot is that an additional component has to be allowed for, namely water. Cold aggregate inevitably contains water and cold mixtures need water in order for bitumen droplets to disperse within the mixture during mixing. Furthermore, an optimum quantity of low-viscosity liquid is needed to allow good compaction. The upshot is that water is an *absolutely essential* ingredient in the mixture.

But if there is water in the mixture then, compared with a hot mix, there has to be less of something else – and that usually means less bitumen. Take, for example, an idealised dense hot-mix asphalt, with 85% of the final volume taken up by aggregate, 10% by bitumen (equivalent to 4–5% by mass) and 5% by air voids. If the same aggregate gradation is used in a cold mix then, in order to achieve good compaction, there still needs to be about 10% fluid content by volume. This could probably be pushed up to around 13%, but it is dangerous to stray too far from optimum fluid content if maximum dry density is to be achieved; it is impossible to squeeze the air void content down to less than about 2% during compaction. However, even 13% fluid by volume is only 5–6% by mass, at least half of which needs to be water in order to facilitate binder mixing. This leaves no more than 2–3% added binder (by mass), commonly no more than about 2% residual bitumen. Bearing in mind that 3.5% is about the lowest binder content commonly used in hot-mix asphalt, 2% is unlikely to be enough to bind the particles together properly – and it leaves cold mix designers with an insurmountable problem: it is *physically impossible* to replicate a well-compacted dense asphalt using conventional aggregate and cold-mix technology; there isn't enough space in the mixture.

The following are the alternative approaches that may be considered (illustrated in **Figure 7**):

- Accept a low bitumen content. Make sure that the filler content is also correspondingly low or contact between bitumen and large aggregate will be poor, resulting in poor mixture properties. The result will be a stiff and deformation-resistant mixture, but one with low durability and fatigue resistance.
- Increase the bitumen content to typical hot-mix asphalt proportions. This means that the total non-aggregate volume (bitumen, water and air) will have to increase significantly, reducing stiffness and deformation resistance. Fatigue and durability, however, should be satisfactory.
- Change to a more open gradation with a naturally higher void content at full compaction. This allows a higher bitumen percentage to be used but also brings a reduced stiffness and deformation resistance in comparison with a dense gradation.

Unfortunately, the presence of water during compaction means there is no way to avoid a relatively high air void content once the water has evaporated. It is tempting to try to produce a direct equivalent to hot mix, using identical gradation and bitumen content, but this is a dangerous road to take; the aggregate skeleton will be less efficient than that in a hot mix because of the lower aggregate density achieved due to the presence of water. Not only so, but there is a danger that the very low air voids left immediately after compaction will inhibit water evaporation, and unless evaporation takes place the result will be a very poor material indeed. Ibrahim and Thom (1997) present data relevant to emulsion mixture design.

The use of an open gradation is the safest option because the high air void content encourages evaporation; however, the resulting mixture will have a relatively low stiffness.

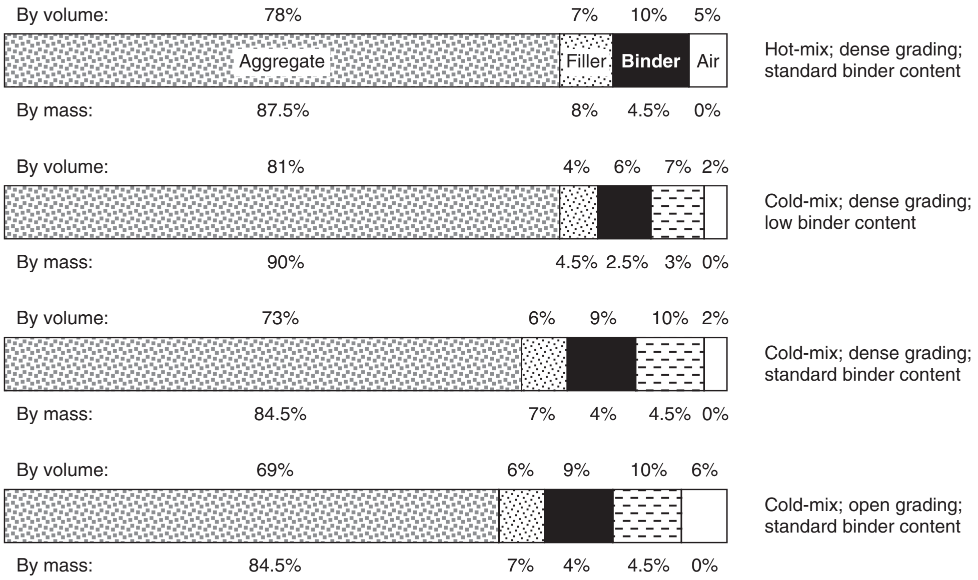

Figure 7 Alternative cold-mix asphalt volumetrics (Thom, 2008)

Addition of cement or lime

One of the techniques that cold-mix asphalt producers commonly use to enhance their materials is to add a small percentage (typically 1–2%) of Portland cement, lime or other hydraulic binder (such as fly ash) to the mixture. During the early part of the curing process, the hydraulic binder reacts with some of the water present, thereby reducing the water content in the mixture. This is helpful as it tends to accelerate the curing process by removing water that would otherwise have inhibited the formation of bond between aggregate and bitumen. The result is that the material reaches its full design strength and stiffness significantly earlier than would otherwise have been the case. There is also the added advantage that the hydraulic binder actually creates bond between pairs or clusters of particles and this has the effect of further stiffening the mixture. **Figure 6** shows three examples of cold asphalt mixtures with additives; the early advantage compared with the no-additive case is clear, as is the difference between cement and lime – in this example at least.

There is some debate as to how much hydraulic binder to add, however, because the higher the ratio of hydraulic to bituminous bonds the more brittle the behaviour will become, i.e. the more sensitive it is to overloading. This may be seen in the fatigue characteristics shown in **Figure 8**, derived for foamed bitumen mixtures.

Use of recycled asphalt pavement (RAP)

An alternative way to increase the binder content within a cold asphalt mixture is to incorporate recycled asphalt pavement (RAP) as part of the aggregate. For example, if 50% of the aggregate comprised RAP and the binder content of the RAP was 5%, this will increase the overall binder content of the mixture by around 2.5%. The key, however, is to be able to activate the binder in the RAP. It will usually have aged over a period of many years and will therefore be much harder than the new emulsion or foamed bitumen binder, and RAP in this state it is often referred to as 'black rock'. Nevertheless, with appropriate addition of chemical rejuvenators, it has been shown that it is possible to induce a degree of blending between old and new binder and therefore to enhance mixture properties (e.g. Walter, 2002).

With or without any performance enhancement from the binder in RAP, cold-mix technology represents a natural choice for recycling of materials in general, including RAP. This is discussed in subsequent sections.

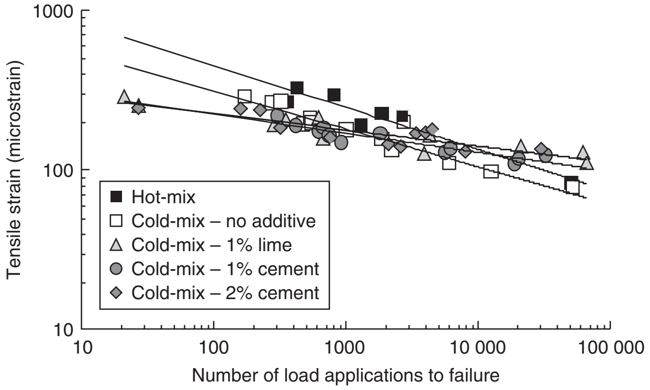

Figure 8 Fatigue characteristics of cold asphalt mixtures (Jitareekul, 2009)

Applications – in-plant mixing

Cold asphalt mixtures have two very significant advantages over hot-mix asphalt, namely:

1. The energy consumption of the plant is typically 10–20% that for hot-mix production.
2. Cold asphalt mixtures can be stored for long periods before use.

The first point leads to clear economic and environmental benefits, offsetting the loss of performance when compared to hot mixtures. The second point means that cold asphalt mixtures make ideal patching and repair materials. There is a long workability period following placement during which compaction can be achieved, so trench backfills and other small areas of construction can be completed to a high standard.

Because of the long workability period, materials can also be stockpiled, for weeks if necessary, which increases the flexibility in plant operation hours, further enhancing the economic advantages of cold asphalt mixtures. However, this advantage reduces significantly for mixtures that contain hydraulic binder, which in practice means for the majority of paving course mixtures. If the hydraulic binder is relatively slow acting (lime and fly ash) then there is still a greatly extended workability period compared to hot asphalt, which means the planning of production, transportation, paving and compaction is still reasonably flexible, but the mixture will have a definite ‘use-by’ time.

The arrival of cold-mix technology has spawned a new term, ‘ex-situ’. This refers to the practice of setting up a mobile mixing plant at or near to a work site, which takes recycled materials from the site and incorporates them into a new mixture. The relative simplicity of cold-mix production means that this is an economically attractive option, saving on transport and making reuse of materials practical.

Whether cold-mix asphalt is suited to use in a particular layer within a pavement depends on material quality in relation to traffic intensity. The following possibilities exist:

- Sub-base: cold asphalt mixtures will perform a similar function to a hydraulically stabilised sub-base, giving a high class foundation for all categories of pavement.
- Base: this is the more usual application, although the category of pavement to which cold-mix asphalt is suited depends on the mixture used; in particular the addition of cement brings cold asphalt mixtures into direct competition with hot asphalt with the option of use on more heavily trafficked pavements.
- Surfacing: thin cold-mix asphalt surfacing layers are widely used, sometimes incorporating modified binder to enhance properties since the stresses that the mixture has to withstand are high.

In practice, the use of cold asphalt mixtures in bases is almost all on relatively lightly trafficked roads because of the need for the material to cure and the danger of damage during early life.

Applications – in-situ recycling

This is a large subject and one that is likely to become increasingly important in the future. The concept is that an existing pavement is milled to a certain depth – machines with capability up to 350 mm exist – and that the broken fragments are then mixed with a new binder and compacted – see **Figure 9**. The result, if design and construction are carried out sensibly, is a strong new pavement, although it will normally be necessary to apply a new surface course since it is unlikely that the requisite surface properties will be present from the recycled material itself.

The potential is clear; no matter how cracked, deformed, worn, aged, etc. the existing pavement is, so long as its broken remains fulfil the role of aggregate, then it is still useful. There is no need for expensive and disruptive transportation of materials (other than binder); there is no requirement for quarrying of virgin rock. All that is needed is the addition of an effective binder. Naturally there are difficulties, and the following are seen as the major issues confronting practitioners and designers.

- Binder is unlikely to be uniformly mixed into the material.
- All pavements are to a certain extent non-homogeneous, notably in layer thickness. In many cases this means that the milled material will present varying proportions of asphalt, base (including hydraulically bound materials), sub-base and even subgrade.
- Water content can only be controlled approximately.
- Aggregate gradation is likely to be low in fines content if the RAP percentage is high.
- Compaction beyond about 250 mm depth will be increasingly ineffective.

Practical advice can be found in Wirtgen’s *Cold Recycling Manual* (Wirtgen, 2004); advice specific to use of emulsions is given by Lillbroända (1997).

Material variability

The first three points in the above list relate to variability. Variability is an issue that cannot be dismissed – nor can

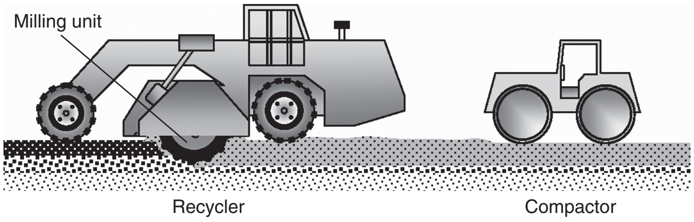

Figure 9 Cold in-situ recycling (Thom, 2008)

technical improvements to the process ever fully overcome it. A key question is to what extent short-wavelength fluctuations in quality will affect performance. It could be argued that, over a thickness of 250 mm or more, deformation susceptibility might be expected to even out. Similarly, cracking could not advance unless it propagated through both good and poor regions of the material. These are reasonable arguments and they suggest that apparent variability from tests on cores may not necessarily be cause for alarm. On the other hand, permeability is critically affected by small changes in gradation and void content, which suggests that an in-situ recycled material will always be relatively permeable since there will always be high-void routes through it.

In conclusion, the following advice is offered:

- Reduce the design stiffness modulus from that derived from laboratory-mixed specimens to take account of variability.
- Be aware that water will be able to ingress. This means that the durability of the recycled material under wet conditions should be assessed and that the likelihood of water reaching the foundation should be recognised.

Compaction

Modern compaction equipment is able to achieve high density in a granular pavement material such as a base or sub-base to a thickness of around 250 mm, and cold-mix recycled materials are effectively granular at the time of compaction. Beyond that depth, the relative density will decrease. To take account of this, it is sensible practice to ignore a proportion of the material thickness lower than about 250 mm from the surface.

Mixture details

The gradation of RAP is quite different from the gradation of the original asphalt mixture, since it consists of agglomerates rather than individual particles, and the key difference is a much reduced fines content. This means that if asphalt represents a significant proportion of the recycled depth then the overall grading of the mixture will be low in fines – and therefore high in void space. The result is a rather lower stiffness modulus and reduced deformation resistance in comparison with a more broadly graded mixture. However, since curing (i.e. strength gain) is critically dependent on water loss from the material, this means that open-graded materials have an advantage in that they allow air to access the voids and hasten evaporation. The inclusion of hydraulic binder to reduce the free water content and encourage curing is a sensible precaution, since the water content cannot be accurately controlled. Clearly, the timing of surfacing application is another important factor in the equation and a responsible organisation will be careful to take account of the weather, not applying the surface immediately after rain has raised the water content of the recycled layer for example.

Pavement design issues

A cold-mix asphalt is still an asphalt; that is to say it is still a mixture of aggregate particles and bitumen and the bitumen still acts to bind the particles together. However, as outlined above, the demands of cold-mix technology mean that:

- the material takes up to 6 months to achieve its design strength and stiffness;
- the void content will be higher than that of an equivalent hot-mix;
- the distribution of binder will be non-uniform, giving reasonable binding action amongst finer particles but reduced binding between larger particles;
- the resulting stiffness and deformation resistance will often be lower than that of an equivalent hot-mix asphalt.

Early-life damage

It is unlikely to be economic to avoid use of a pavement during its curing period yet, if it is used, then structural damage has to be expected, even while curing is taking place. The unknown quantity is the extent to which such damage will self-heal during subsequent curing, and also how long the curing process takes. The advice contained in **Table 1**, relating to cold-mix base layers, should therefore be seen as indicative only. It is primarily based on small-scale experimental data obtained at the University of Nottingham and assumes that normal traffic levels are allowed onto the pavement within a few days of the layer being placed. The intention is that a long-term stiffness modulus can be estimated, which can then be used in pavement design. Note that actual behaviour will also be a function of the details of pavement construction, in particular the thickness of overlying hot-mix asphalt layers.

The clear conclusion is that cold-mix bases are best suited to low-traffic locations – unless early-life traffic can realistically be either excluded or greatly restricted.

Design approaches

There are basically two approaches to pavement design against cracking where cold-mix asphalt is involved; either the material can be treated in the same way as a hot-mix (which implies that early-life damage can be

Time to 90% of design strength: days	Suggested design stiffness (% of fully cured undamaged)
28	60
56	40
90	30
180	20

Table 1 Early-life trafficking of cold-mix recycled layers (Thom, 2008)

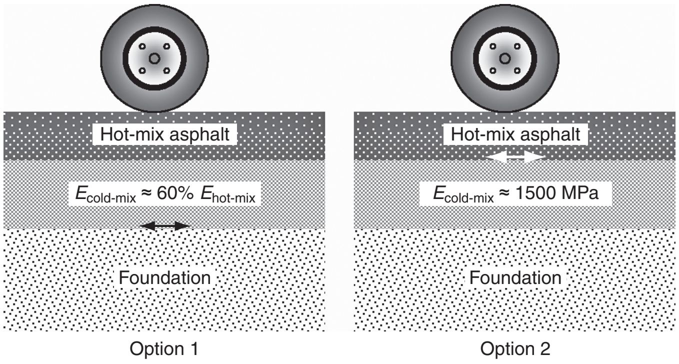

Figure 10 Design with cold-mix asphalt base (Thom, 2008)

neglected), or it can be considered as an ultra-high quality granular material. If considered in the same way as a hot-mix (Option 1 in **Figure 10**), then design against cracking is required. Since the stiffness will usually be lower than that of hot-mix while the fatigue resistance will probably be similar (at a given level of strain), this means that the design thickness will typically be greater. The estimate of 60% of the stiffness of an equivalent hot-mix, i.e. one with the same grading, binder content and grade, is based on the assumption that the void content of the cold-mix will be some 5% higher; where hydraulic binder is included the stiffness may be significantly greater than this. However, if the material stiffness really was 60% that of hot-mix, the cold-mix design thickness in Option 1 would be some 25% greater than the equivalent hot-mix thickness. Current UK practice, when using cold-mix asphalt as a base layer, follows Merrill *et al.* (2004), which assigns a stiffness (at 20°C) of 2500 MPa for plant-mixed cold asphalt material and 2000 MPa for in situ-mixed. The resulting design thicknesses are slightly higher than those using a 125 pen hot-mix Dense Bitumen Macadam base.

The advantage of treating cold-mix as a high-quality granular material (Option 2 in **Figure 10**) is that no design against cracking is required. It is assumed, in effect, that cracking (in the form of missing inter-particle bonds) has taken place already. The 1 500 MPa shown in design Option 2 is intended to be a reasonable long-term value accepting that trafficking during the first few months of life will limit the final stiffness achieved and that a significant number of micro-cracks or missing inter-particle bonds will be present. The figure is a little lower than would probably be found as an average for the layer since it is intended to represent the least intact locations. The actual stiffness applying will also depend on mixture details. Highways Agency (2006) foundation designs effectively adopt this approach, in which cold-mix asphalt could logically form the sub-base element in a Class 3 or Class 4 (i.e. high class) foundation, allowing reduced upper pavement thickness.

Despite an understandable desire to achieve hot-mix equivalence, it is advised here that Option 2 is the safer, and often the more realistic. By nature, a cold-mix has discontinuities in it which are not present in a hot-mix, due to unequal distribution of binder, residual water films and early-life trafficking, and it seems sensible to acknowledge this in design. As it happens, the resulting design thickness is often no greater when using the Option 2 approach than for Option 1.

With respect to design against rutting however, it is advised that hot-mix principles are followed. Either a wheel-tracking test or a repeated load axial test will give a comparative measure of deformation resistance. It will usually be lower than for a hot-mix because of the higher void content which, depending on traffic levels, often dictates that a hot-mix surfacing is used.

Warm mixtures

The technologies involved in creating what are known as warm mixtures are quite different from those involved in cold-mix asphalt. The only real similarity is that both address the issue of reducing energy consumption in asphalt. Warm mixtures are, in effect, hot mixtures that can be created at lower temperatures than is conventionally possible, but the production process is essentially that for hot-mix asphalt. They are created in two basic ways, either by altering the hot-mix production process or by adding an ingredient to change the properties of the bitumen during mixing, placing and compaction.

Changes to the production process

Different producers have differing techniques, but they all tend to make use of two separate binders rather than a single grade, taking advantage of the temperature reduction possible when working with a soft bitumen. Generally, a soft bitumen is mixed with the aggregate and a much harder bitumen is then added during the second half of the mixing phase; the mix temperature, however, is that appropriate to the softer bitumen, which can be as much as 30°C lower than would have been used for a conventional grade binder. The result is that the aggregate is fully coated (by the soft bitumen) and the long-term properties will be those expected from an intermediate grade of binder as the two components coalesce within the mixture. See, for example, Larsen *et al.* (2004).

It is also possible to use the foamed bitumen technique to introduce the harder component into the mixture. This has the advantage of giving additional fluidity to the bitumen during the mixing, laying and compaction processes because of the residual inclusions of air and water contained within the bitumen. The expectation is that these inclusions disappear fairly rapidly following compaction.

Additives to the binder

There are two classes of additive that are used to enable increased fluidity of bitumen at mixing, laying and compacting temperatures. The first is an aerogenous agent, a chemical that induces small bubbles to appear within the binder in a similar way to air-entraining agents in concrete, and current usage appears to be restricted to a mineral known a zeolite. Zeolites are crystalline hydrated aluminium silicates and when they are added into the mixture at the same time as bitumen they induce microscopic pores. Conceptually these pores are not unlike the bubbles created in foamed bitumen and they have a similar influence on fluidity and therefore compactability of the mixture, although not on quite such a dramatic scale. Zeolite inclusion can allow a similar lowering of mix temperature to that from the use of two separate binders, i.e. by about 30°C (Barthel *et al.*, 2004). The theory is that the pores disappear in the long term, returning the binder to its original state.

The second class of additive is paraffin (Richter, 2001). The inclusion of a selected paraffin can create a special bitumen with the properties of conventional binder at in-service temperatures but an increased workability at temperatures above about 80°C. This is because the paraffin element crystallises at lower temperatures, increasing viscosity, but remains mobile at higher temperatures, enhancing workability. It is claimed that temperature reductions of up to 40°C are possible with these types of binder.

Applications

There are three main reasons for using warm asphalt mixtures, namely:

- to reduce energy consumption during production;
- to increase the time available for laying and compaction;
- to increase the percentage of recycled asphalt pavement (RAP) used in the mixture.

An increase in the RAP percentage, typically to at least 30%, is possible because of the reduced mixing temperature, meaning that it is unnecessary to heat the RAP as much as in a conventional hot-mix asphalt.

Summary

Both cold and warm asphalt technologies are now well established, although continuing advances in such matters as emulsion chemistry, additives for warm mixtures, etc. mean that the resultant materials are likely to continue to improve in terms of their achievable performance. Bearing in mind the challenges presented by the need to conserve energy and maximise the use of marginal and recycled materials, it seems inevitable that use of both types of mixture will continue to increase. However, if use is to increase then the key need now is for design assurance to develop with it. The opening up of asphalt to a much increased range of products with differing levels of performance means that the design approaches used need to be flexible enough to cope, and this is not the case where designs are based on empirical rules derived from past experience.

The challenge for the industry therefore lies not so much in the technology of the materials themselves but in developing improved methodologies for their application. For example, it seems unlikely that the same level of assurance can ever be given for cold mixtures as for conventional asphalts, and that level of assurance can only decrease when recycled materials are incorporated, either fully or partially. This means that design guides have to include the facility to take account of this when determining the appropriate layer thickness, foundation quality, etc. to specify. Once this is in place, both cold and warm asphalt mixtures will have an expanding role, particularly in pavement rehabilitation work.

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CONTENTS

Chapter 29

Mechanical models for simulating bitumen and asphalt behaviour

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The objective of this chapter is to introduce some simple models that can be used to simulate the behaviour of bitumen and/or asphalt. Linear viscoelastic models are able to model deformation (both recoverable and irrecoverable). A linear spring and a linear dashpot are used to represent elastic (recoverable) and viscous (irrecoverable) behaviour respectively and a linear spring connected in parallel to a linear dashpot used to represent delayed (recoverable) elastic behaviour. The response to creep loading, where a load is suddenly applied and held constant, is investigated and a method is introduced to deal with more generalised loading. Linear Elastic Fracture Mechanics and the Paris law are introduced to model crack propagation from an initial crack or flaw. A simple laboratory test is discussed as a means for obtaining Paris law parameters and a simple method is developed for applying the results to crack propagation in a pavement structure. The first two approaches treat the asphalt as a continuum, whereas the third approach recognises its particulate nature. Discrete element modelling is used to simulate the behaviour of an asphalt at the scale of an aggregate particle. The method treats the asphalt as an agglomeration of particles bonded together. The bond stiffness is time dependent to give viscoelastic behaviour and bond breakage is allowed to simulate softening as the material is progressively damaged.

Introduction

Asphalt is a complex multi-phase material comprising bitumen, graded mineral aggregate and air. A range of asphalt types and compositions can be produced depending on the types and proportions of these components and on the aggregate grading. A typical continuously graded mixture (e.g. asphalt concrete) relies on an interlocking aggregate skeleton for its strength with the binder primarily acting as a lubricant to aid compaction and 'glue' the mixture together. At the other extreme, a typical gap-graded mixture (e.g. stone mastic asphalt) will have a discontinuous aggregate grading (i.e. some stone sizes will not be present) and relies on a coarse aggregate skeleton bound by a bitumen/filler 'mortar' for its strength. It has been shown that for both asphalt mixture types the micromechanical behaviour, at the scale of an aggregate particle, is an important factor in terms of overall material performance (Collop *et al.*, 2006).

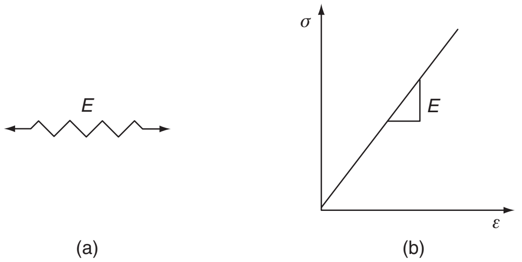
The traditional approach to modelling damage in asphaltic materials is to treat them at the macro-scale using continuum-based models (Blaauwendraad and Scarpas, 2000; Erkens *et al.*, 2002; Collop *et al.*, 2003). This usually involves undertaking careful experiments over a range of conditions (e.g. stress levels, loading rates, temperatures, etc.), measuring the macroscopic response of the material and fitting continuum-based constitutive models (which can be extremely complex) to the measured behaviour. One of the major limitations with all the continuum approaches is that the micromechanical behaviour of the mixture is not explicitly included in this approach which means that it is not easy to relate observed behaviour to the micromechanics of the material.

Micromechanical models have been applied to rigorously analyse the damage behaviour of cementitious materials since the early 1990s. This has been addressed using various numerical techniques, such as the finite-element method, discrete element modelling and the boundary element method (see Kim *et al.*, 2007 for a comprehensive literature review). However, most studies conducted to date have been limited to modelling stationary cracks in 2D.

This chapter discusses three simple approaches that can be used to predict bitumen and asphalt behaviour both in terms of deformation and fracture/fatigue. Linear viscoelasticity and linear elastic fracture mechanics are continuum-based approaches where the particulate nature of asphalt is not explicitly considered, whereas discrete-element modelling simulates movements of individual particles.

Linear viscoelasticity

A linear viscoelastic model is the simplest way of explicitly introducing time-dependent behaviour. Probably the easiest way of visualising this type of model is to think of combinations of linear springs and dashpots (dampers).



(a) (b)

Figure 1 Linear spring

These basic elements can be considered to be the building blocks for viscoelastic models of bitumen and asphalt behaviour.

Linear spring

A simple linear spring is shown schematically in **Figure 1(a)**. When a load (or stress) is applied there is a displacement (or strain) that depends on the stiffness of the spring and, when the load is removed, the spring recovers its original length. Mathematically, this can be expressed as:

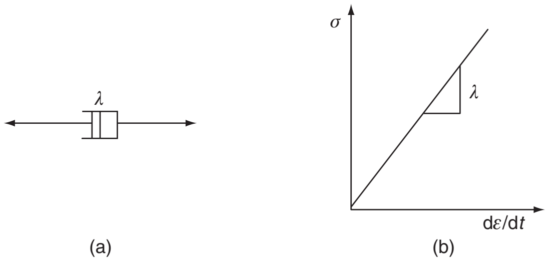
$$\varepsilon = \frac{\sigma}{E} \tag{1}$$

where σ is the stress (force divided by cross-sectional area), ε is the strain (change in length divided by original length) and E is the stiffness modulus. The relationship between stress and strain defined by Equation (1) is plotted schematically in **Figure 1(b)**. This element can be considered to represent elastic behaviour.

Linear dashpot

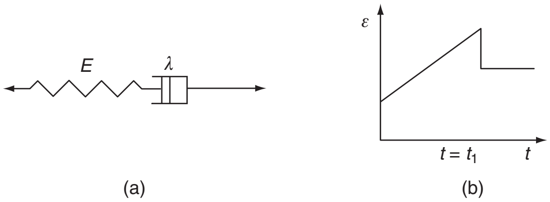
A linear dashpot is shown schematically in **Figure 2(a)**. This element can be considered to represent viscous behaviour. When a load (or stress) is applied, the dashpot increases in length at a constant rate that depends on the viscosity of the dashpot and, when the load is removed, there is no recovery and the dashpot remains permanently extended. Mathematically, this can be expressed as:

$$\dot{\varepsilon} = \frac{\mathrm{d}\varepsilon}{\mathrm{d}t} = \frac{\sigma}{\lambda} \tag{2}$$



(a) (b)

Figure 2 Linear dashpot



(a) (b)

Figure 3 Maxwell element

where $\dot{\varepsilon}$ is the strain rate and λ is the viscosity. The relationship between stress and strain rate, defined by Equation (2), is plotted schematically in **Figure 2(b)**.

Maxwell element

A Maxwell element comprises a spring and a dashpot connected in series, as shown in **Figure 3(a)**. Consider the response of this element when subjected to creep loading where a stress is applied rapidly and then kept at a constant level (σ_0). It can be seen from **Figure 3(a)** that, because the spring and the dashpot are connected in series, the strains (displacements) are additive and the stress (force) is equal across both elements. This gives:

$$\varepsilon(t) = \varepsilon_{\text{spring}} + \varepsilon_{\text{dashpot}} \tag{3}$$

The strain across the spring in Equation (3) can be directly obtained using Equation (1). To obtain the strain across the dashpot, Equation (2) needs to be integrated which gives:

$$\varepsilon(t) = \frac{\sigma_0}{\lambda}\int_0^t \mathrm{d}t = \frac{\sigma_0 t}{\lambda} \tag{4}$$

Substituting Equations (1) and (4) into (3) gives:

$$\frac{\varepsilon(t)}{\sigma_0} = \frac{1}{E} + \frac{t}{\lambda} = J(t) \tag{5}$$

$J(t)$ is referred to as the creep compliance and is a material property. It can be seen from Equation (1) that the creep compliance is the inverse of the stiffness.

The strain from Equation (5) is shown plotted in **Figure 3(b)**. It can be seen from this figure that it comprises an instantaneous elastic component (σ_0/E) governed by the stiffness of the spring and a viscous component ($\sigma_0 t/\lambda$) which accumulates in proportion to time at a rate governed by the viscosity of the dashpot.

If the load is suddenly removed at time $t = t_1$, the spring will instantaneously recover its deformation and the dashpot will remain extended giving a permanent strain of $\sigma_0 t_1/\lambda$.

Kelvin element

A Kelvin element comprises a spring and a dashpot connected in parallel, as shown in **Figure 4(a)**. Consider the response of this element when subjected to the same creep loading as for the Maxwell element. It can be seen from **Figure 4(a)** that, because the spring and the dashpot

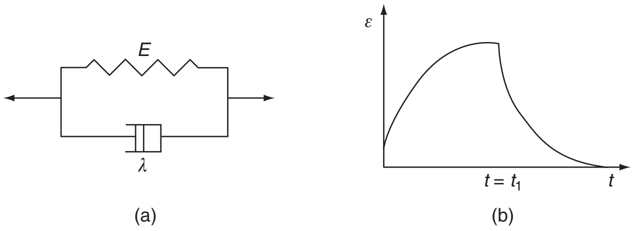


Figure 4 Kelvin element

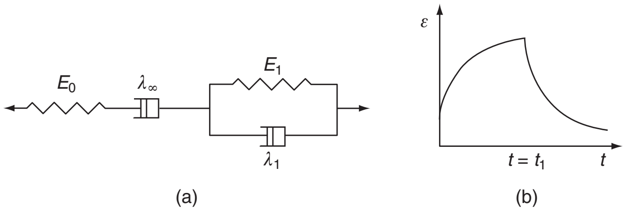


Figure 5 Burger's model

are now connected in parallel, the stress (force) is additive and the strain (displacement) is equal across both elements. This gives:

$$\sigma_0 = \sigma_{\text{spring}} + \sigma_{\text{dashpot}} \tag{6}$$

As with the Maxwell element, the stress across the spring can be directly obtained from Equation (1). The stress across the dashpot can be obtained by substituting Equation (2) into Equation (6) giving:

$$\sigma_0 = E\varepsilon + \lambda\dot{\varepsilon} \tag{7}$$

This is a first-order differential equation in terms of strain, which can be shown to have the following solution:

$$\frac{\varepsilon(t)}{\sigma_0} = \frac{1}{E}(1 - \mathrm{e}^{-t/\tau}) = J(t) \tag{8}$$

where τ, which is known as the relaxation time, is given by λ/E. The strain from Equation (8) is plotted in **Figure 4(b)**. It can be seen from this figure that the strain increases exponentially from zero as a function of time towards a maximum value of σ_0/E.

If the load is suddenly removed at time $t = t_1$, all the deformation will be gradually recovered due to the presence of the spring pulling the dashpot back to its original position. Mathematically, it can be shown that this is given by:

$$\frac{\varepsilon(t > t_1)}{\sigma_0} = \frac{\mathrm{e}^{-t/\tau}}{E}(\mathrm{e}^{-t_1/\tau} - 1) \tag{9}$$

It can be seen from Equation (9) that as t gets large, the strain returns to zero. This behaviour is termed delayed elasticity.

Burger's model

It can be seen from the discussion above that combining the Maxwell element in series with a Kelvin element (see **Figure 5(a)**) will result in initial elasticity and permanent deformation due to the Maxwell element and delayed elasticity due to the Kelvin element. This is known as the Burger's model. Since in a series model the strains are additive, the response of the Burger's model to creep loading can simply be obtained by adding Equations (5) and (8) giving:

$$\frac{\varepsilon(t)}{\sigma_0} = \frac{1}{E_0} + \frac{t}{\lambda_\infty} + \frac{1}{E_1}(1 - \mathrm{e}^{-t/\tau}) = J(t) \tag{10}$$

where $\tau = \lambda_1/E_1$. The strain from Equation (10) is plotted in **Figure 5(b)**. If the load is suddenly removed at time $t = t_1$, the deformation in the Kelvin element will be gradually recovered due to the presence of the spring pulling the dashpot back to its original position. The spring in the Maxwell element will instantaneously recover its deformation and the dashpot in the Maxwell element will remain extended giving a permanent strain of $(\sigma_0 t_1/\lambda)$. Mathematically, it can be shown that this is given by:

$$\frac{\varepsilon(t > t_1)}{\sigma_0} = \frac{t_1}{\lambda_\infty} + \frac{\mathrm{e}^{-t/\tau}}{E_1}(\mathrm{e}^{-t_1/\tau} - 1) \tag{11}$$

The Burger's model can easily be generalised to include more than one relaxation time (and thus more complicated behaviour) by simply adding additional Kelvin elements in series. Consequently, Equations (10) and (11) can be generalised to give:

$$\frac{\varepsilon(t)}{\sigma_0} = \frac{1}{E_0} + \frac{t}{\lambda_\infty} + \sum_{i=1}^{N} \frac{1}{E_1^i}(1 - \mathrm{e}^{-t/\tau^i}) = J(t) \tag{12}$$

$$\frac{\varepsilon(t > t_1)}{\sigma_0} = \frac{t_1}{\lambda_\infty} + \sum_{i=1}^{N} \frac{\mathrm{e}^{-t/\tau^i}}{E_1^i}(\mathrm{e}^{-t_1/\tau^i} - 1) \tag{13}$$

where $\tau^i = \lambda_i/E_i$ and N is the number of Kelvin elements.

Hereditary integral

So far, only creep loading has been considered where a stress is rapidly applied to the material and then maintained at a constant level. While this is useful for interpretation of results from creep tests undertaken in the laboratory, it is helpful to consider the response to more generalised loading. The viscoelastic models dealt with here are linear so the rule of linear superposition can be applied to calculate the response due to several loads. For example, the response to creep loading applied at time $t = 0$ has been shown above to be given by $\varepsilon(t) = \sigma_0 J(t)$. If now an additional stress $\Delta\sigma'$ is applied at time $t = t'$ an additional strain would be produced that is proportional to $\Delta\sigma'$ and which depends on the same creep compliance but with time starting at $t = t'$. Mathematically this is given by:

$$\varepsilon(t) = \sigma_0 J(t) + \Delta\sigma' J(t - t') \tag{14}$$

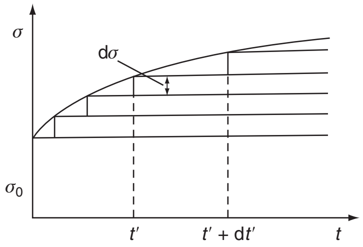


Figure 6 Hereditary integral

Assume now that a stress σ_0 is applied at time $t = 0$ which then varies as an arbitrary function of time as shown in **Figure 6**. It can be seen from this figure that the arbitrary variation can be considered to comprise a sequence of infinitesimal steps $d\sigma'$ which means that Equation (14) can be generalised to give:

$$\varepsilon(t) = \sigma_0 J(t) + \int_0^t J(t - t') \frac{d\sigma'}{dt'} dt' \tag{15}$$

Equation (15) is known as a hereditary integral and shows that the strain at time t depends on the entire stress history which is quite different from an elastic material where the strain at time t depends only on the stress at time t. It can be shown that Equation (15) can be integrated by parts to give the following alternative, often useful, form:

$$\varepsilon(t) = \sigma(t) J(0) + \int_0^t \sigma(t') \frac{dJ(t - t')}{d(t - t')} dt' \tag{16}$$

where $J(0)$ is the creep compliance at time $t = 0$.

For example, assume that a sinusoidal stress of the form $\sigma(t) = \sigma_0 \sin \omega t$ is applied and the strain response is required. Using Equation (16), it can be shown that this is given by:

$$\frac{\varepsilon(t)}{\sigma_0} = \frac{1}{E_0} \sin \omega t + \int_0^t \sin \omega t' \left(\frac{1}{\lambda_\infty} + \frac{1}{\lambda_1} e^{-(t-t')/\tau} \right) dt' \tag{17}$$

Evaluating the integral in Equation (17) and neglecting terms that are due to the initial conditions gives the steady-state response as:

$$\frac{\varepsilon(t)}{\sigma_0} = \left[\frac{1}{E_0} + \frac{1}{E_1(1 + \omega^2 \tau^2)} \right] \sin \omega t + \left[-\frac{1}{\omega \lambda_\infty} - \frac{\omega \lambda_1}{E_1^2 (1 + \omega^2 \tau^2)} \right] \cos \omega t \tag{18}$$

Fit to experimental data

An example of Equation (10) fitted to data from a creep test on a dense bitumen macadam at 20°C and a stress level of 1 MPa, is shown in **Figure 7**. The fitted parameters are: $E_0 = 300$ MPa, $\lambda_\infty = 60$ GPa s, $E_1 = 140$ MPa, $\lambda_1 = 6$ GPa s. It can be seen from this figure that the fit is reasonably good apart from the region after approximately 1200 s. This is due to damage accumulating in the specimen resulting in an accelerating strain rate which is not included in the model. A similar fit could have been obtained at different temperatures and strain rates. However, this would result in different values of the fitted constants indicating that they are a function of both temperature and stress level.

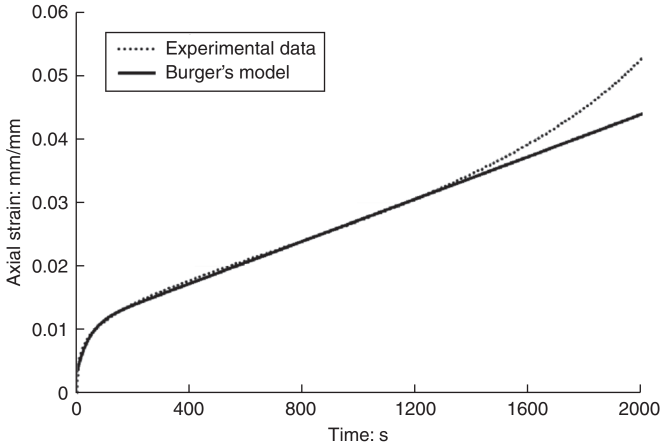


Figure 7 Fit to experimental data

Non-linearities

Previous research has shown that the steady-state behaviour of a range of bitumens and asphalt mixtures is complex, ranging from linear viscous at low stress levels and power law creep at high stress levels with a creep exponent of between 2 and 2.5 (Cheung and Cebon, 1997; Deshpande and Cebon, 1999; Collop and Khanzada, 1999). It has also been observed that the steady-state behaviour (in both the linear and non-linear regions) depends on the level of confining pressure (in a triaxial test) and temperature and during the latter stages of a creep test the strain rate can increase dramatically as the specimen is progressively damaged (Deshpande and Cebon, 1999; Collop and Khanzada, 1999).

Collop *et al.* (2003) introduced a model of the following form for determining the equivalent viscosity as a function of the stress conditions, temperature, degree of confinement and damage:

$$\begin{aligned} \lambda_\infty &= \lambda_{\text{uni}}(T) \left(\frac{\sigma_e}{\sigma_0} \right)^{1-n} 10^{B(\eta + 1/3)} (1 - D)^n \quad (\sigma_e > \sigma_0) \\ &= \lambda_{\text{uni}}(T) 10^{B(\eta + 1/3)} (1 - D)^n \quad (\sigma_e \leqslant \sigma_0) \end{aligned} \tag{19}$$

where σ_e is the Von Mises equivalent stress ($\sigma_e = \{3/2 s_{ij} s_{ij}\}^{1/2}$), s_{ij} is the deviatoric stress tensor

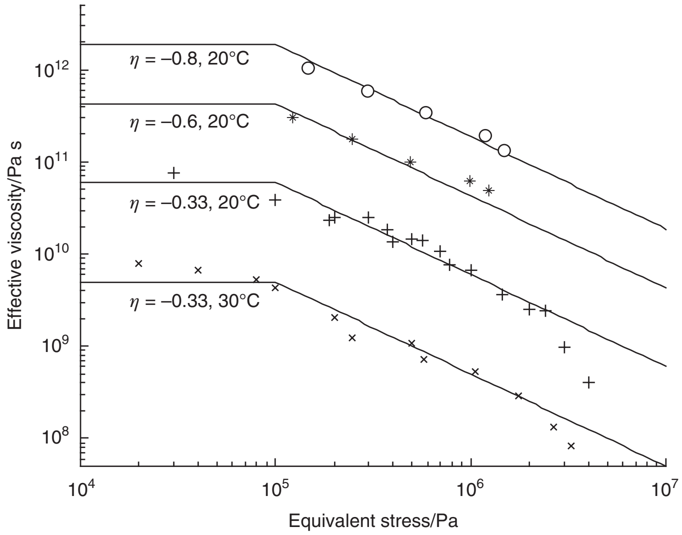


Figure 8 Experimental viscosity data

$(s_{ij} = \sigma_{ij} - 1/3\delta_{ij}\sigma_{kk})$, δ_{ij} is the Kronocker delta, p is the mean stress $(p = 1/3\sigma_{kk})$, η is the stress ratio $(\eta = p/\sigma_e)$, λ_{uni} is the uniaxial viscosity measured from a uniaxial compression test (where $\eta = -1/3$), T is temperature, n, σ_0, B, are material constants and D is a damage variable that ranges between 0 (undamaged) and 1 (fully damaged). The evolution equation for the damage variable D was taken to be:

$$\frac{\mathrm{d}D}{\mathrm{d}t} = \left(\frac{\sigma_e}{\tilde{C}}\right)^{(n+1)/2} \frac{1}{(1-D)^{(n+1)/2}} \tag{20}$$

where $\tilde{C}$ is a material constant. **Figure 8** shows the fit from Equation (19) (solid lines) to experimental data for an idealised bituminous mixture (sand asphalt) tested by Collop and Khanzada (1999). The fitted material parameters are: $\lambda_{\text{uni}}^{20C} = 60\,\text{GPas}$, $\lambda_{\text{uni}}^{30C} = 5\,\text{GPas}$, $\sigma_0 = 100\,\text{kPa}$, $n = 2$ and $B = -3.2$ (note that D has been taken to be zero). It can been from this figure that Equation (19) models the behaviour reasonably well, although the temperature and stress ratio ranges are somewhat limited. It should also be noted that, although Equation (19) is in a form where it can be applied to any stress state, the experimental data from which it was derived were for only negative stress ratios (i.e. compressive mean stress).

Fracture mechanics

One of the simplest theories relating to crack growth is linear elastic fracture mechanics (LEFM). This approach originated with Griffith (1921) who formulated criteria for the unstable extension of a crack in a brittle solid in terms of a balance between changes between mechanical and surface energies.

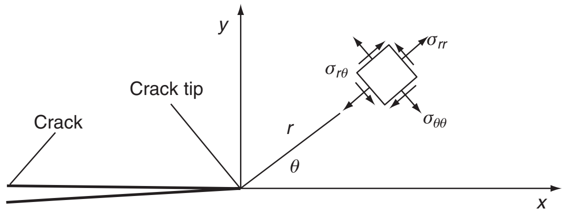


Figure 9 Stresses ahead of a crack tip

Stress and displacements in the vicinity of a crack tip

For plane problems (see **Figure 9**) where the crack is subject to tensile (termed Mode I) opening, it can be shown that the stresses and displacements close to a sharp crack tip are given by:

$$\begin{Bmatrix}\sigma_{rr}\\ \sigma_{\theta\theta}\\ \sigma_{r\theta}\end{Bmatrix} = \frac{K_I}{\sqrt{2\pi r}}\cos\frac{\theta}{2}\begin{Bmatrix}1+\sin^2\frac{\theta}{2}\\ \cos^2\frac{\theta}{2}\\ \sin\frac{\theta}{2}\cos\frac{\theta}{2}\end{Bmatrix} \tag{21}$$

$$\begin{Bmatrix}u_r\\ u_\theta\end{Bmatrix} = \frac{K_I}{2E}\sqrt{\frac{r}{2\pi}}\begin{Bmatrix}(1+\upsilon)\left[(2\kappa-1)\cos\frac{\theta}{2} - \cos\frac{3\theta}{2}\right]\\ (1+\upsilon)\left[-(2\kappa-1)\sin\frac{\theta}{2} + \sin\frac{3\theta}{2}\right]\end{Bmatrix} \tag{22}$$

where $\kappa = (3-\upsilon)/(1+\upsilon)$ for plane stress and $\kappa = (3-4\upsilon)$ for plane strain.

The term K_I is called the Mode I stress intensity factor (SIF), which incorporates the boundary conditions of the cracked body and is a function of loading, crack length and geometry. When the SIF reaches a critical value, initiation of crack growth will occur. This critical value under plane strain conditions is referred to as the critical stress intensity factor K_{IC} and is a material constant.

Compact tension stress intensity factor

K_I has been calculated for a number of standard geometries (e.g. Tada *et al.*, 1973). For example, a version of the compact tension (CT) test used for asphalt (Collop *et al.*, 2004a) is shown in **Figure 10** and its SIF is given by:

$$K_I = \frac{2P(2b+a)}{(b-a)^{3/2}}F\left(\frac{a}{b}\right)$$

$$F\left(\frac{a}{b}\right) = 0.443 + 2.32\left(\frac{a}{b}\right) - 6.66\left(\frac{a}{b}\right)^2 + 7.36\left(\frac{a}{b}\right)^3 - 2.8\left(\frac{a}{b}\right)^4 \tag{23}$$

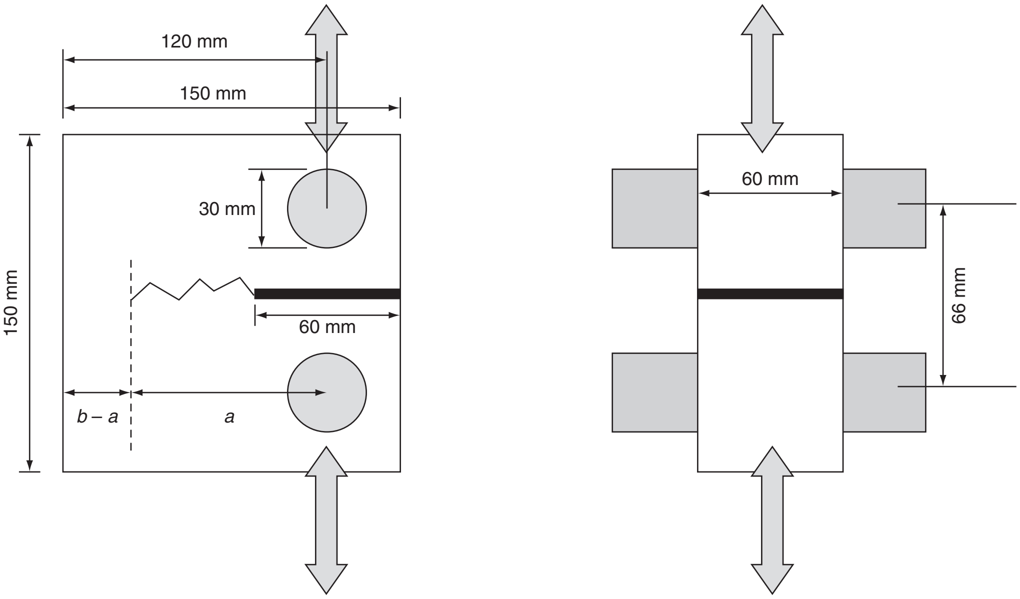


Figure 10 Compact tension (CT) test

Paris law

Under cyclic loading conditions, the Paris law is typically used to quantify crack growth from a pre-existing flaw or defect which is given by:

$$\frac{\mathrm{d}a}{\mathrm{d}N} = A\Delta K_I^n \tag{24}$$

The parameters A and n in Equation (24) can be determined from crack propagation testing. For example, **Figure 11** shows the crack length measured from the CT test plotted as a function of the number of loading cycles. A smooth line has been fitted to the data to reduce the scatter when the crack length is differentiated with respect to the number of loading cycles as required by Equation (24). The SIF at each crack length can be calculated using Equation (23) allowing $\mathrm{d}a/\mathrm{d}N$ to be plotted against ΔK_I^n on double logarithmic scales (see **Figure 12**) and hence the constants A and n to be determined.

Figure 13 shows fitted values of the exponent n in the Paris law as a function of stiffness for asphalts made using different penetration grade binders. It can be seen from this figure that the general trend is for the exponent to increase as the stiffness of the asphalt increases.

Figure 14 shows the constant A in the Paris law plotted as a function of the constant n. It can be seen from this figure that there is a good correlation between these constants

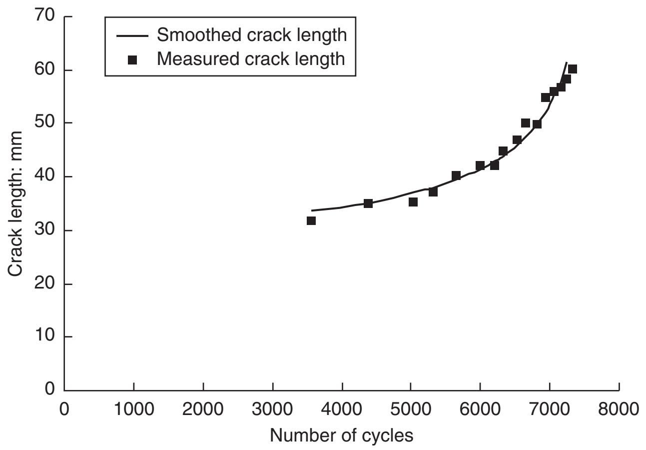


Figure 11 Crack length growth curve

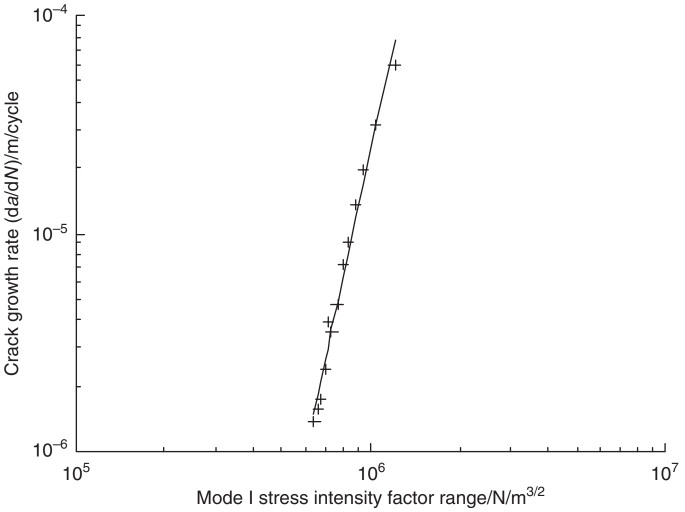


Figure 12 Fitted Paris law

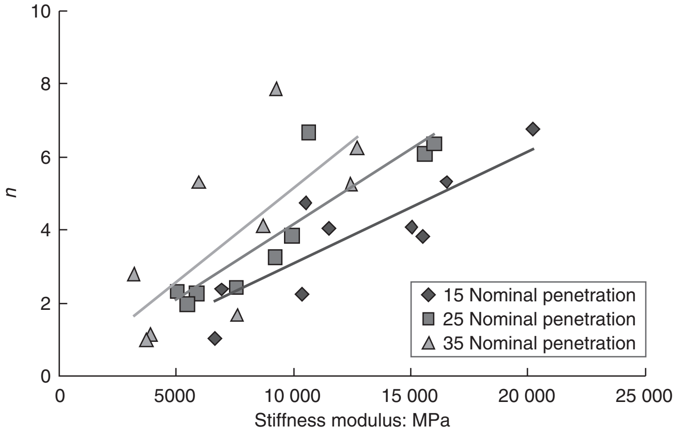


Figure 13 *n* plotted as a function of stiffness modulus

which means that if one is known, the other can be predicted.

Application to a pavement structure

If the SIF can be calculated for a crack in the asphalt layer in a pavement structure, the Paris law can be used to calculate the rate of crack growth and hence pavement life. For example, assuming the asphalt layer of a pavement structure can be idealised as a plane strain plate in bending (as shown in **Figure 15**), it can be shown that the SIF is given by:

$$K_I = \frac{Mb}{2I}\sqrt{\pi a}F\left(\frac{a}{b}\right)$$

$$F\left(\frac{a}{b}\right) = \sqrt{\frac{2b}{\pi a}\tan\left(\frac{\pi a}{2b}\right)} \times \left\{\frac{0.923 + 0.199[1 - \sin(\pi a/2b)]^4}{\cos(\pi a/2b)}\right\} \tag{25}$$

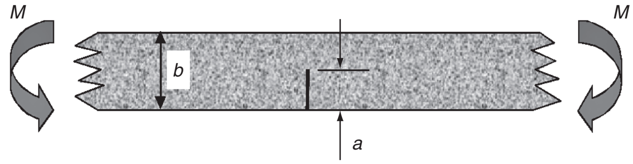


Figure 15 Idealised asphalt layer

where I is the second moment of area of the plane strain plate. It can be seen from Equation (25) that the bending moment M is required to be able to calculate the SIF. This was calculated using the layered elastic computer program (BISAR) developed by Shell, assuming that the thicknesses of the asphalt layer and granular sub-base were 300 mm and 200 mm respectively, a 40 kN standard wheel load applied over a circular contact area of diameter 315 mm (simulating a super-single tyre) was applied to the pavement surface, the stiffness moduli and Poisson's ratios for the subgrade and granular sub-base were taken to be 60 MPa and 0.4 and 100 MPa and 0.4 respectively and Poisson's ratio for the asphaltic material was taken to be 0.35. The stiffness modulus of the asphaltic material was obtained from testing.

Pavement life (number of cycles to failure) can be calculated by rearranging the Paris law to give:

$$N_f = \frac{1}{A}\int_{a_0}^{h}\frac{\mathrm{d}a}{\Delta K_I^n} \tag{26}$$

where N_f is the number of cycles to failure, a_0 is the initial crack length (taken as 1 mm) and h is the thickness of the asphalt layer. It can be seen that Equation (25) is a complex function of a so Equation (26) has to be integrated numerically.

Predicted pavement life for a range of mixtures, plotted as a function of the asphalt stiffness modulus, is shown in **Figure 16**. It can be seen from this figure that as the stiffness

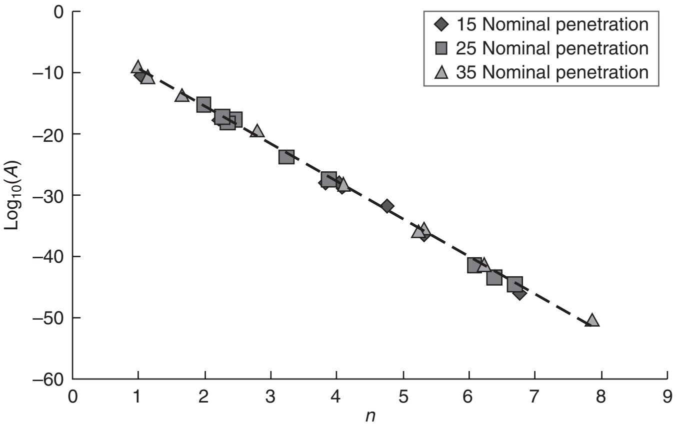


Figure 14 *n* plotted as a function of *A*

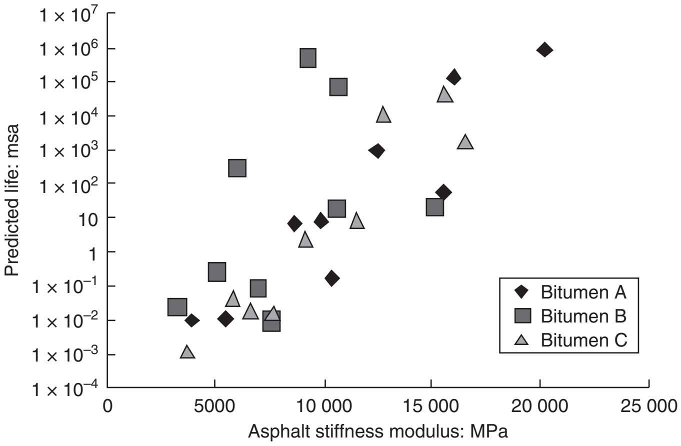


Figure 16 Predicted pavement life

of the asphalt layer increases, the predicted life also increases.

Non-linearities

LEFM assumed that the material is isotropic, linear elastic, homogeneous and the size of the inelastic zone around the crack tip is small compared to the crack length and other specimen dimensions. Although, at lower temperatures and/or faster strain rates this assumption is reasonable (Collop and Cebon, 1995), the size of the inelastic zone at higher temperatures and/or strain rates can be large compared to the crack size. Other more complex techniques, such as the J-integral and the C^*-integral, have been introduced to deal with such cases but they are beyond the scope of this chapter (see, for example, Schapery, 1975).

Discrete element modelling (DEM)

The above approaches essentially model asphalt as a continuum. An alternative approach, where the particulate nature of the material is explicitly taken into account, is Discrete Element Modelling (DEM). DEM was developed by Cundall and Strack (1979) for simulating movement and interaction of assemblies of rigid particles. In their original model, the movement and interaction of unbonded assemblies of rigid spherical particles subjected to external stress was simulated. Later developments of this approach resulted in the PFC3D software (Itasca, 2003). This program permits the incorporation of user-defined contact models and bonding of particles. The particles displace independently and only interact at the contact points through finite contact stiffness. PFC3D uses the so-called 'soft contact approach' whereby the particles are assumed to be rigid (non-deformable) but can overlap at contact points allowing deformation. The magnitude of overlap is related to the contact force via a force–displacement law. The mechanical behaviour of the system is described in terms of movement of discrete particles and inter-particle forces, which are controlled by Newton's second law of motion.

Figure 17(a) shows a numerical sample of a single-sized idealised (sand) asphalt mixture containing 6000 particles that are 1.8 mm in diameter. The specimen is 26 mm in diameter and 52 mm in length. The average number of contacts per particle for the sample shown in **Figure 17(a)** is 5.8 and the particles occupy 63.5% of the total volume. **Figure 17(b)** shows a sample of the same idealised (sand) asphalt (containing fewer particles) but where clumps (groups of spherical particles) have been used to introduce the effects of particle shape. In this case, the average number of contacts per particle has increased to 7.4.

A detailed procedure for generation of numerical samples can be found in Collop *et al.* (2004b).

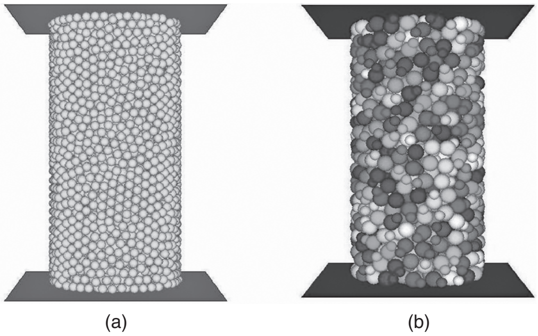


Figure 17 Numerical sample of an idealised asphalt mixture

Contact model

In DEM, contact models are required for both normal (tensile and compressive) and shear movements between adjacent particles. For normal (tensile) and shear behaviour, the Burger's model discussed above can be used to introduce time-dependent behaviour (Collop *et al.*, 2007). An elastic contact model can be used as a first approximation for normal (compressive) behaviour reflecting the fact that as particles move closer together the contact stiffness will tend towards the value for direct contact between sand particles.

In DEM normal (tensile) and/or shear bonds between particles can be broken to simulate accumulation of damage. Typically, a bond is broken when the normal (tensile) and/or shear force between adjacent particles reaches a pre-defined level, although a displacement criterion can also be introduced. Once a bond is broken, frictional behaviour is assumed between adjacent particles.

Creep test simulation

Figure 18 shows the predicted axial strain plotted as a function of loading time for an applied axial stress of 400 kPa. Simulations containing 1000 and 6000 particles have been

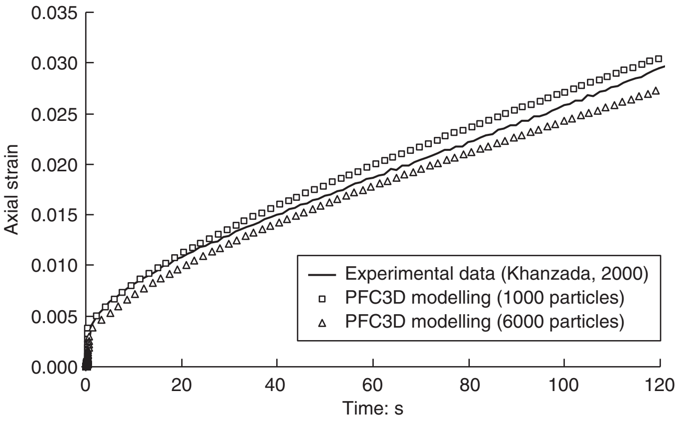


Figure 18 Predicted and measured creep response

Shear contact		Tension contact		Compression contact	
Property	Value	Property	Value	Property	Value
K_o^s: MN/m	0.12	K_o^n: MN/m	0.36	E_o^n: MN/m	0.70
C_∞^s: MNs/m	0.80	C_∞^n: MNs/m	2.40		
K_1^s: MN/m	0.03	K_1^n: MN/m	0.09		
C_1^s: MNs/m	0.80	C_1^n: MNs/m	2.40		

Table 1 Burger's contact model parameters for viscoelastic creep test simulations

undertaken. The model parameters are detailed in **Table 1** and were chosen to give sensible levels of axial strain and a similar shape creep curve compared to previously gathered experimental data (Khanzada, 2000). Note that no bonds were allowed to break during the simulation. It can be seen from **Figure 18** that, as expected for this type of contact model, the strain response is a function of loading time comprising elastic, delayed elastic and viscous components. A measured curve for the same stress conditions (Khanzada, 2000) is also shown in **Figure 18** from where it can be seen that the predicted and measured curves are similar in magnitude and shape.

It can also be seen from **Figure 18** that predicted strain from the model with 1000 particles is greater than the predicted strain from the model using 6000 particles. Collop *et al*. (2004b) investigated the effect of the number of particles (sample size) on bulk properties using elastic contact properties. **Figure 19** shows a plot of Young's modulus and Poisson's ratio versus the number of particles in the sample. The ratio of specimen height to specimen diameter was kept constant at 2:1 for all specimens. In these simulations the absolute values of the shear and normal contact stiffnesses were taken to be 1.8 MN/m and 20 MN/m respectively.

It can be seen from **Figure 19** that the general trend is for Poisson's ratio to decrease and Young's modulus to increase as the sample size (number of particles) is increased, tending towards constant values of 0.34 and 3.6 GPa respectively. It can also be seen from this figure that Young's modulus and Poisson's ratio calculated from the sample containing only 300 particles are approximately 30% lower and 28% higher respectively than those calculated from the sample containing 6000 particles. It can be concluded from **Figure 19** that at least 4500 particles are required for the results to be within 2% of those obtained using 12 000 particles. However, it should be noted that the computation time increases significantly as the number of particle is increased and it is often not practical to run simulations with large numbers of particles.

As noted above, one of the main benefits of DEM is that the internal particle geometry is explicitly taken into account which, if correct, should give realistic volumetric behaviour. **Figure 20** shows a plot of volumetric strain versus distortional (shear) strain from a number of triaxial simulations undertaken at different stress conditions for the idealised (sand) asphalt. Note here that the symbol η is used here as the stress ratio which is the mean stress divided by the deviator stress. In these simulations it has been assumed that 20% of the contacts are frictional in nature with a coefficient of friction of 0.7 and no additional bond breakage during the simulations has been allowed. It can be seen from this figure that the results for all stress conditions follow a similar pattern where initially there is a small decrease in specimen volume (compaction) followed by a significant amount of dilation where the specimen is increasing in volume at an approximately constant rate. Also shown in **Figure 20** are experimental results for Khanzada (2000) and it can be seen that there is good agreement between the measured and simulated results.

Constant strain rate test simulation

DEM can also be used to simulate results from constant strain rate failure tests. Results from a series of simulations

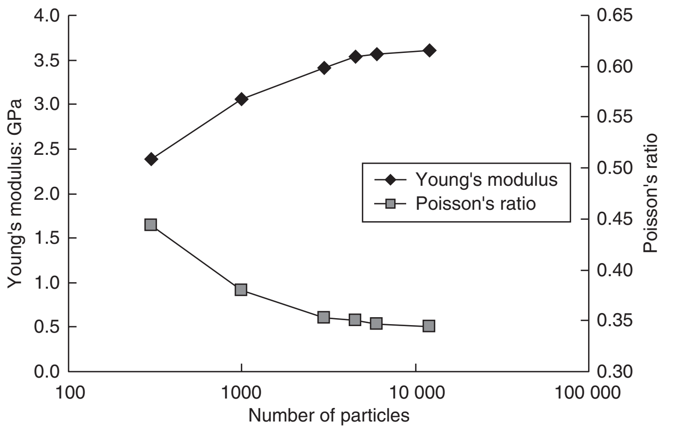


Figure 19 Effect of number of particles

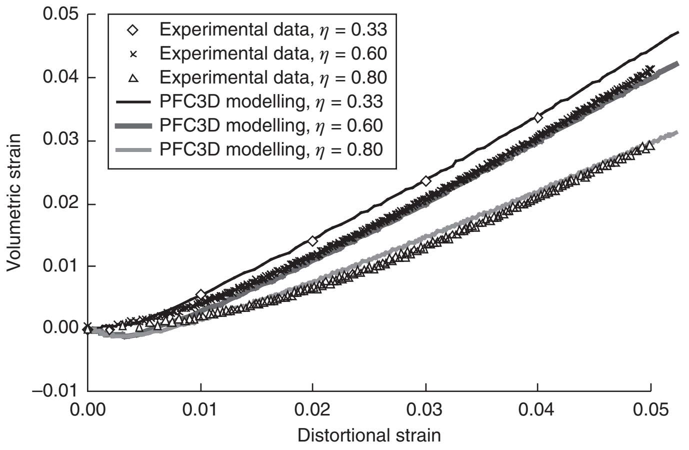


Figure 20 Predicted and measured volumetric behaviour

Shear contact		Tension contact		Compression contact	
Property	Value	Property	Value	Property	Value
K_o: MN/m	3.0	K_o: MN/m	3.0	E_o: MN/m	3.0
C_∞: MNs/m	1.5	C_∞: MNs/m	1.5		
K_1: MN/m	3.0	K_1: MN/m	3.0		
C_1: MNs/m	1.5	C_1: MNs/m	1.5		

Table 2 Burger's contact model parameters for viscoelastic constant strain rate test simulations

performed using a sample containing 1000 bonded particles and the parameters given in **Table 2** are shown in **Figure 21**. The normal (tensile) and shear bond strengths were scaled using experimental results where it was found that the strength is approximately proportional to strain rate on double logarithmic scales with a gradient of approximately 0.43 for the sand asphalt. As before, 20% of the initial contacts were assumed to be frictional with a friction coefficient of 0.7. It can be seen that agreement is generally good both in the pre-peak region and in the post-peak softening region over the range of strain rates simulated.

Summary

Viscoelastic models comprising springs and dashpots connected in series and parallel can be used to represent the time-dependent behaviour of bitumen and asphalt. Nonlinearities can be introduced in a relatively simple way and material parameters can be determined from creep and relaxation testing undertaken in the laboratory. Permanent deformation is predicted to be a function of the viscosity of the material. These models can be readily implemented into finite element (FE) software which allows the behaviour of more complex boundary value problems (such as pavement structures) to be investigated.

Linear elastic fracture mechanics (LEFM) can be used to calculate a stress intensity factor (SIF) which quantifies the stress conditions in the vicinity of a crack tip. The Paris law can then be used to quantify the rate of crack growth under repeated (fatigue) loading and material parameters can be obtained from laboratory testing. The method can be extended to predicting crack growth in more complex structures such as a pavement provided the SIF can be calculated which typically requires the use of FE software. One of the key assumptions in this approach is that the process zone around the crack tip is small compared to other dimensions such as the crack length. For materials such as bitumen and asphalt, the size of the process zone increases significantly at higher temperatures due to viscous flow, making this approach more applicable to lower temperatures where the response is more elastic in nature.

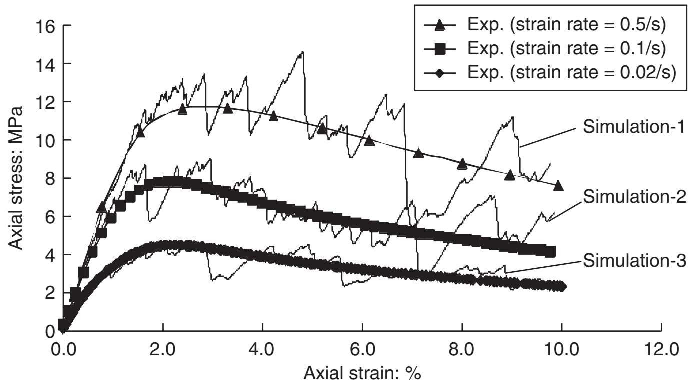


Figure 21 Predicted and measured constant strain rate response

Discrete element modelling (DEM) explicitly includes the particulate nature of asphalt. The particles are assumed to be rigid and deformations are focused at the contact points between particles. Time dependence can be introduced using viscoelastic contact laws and softening can be simulated by allowing contact bonds to progressively break. The main disadvantage of this type of model is that it is extremely computer intensive and simulations are typically limited to modelling the behaviour of a representative volume of material in a laboratory test.

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CONTENTS

Chapter 30

Masonry: an introduction

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Evidence of masonry, dating back as far as 8000 BC, can be witnessed today in Jericho, near Jerusalem. This section briefly describes the history of masonry units from this first recorded mud brick masonry to the main types of masonry that exist in the UK today (namely clay, aggregate concrete, calcium silicate, autoclaved aggregate concrete, stone and reconstructed stone). The chapter touches on the potential problems that can be experienced by designers of masonry due to the inherent variability of the masonry units and the ease with which the properties of mortar can be affected. It also suggests that masonry in the UK can prove itself to be a sustainable option for the twenty-first century.

Background

Masonry, in particular the awareness of brickwork, is ingrained within our language and culture. We have long seen the brick as a symbol of solidity; we often talk about an object being as hard or as solid as a brick and the metaphors of 'knocking heads against brick walls' and 'sinking like a brick' are still common today. Perhaps less common these days is to pay someone the compliment of calling them a brick. This is believed to derive from the reply of an Eastern king to an invading army which made fun of his unwalled towns: 'My troops are my walls and every soldier is a brick' (Woodforde, 1976). Within literature, one of our most famous authors, Charles Dickens in *Bleak House* (1853) attributes to his most 'brutish' character the profession of a brickmaker (the brickmakers of the Victorian times were allegedly as hard-drinking as railway navvies and suffered similar endurances in their work).

The aim here is to provide a brief history of the main types of masonry unit currently used in the UK today, namely clay brick, aggregate concrete, autoclaved aerated aggregate, calcium silicate, stone and reconstructed stone units. Evidence suggests that the oldest form of masonry construction, dating back to at least 8000 BC, is clay (mud) brick masonry. Walls constructed using sun-dried mud bricks can still be witnessed today in the town of Jericho (El Ariha), near Jerusalem in the West Bank. Units taken from these walls were clearly hand-formed (definite evidence of Neolithic hand marks remain – imprints which would help the brick bond with the mud mortar). Globally, dried mud brick (or adobe, which is Spanish for mud) masonry was the most popular form of construction between 8000 and 700 BC. However, from around 3000 BC it is apparent that mud bricks were also 'kiln-fired'; evidence of fired bricks can be found today in structures built in several Sumerian cities of present-day Iraq, i.e. a palace in Erech (c. 2400 BC) and a temple in Ur (c. 2300 BC). During this period up to 77 BC, the size of the units varied greatly. The Babylonians tended to use larger units (up to 450 mm square by at least 75 mm thick) and perhaps the most well-known structure which used these units was the Ishtar Gate in Babylon, constructed under Nebuchadnezzar II (604–562 BC). This structure is also of interest as for the first time the units were embedded in a bitumen mortar, which at the time was very innovative. After the collapse of the Babylonian Empire under Nebuchadnezzar, the Persians abandoned the use of bitumen in favour of lime mortars (it was nearly 2000 years before bituminous mixtures were used again in construction).

The mud used for these bricks was simply extracted from the ground. Another construction material also available in this way is stone. However, while forming mud bricks is an easy process requiring little specialised knowledge or equipment, stone will often require quarrying and cutting, which is much more demanding technically and hence more expensive (it is also not as readily available as mud and so was also more likely to incur additional costs from transport). As civilisations developed, so did their skills and technology and stone masonry became more popular, particularly for more prestige constructions. The Romans (c. 400 BC to approximately AD 500) (*The Times History of Europe*, 2002) were prolific users of unreinforced brick and stone masonry. This type of masonry was recognised as being good in compression and was therefore used for the construction of walls and columns (see **Figure 1**). However, the Romans also appreciated that their country was subject to seismic activity and that this type of ground movement could easily produce tensile forces in the masonry which possessed little or no tensile strength. Perhaps this is why in many of their temples the columns are surmounted by a large stone/timber roof structure which is not just architectural but acts structurally by compressing the columns and making it harder for tension forces to develop (see **Figure 2**). Brick and stone masonry could also be used to span gaps using the arch principle

Figure 1 Roman brick column, Paestum, Italy

Figure 2 Temple of Neptune, Paestum, Italy

and the semicircular arch (whether as a lintel over a door opening or as a bridge over a river) is a major characteristic feature of Roman architecture (see **Figure 3(a)** Pont du Gard and **Figure 3(b)** Segovia Aqueduct).

Burnt or fired brick units were introduced into the UK by the Romans during the first century AD. It is of interest that the Romans fired the bricks they manufactured in the UK but left the same bricks manufactured back in Rome to dry in the sun. They clearly understood the effect of the harsher climate present in the UK on the durability of the units and their corresponding structures. Masonry walls constructed using burnt bricks were actually solid walls faced with bricks and with a core of 'concrete' (lime and earth or lime and volcanic ash, which is pozzolanic by nature).

(a)

(b)

Figure 3 (a) Pont du Gard, France; (b) Segovia Aqueduct, Spain

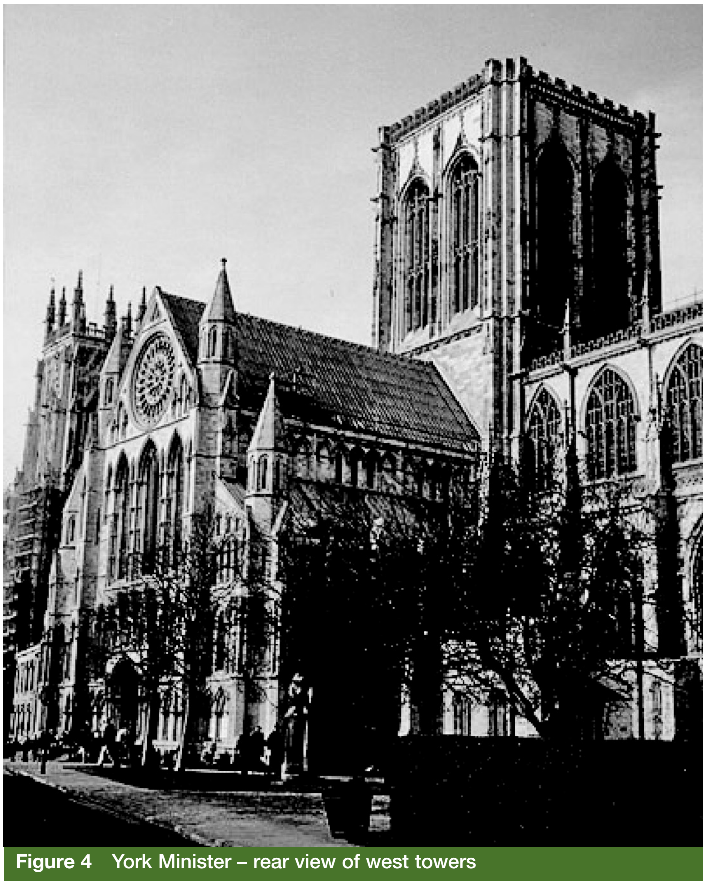

Figure 4 York Minister – rear view of west towers

Up until the twelfth century there was little development in brick masonry although there was evidence that brick units were being incorporated into stone masonry more regularly and as an increasing percentage of the overall structure. However, in AD 1190 the first English brick, the Mediaeval Great Brick, was introduced. There was also the introduction of the Flemish brick into the UK (the English and Flemish brick bonding patterns are still common today). At this time, parallel developments in brick units were occurring in the Low Countries (Germany, Belgium and Holland – all countries where clay bricks are still popular today). The increase in immigration and trade across the North Sea into the UK, which happened in the twelfth and thirteenth centuries, perhaps explains the presence of the Flemish brick in this country at that time.

As mentioned earlier, stone was still the material that was used for the more prestigious structures and stone architecture continued to develop on into the Gothic period. The fourteenth century perhaps sees the best exploitation of stone masonry technology with the construction of structures such as York Minster (around 1350) (see **Figure 4**). This period arguably represented the pinnacle of stone architecture; beyond the improvement in tools and manufacturing techniques there has been no real technical development (structurally) in this material since the fourteenth century.

Tattershall Castle, Lincolnshire (a consequence of the Hundred Years War; hence why the architecture is so ‘French’) was finished in 1449 (see **Figure 5**) and was perhaps the precursor to the next major step change in brick and stone masonry which occurred during the Tudor Period (1485 to 1603). It was during this period that brick finally usurped stone as the material of choice of the then rich upper classes. King Henry VIII chose to construct St James’s Palace using brick masonry (see **Figure 6**) which established brick as a new status symbol. Henry VIII’s reign (1509 to 1547) was the first great age of clay brick masonry. The use of brick masonry further increased during the seventeenth century, particularly following the Fire of London (1666) and other fires (Marlborough, Northampton, etc.) where all dwellings were rebuilt using brick. The increased demand for brick during the seventeenth and eighteenth centuries produced the need for more industrial scale brickmaking. However, manufacturing the bricks was still very labour intensive. There was a slight downturn in the popularity of brick masonry during the early nineteenth century due to the brief fad for stucco facings to buildings. However, during the majority of the Victorian era the fired clay brick

Figure 5 Tattershall Castle

Figure 6 St James's Palace, Pall Mall, London

continued to increase in popularity. It was also during this time that several major innovations were introduced, i.e. large-scale automatic grinding equipment, 'automated' moulding equipment, wire cutting technology and the invention of the Hoffman kiln – all of which significantly increased the numbers of bricks produced and significantly reduced the required labour force.

Burnt clay bricks were now being used for large and iconic structures. It was not unusual to see viaducts and bridges using brick masonry construction. The structural sections, which were very thick (often >1000 mm wide) tended to consist of two elements: a core of common brickwork and an external facing of brickwork which was keyed into the core. Prestigious buildings such as the Prudential Building, Holborn London (1879), Butterfield's Keble College Chapel, Oxford (1870) and St Pancras Station, London (1869) (see **Figure 7**) also used load-bearing clay brick masonry. Examples of load-bearing masonry skyscrapers can also be found in Chicago, USA (Chicago Landmarks, 2009). Monadnock Block (1891) provides one of the finest examples of a skyscraper built only from load-bearing masonry (see **Figure 8**). Seventeen/eighteen stories high, the walls were 1828 mm thick at the base.

Figure 7 St Pancras Station, London

Portland cement, patented in 1824 by Joseph Aspdin, a Leeds bricklayer, did not become available on the market for general use until around the start of the twentieth century. Therefore, lime mortar was used to 'bed' the masonry units (brick and stone) prior to the introduction of (Portland) cement mortars. The availability of Portland cement also led to the introduction of a new masonry unit, the concrete aggregate block unit. This new unit was introduced following World War I (around the early 1920s) and was developed due to the dearth of adequate or any housing at that time and the lack of qualified bricklayers (many of whom who had been lost during the Great War). The blocks were developed in two sizes – 457 mm × 228 mm × 76 mm (thick) (18 inches × 9 inches × 3 inches) and 610 mm × 228 mm × 51 to 64 mm (thick) (24 inches × 9 inches × 2 to 2.5 inches) and were typically

Figure 8 Monadnock Block skyscraper, Chicago, USA

made from ballast aggregate (granite, hard stone or gravel) or clinker aggregate and cement. The thicker unit was made using a 1:6 (cement:aggregate) mix and the other units were made using a 1:4 (cement:aggregate) mix. The blocks were used for the inner and outer leaf of masonry, separated by a 51 mm gap and tied (similar in arrangement to the cavity wall system used today).

Further innovations in masonry units occurred due to the advent of the autoclave (a vessel which can apply heat at elevated pressures) in the late nineteenth century. An inventive and opportunistic industry saw the potential to produce:

1. A calcium silicate brick: this was a unit formed from a mix of sand and lime and cured using steam under pressure. The process, which is still used today, was patented in Germany in 1880 (initially patented in the UK in 1866). Mass production of these units began in 1894. While popular in mainland Europe and in many parts of the World, these types of units have never competed well with more traditional clay units and currently there are only two manufacturers in the UK (Esk Building Products, 2009).
2. An autoclaved aerated concrete block unit which went into production in the late 1920s (for further details of both see Chapter 31).

More recently (in the twentieth century) traditional unreinforced masonry in the UK has predominantly been used within a cavity wall system which was introduced in its current form in the 1940s. Wall sections are now much thinner and this slenderness makes it very difficult to use masonry structurally. It is mainly used for domestic house building, cladding and internal/partition walls and, unfortunately, very rarely appears as a major structural material in prestigious projects. The reasons are unclear for this. Perhaps it has fallen out of favour in its current form. Perhaps there is now a lack of design engineers with the experience to use masonry beyond the limiting guidance provided by the codes, or a lack of bricklayers experienced and proficient enough to work with masonry in innovative ways. Towards the end of the twentieth century there was a mini-revival in the use of masonry both prestigiously and innovatively. An example of the latter is Winterton House in London where clay masonry was used to strengthen and refurbish an existing steel-framed high-rise tower block (Bird and Hitchens, 2000). The ‘off-the-frame’ cladding system developed by the engineers Ramboll Whitbybird (formerly Whitby Bird) allowed the capacity of the steel frame to be improved by ‘jacking’ it against the masonry walling. This successful solution was also impressive as, being beyond the limits of the code guidance, much of the knowledge had to be gained specifically for the project. Now, at the start of the twenty-first century, it is apparent that new masonry construction techniques are being developed (see Chapter 35). It is hoped that these new systems will once again help to elevate masonry to be one of the major construction materials of choice, not just in terms of its structural and aesthetic potential but also because of its durability and sustainability.

Properties of masonry

In order to assess the load-carrying capacity of unreinforced masonry in, for instance, the vertical and lateral directions it is necessary to determine the characteristic compressive and flexural strengths of the masonry, respectively. Ignoring constants, these characteristic strengths depend on the normalised compressive strength of the unit and the compressive strength of the mortar. (Note: the fundamental design of masonry is considered in Chapter 34.) This introduces a very pertinent phenomenon of masonry units (in particular clay masonry units) and that is their inherent variability. There is also the potential for variation in mortars due to the influence of curing, environmental conditions and the interaction of the mortar with the unit component of the masonry. The following describes some of the main reasons for this variability and the potential implications.

To conform to the Eurocodes, manufacturers must be able to provide specific data on a range of pre-described properties of their bricks and monitor any changes to these properties on a regular basis (see Chapter 2). Periodic monitoring is required as variability exists within the individual brick types due to several factors; the first major factor considered here is the variation in the mineral composition of the clay brick material (although the nature of the minerals in a single clay type is unchanged, the percentage amounts differ). The design stage of a project could potentially be up to 18 months before the project goes to site. Therefore, prior to construction, designers can only get indicative property values by referring to the product literature, which reasonably can only suggest conservative values for the properties of a brick or a range of values from which the designer will almost certainly go for the lowest, again to be conservative. The variation in properties is a result of the depth of the clay workface, which, in order to be economic, needs to be a minimum of 2 m and more often 15–25 m deep (see **Figure 9**). Within vertical heights of this magnitude, wide limits of clay strata exist. The position can be complicated further by a geological dip in the strata, so that different beds or levels may be exposed and worked throughout the lateral extent of the quarry. The second major factor affecting the properties of a brick is the non-uniformity present in the firing process (Bonner and Butterworth, 1950; Wyatt, 1978), i.e. temperature variations within some kilns. It has been shown by Bonner and Butterworth (1950) that the main properties influenced by the firing temperature are:

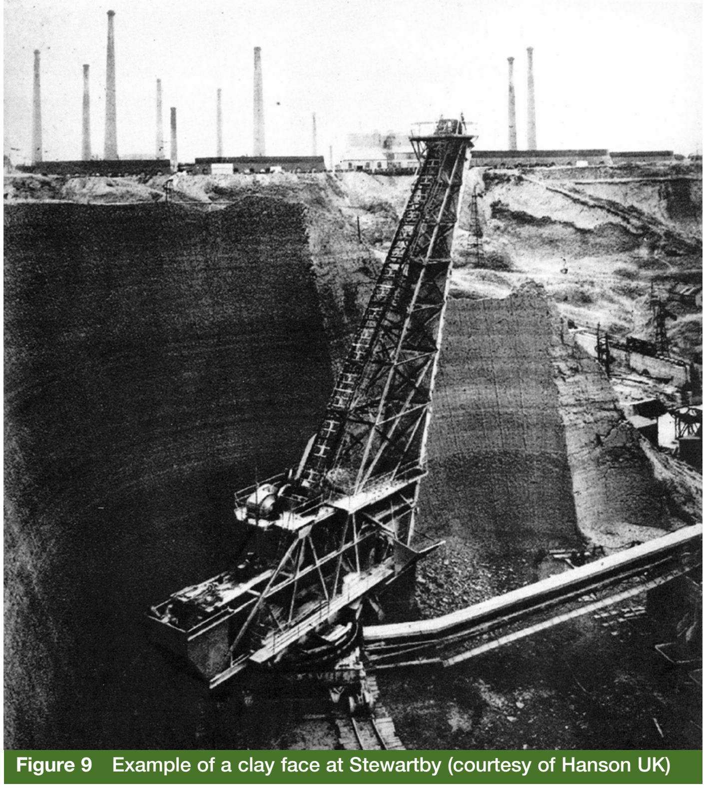

Figure 9 Example of a clay face at Stewartby (courtesy of Hanson UK)

(i) compressive strength
(ii) water absorption
(iii) linear firing shrinkage.

They also illustrated how susceptible these properties were to a change in firing temperature, i.e. when the temperature was raised from 1000°C to 1100°C, it resulted in a correspondingly proportionate linear change in:

(i) compressive strength from 40.1 MPa up to 63.6 MPa
(ii) water absorption from 37.5% down to 25%
(iii) linear firing shrinkage from 1.2% up to 4.7%.

If we consider the fact that the variation in temperature within a specific area of a kiln can be as much as 40°C, it is clear to see how these properties can be affected and how the classification of a brick can be altered, i.e. in terms of its durability and frost resistance. In addition, it was reported how its classification in terms of soluble salt content could also be affected by a change in firing temperature.

Masonry mortar (which is discussed thoroughly in Chapter 32), required for more significant construction projects, is predominantly delivered to site ready mixed. Ready-mixed mortar is also more often than not used for small-scale construction projects (i.e. house extensions) although mortar is still mixed by hand on site. When hand mixed on site, the quality control of the components (cement, sand, water, etc.) of the mortar is reduced and often the mortar is only deemed acceptable by the bricklayer according to the bricklayer's own personal preference for laying the bricks. Ready-mixed mortar is quality controlled and batching is controlled at least by strength properties. However, once the mortar arrives on site it can be subjected to a whole range of variables which will affect the development of its properties and the way it performs in the masonry. The design codes, on the whole, take adequate account of this, although in doing so their approach is, as it has to be, conservative. However, it is worth bearing in mind that underestimating the properties of a material can have certain negative implications. For instance, an underestimation of the characteristic compressive strength of the masonry (resulting from the choice or use of conservative values of brick and mortar) could easily lead a designer to overestimate the amount of creep in design (this is discussed more in Chapter 37).

All of which often leave the designer with the impression that masonry never does what it is supposed to do!

Sustainability of masonry

If masonry is considered from 'cradle to grave', i.e. the whole-life of the material, then there are strong arguments to support the claim that masonry is in fact a sustainable material. However, if the material is considered from 'cradle to gate', i.e. the manufacturing period, it has to be accepted that the sustainability of the material (some unit types more than others) can be improved. Irrespective of the arguments relating to the weighting of the manufacturing process on the whole-life performance of the material, the industry has a duty to attempt to improve the sustainability of masonry at this stage. The manufacturing processes of the six major masonry units discussed here involve one or more of the following: energy-intensive practices (i.e. firing/curing/pressure); the extraction of raw materials; or the use of cement. Chapter 36 discusses the sustainability of masonry in much more depth and illustrates how the industry is striving to improve its performance from 'cradle to gate'. As an example, it has been replacing naturally extracted aggregates with recycled aggregates and in the manufacture of clay bricks, where partial replacement is occurring, their inclusion is having the added benefit of producing a reduction in the kiln firing temperatures. This is therefore reducing the energy requirements of the manufacturing process and reducing emissions. Other examples in Chapter 36 show, encouragingly, how the industry has actually grasped this opportunity and is meeting this challenge head-on. It also provides evidence which suggests that the masonry industry has in many ways been at the forefront of the drive by the whole of the construction industry to improve the sustainability of construction materials in general.

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Chapter 31

Masonry units

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Traditional unreinforced masonry can be considered a composite composed of two major elements: the unit and the mortar. Both elements are in many ways equal in importance, however the unit will always constitute a much greater proportion by volume of the masonry. This chapter concentrates on the unit (i.e. clay, concrete aggregate, aerated and reconstructed stone, calcium silicate and natural stone) element of the masonry and discusses the properties of these different types of units in relation to the codes with which each unit has to comply. The chapter also describes the most common manufacturing processes used in the UK for each unit and indicates the materials from which the units are composed. It is important to appreciate that the recently introduced codes for the specification of all masonry units are written as performance standards. As such, all requirements are based on the finished product.

CONTENTS

Overview

Eurocode 6 Part 1 (BS EN 1996-1-1: 2005) defines a masonry unit as a preformed component, intended for use in masonry construction. In both this code and BS EN 1996-2: 2005 six types of masonry units are named which could potentially satisfy this definition, providing that they comply with the relevant parts of BS EN 771: Specification for Masonry Units:

- clay units – BS EN 771-1: 2003
- calcium silicate units – BS EN 771-2: 2003
- aggregate concrete units (dense and lightweight) – BS EN 771-3: 2003
- autoclaved aerated concrete units – BS EN 771-4: 2003
- manufactured stone units – BS EN 771-5: 2003
- dimensioned natural stone units – BS EN 771-6: 2003.

Within the various parts of the code, guidance is given on the criteria with which the units have to be measured in order to satisfy conformity requirements. For specific details on how and when to perform these conformity checks, reference should be made to BS EN 772: Methods of test for masonry units:

- determination of compressive strength BS EN 772-1: 2000
- determination of air voids in masonry units (using paper indentation) BS EN 772-2: 1998
- determination of net volume and percentage of air voids of clay masonry units by hydrostatic weighing BS EN 772-3: 1998
- determination of real and bulk density and of total and open porosity for natural stone masonry units BS EN 772-4: 1998
- determination of the active soluble salts content of clay masonry units BS EN 772-5: 2001
- determination of bending tensile strength of aggregate concrete masonry units BS EN 772-6: 2001
- determination of water absorption of clay masonry damp proof course units by boiling in water BS EN 772-7: 1998
- determination of volume and percentage of voids and net volume of clay and calcium silicate masonry units by sand filling BS EN 772-9: 1998
- determination of moisture content of calcium silicate and autoclaved aerated concrete masonry units BS EN 772-10: 1999
- determination of water absorption of aggregate concrete, autoclaved aerated concrete, manufactured stone and natural stone masonry units due to capillary action and the initial rate of water absorption of clay masonry units BS EN 772-11: 2000
- determination of net and gross dry density of masonry units (except for natural stone) BS EN 772-13: 2002
- determination of moisture movement of aggregate concrete and manufactured stone masonry units BS EN 772-14: 2002
- determination of water vapour permeability of autoclaved aerated concrete masonry units BS EN 772-15: 2000
- determination of dimensions BS EN 772-16: 2000
- determination of freeze–thaw resistance of calcium silicate masonry units BS EN 772-18: 2000
- determination of moisture expansion of large horizontally perforated clay masonry units BS EN 772-19: 2000
- determination of flatness of faces of masonry units BS EN 772-20: 2000.

Masonry units are now categorised into four groups in accordance with the geometry of their perforations and holes. Autoclaved aerated concrete, manufactured stone and dimensioned natural stone units are considered to be in Group 1. Clay units manufactured in the UK are more

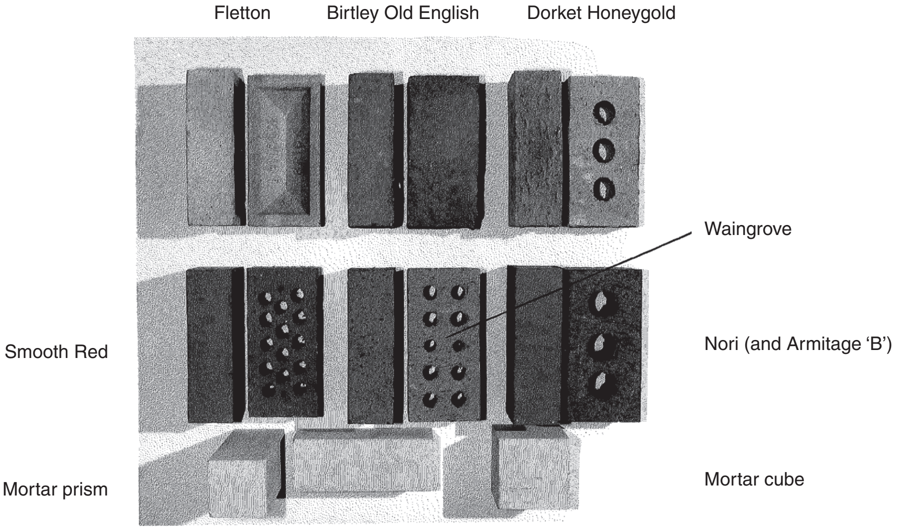

Figure 1 Samples of clay brick

likely to fall into either Group 1 or Group 2 which limits the vertical holes in the unit to no more than 55% of the gross volume of the unit. Aggregate concrete units are also classified in the UK as either Group 1 or 2. The geometrical requirements for the grouping of all units are given in Table 3.1 of BS EN 1996-1-1: 2005.

In relation to durability, according to BS EN 1996-2: 2005 masonry units should be selected from Table B1 of the code according to the exposure class of the masonry determined from Table A1, again from this code. Masonry is considered to be potentially subjected to five micro conditions of exposure: dry environment; exposed to moisture or wetting; exposed to wetting plus freeze–thaw cycling; exposed to saturated salt air, seawater or de-icing salts; and an aggressive chemical environment. Protected masonry is masonry which is protected against water penetration. It can either be external wall masonry (which can for instance be protected by a layer of suitable render or by cladding), the inner leaf of a cavity wall or an internal wall. A protected wall may or may not be load-bearing. Clay masonry units with a low gross dry density (LD unit) are used in protected masonry. A high gross density clay unit (HD unit) is normally used in unprotected masonry but can be used in protected masonry. (Note: BS 5628-3: 2005 covers durability in more detail than the Eurocode (BS EN 1996-2: 2005) and a residual British Standard is planned to pick up the aspects missed by BS EN 1996-2: 2005.)

There are several examples of alternative masonry units (those units which are not covered by the six types mentioned above) which are starting to enter the construction market. Certification of these products should ensure that these products still conform to BS EN 772 and all certification documentation should reflect this.

In the following sections each type of masonry unit is discussed in terms of its materials and manufacture and its specific tests for conformity.

Clay bricks

Introduction

Clay bricks are manufactured to BS EN 771-1: 2003. In the UK, the standard size of the unit is 215 mm (length) × 102.5 mm (width) × 65 mm (height). However, for sizing of masonry, the dimension of 225 mm × 112.5 mm × 75 mm should be used (this accounts for a nominal 10 mm mortar joint). Many brick manufacturers also produce specially shaped bricks to BS 4729: 2005 but these are not cited in the European Standards.

Clay bricks can be solid, frogged or perforated. The common formats produced in the UK are shown in **Figure 1**. The units commonly used in mainland Europe generally have a higher percentage of voids and perforations.

The UK has an extensive history of clay brick manufacture with hundreds of brick types available on the market. The variation in units is partly due to the variability in raw materials across the UK. The main raw material of clay bricks is naturally occurring clay. **Table 1** illustrates several examples of the major deposits of clay found in the UK used to manufacture clay bricks. The distribution of clays throughout the UK is well mapped and the types of clay extracted through quarrying are often named after the location in which they are found, although deposits often stretch far further than the original point of extraction, for example Oxford clay, most known for making Fletton bricks, can refer to a deposit as far apart as Dorset and Yorkshire. The significant variation in the content of

Clay brick unit type	Clay type	Manufacturing process	Compressive strength: MPa	
			Measured	Manufacturer's
Fletton	Lower Oxford	Pressed	28.2	20
Birtley Old English	Team Valley Wash	Slop moulded	31.5	21
Dorket Honeygold	Keupar Marl	Extrusion	54.0	43
Smooth Red	Eturia Marl	Extrusion	92.2	50
Nori	Lower Coal Measure	Extrusion	108.0	92
Waingrove	Coal Measure Shale/Carboniferous	Extrusion	123.7	>100
Armitage Class B	–	Extrusion	98.3	70

Table 1 Examples of clay types previously used in the manufacture of clay bricks

clays found in different locations throughout the UK and those extracted from certain deposits will produce units with differing properties.

The origins and compositions of clay materials are discussed at length in Reeves (2006). In very simple terms, clays can be described as naturally occurring aluminosilicates. Mineral clay is produced as a result of the erosion and decomposition of igneous rocks.

The compressive strengths of the different bricks available can range from below 10 MPa to above 100 MPa. However, for strength purposes it is very rare in design that bricks of greater than 30 MPa compressive strength are required in unreinforced masonry. The controlling design criterion is normally durability and, architecturally, it is the colour or the aesthetics of the brick. **Table 1** illustrates the potential difference that can be achieved between the manufacture's value of compressive strength and those measured on an individual batch as delivered to site.

Manufacture of clay bricks

There are three main stages in the manufacture of clay bricks. These are known as the mixing stage, the forming stage and the firing stage and they are described below. For convenience, the mixing and forming stages have been combined (the forming technique can depend on the clay and its water content).

Mixing and forming

There are several techniques used for the shaping of clay bricks. The main processes revolve around an extrusion technique; this process could be considered more suitable for mass production and produces predominantly perforated bricks. The soft mud process is more appropriate for the manufacture of brick 'specials' (i.e. special shapes where perhaps a mould is manufactured to produce only one brick). Bricks can also be pressed; the most common brick in this category is the Fletton (one of the most common bricks used in the UK).

Stiff extrusion

In this process the clay is mixed with a lower water content (10–15%) to produce a far stiffer mixture. Upon removal from the mixer the material is cut into approximate units before being pressed in a die to a specific size and shape. Since these units have a low moisture content, their shrinkage is also low, and they retain their shape well after pressing.

Soft extrusion

The clay in this process is softer due to a water content of 20–25%. The mixture is extruded through a die as a column with the desired length and width, and is then cut to the correct height using wires (see **Figure 2**). Allowances are made for the dimensional changes that occur during drying (resulting from the higher water content). The wire cutting produces units with smooth, uniform faces and no depressions. The use of moulds with elements to form perforations along the length of the column as it is extruded is common, and has several benefits:

- The amount of material used is lowered, reducing the cost and environmental impact.
- 'Opening'[1] of the unit causes the firing process to be less energy intensive.
- Thermal insulation of the masonry is improved since the voids act as a heat sink.

Soft mud process

Once extracted, the clay is crushed, ground and then mixed with 25–30% of water (by weight) to form a highly wet mixture (e.g. a soft paste). Historically, the mixture was

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[1] 'Opening' simply means exposing more of the surface area of the brick thus reducing the drying/heat transfer path. The production of units with voids does however require that the clay in the mixture be extremely consistent and uniform to limit the appearance of imperfections in the column of extruded clay.

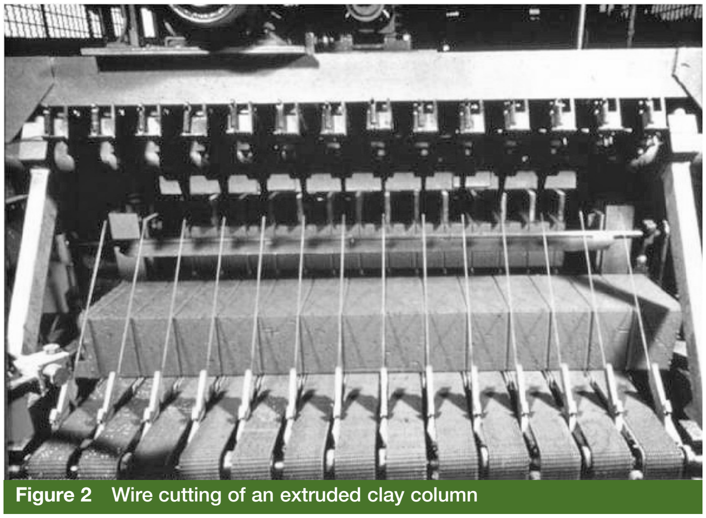


Figure 2 Wire cutting of an extruded clay column

thrown by hand into a mould and then cut to the desired shape and size by a wire. The mould can either be lubricated with sand or water, and bricks are denoted as 'sand-struck' or 'water-struck', respectively. The smoothness of finish is affected by the choice of lubricant. This manufacturing process is still used today but it is typically carried out by machinery although some manufactures still do form the bricks by hand.

Bricks produced in this way are typically 'frogged', i.e. contain a depression in their centre formed due to a raised section in the mould (see **Figure 1** – Fletton). Although this process can also be used to produce solid bricks (see **Figure 1** – Birtley Old English). Bricks produced by way of the soft mud process have a significant variability in their shape and dimensions, making them unsuitable for applications where precision is required. This variability is mainly due to the high plasticity and shrinkage of the clay resulting from the high moisture content of the formed material. The production of soft mud bricks is labour intensive and expensive, and is limited to mainly decorative or specialised bricks.

Dry pressing

Dry pressing is suitable for clay mixtures with very low water content, and involves the use of high compaction pressures to form a smooth-faced brick in a steel mould. Fletton bricks (named after the village of Fletton near Peterborough where this type of clay was 'discovered') are produced using a process based on this method. These bricks are formed using a specific clay with an unaltered moisture content; the clay is separated into fractions of coarse and fine material, which are used for either the manufacture of common bricks or facing bricks, respectively. Flettons are generally frogged and if they are to be used as facing bricks will either be rusticated or have a coloured sand applied.

Firing

Clay bricks are ceramic masonry units formed from natural clay mixtures fired at high temperatures to produce the irreversible transition of the green, plastic clay brick to this ceramic unit. The process of firing involves the use of specialised kilns and very high temperatures (up to approximately 1400°C). Firing is responsible for much of the cost of the brick manufacturing process and it is crucial to the final properties of the unit.

The typical chemical processes and the temperatures at which they occur during the firing of the clay products can briefly be summarised as follows. Initially, the units are heated to 100–200°C so as to evaporate both the free water that remains after drying and the weakly bound water contained within the minerals. The temperature is then raised and, between 400 and 700°C, these minerals decompose and yet more water evaporates. Crucially, the decomposition of these minerals leaves behind amorphous silica and alumina, which then crystallise at 900°C and above. The key changes occur above a temperature of 1000°C when vitrification and sintering take place, forming the mineral mullite, an aluminosilicate. (Vitrification refers to the alteration of the clay matrix into a glassy, less porous material and sintering to the fusing together of ground particles under extreme conditions.) These processes are responsible for the formation of a ceramic material; however, some porosity is desirable so control of the vitrification process is essential to good firing. There are several other less well understood chemical processes that occur during firing. These mainly involve the impurities present in the clays, such as gypsum, which may give rise to undesirable decomposition products.

One key characteristic of fired bricks is their colour. This is influenced by the oxidation state of the iron (Fe) within the clay mixture. The red colour occurs due to Fe(III) oxide, but under certain conditions, such as limited air, there may be a greater presence of Fe(II) oxide or mixed oxidation states which can influence the colour of the brick. Other constituents such as lime and artificial pigments may also affect the product.

Kilns

There are two main categories of kiln: one is the intermittent kiln and the other is the continuous kiln. Intermittent kilns tend to be used only for small batch processes of specialised units. Most kilns are continuous and can be further subdivided based upon the nature of their operation.

Chamber or Hoffman kilns (named after Friedrich Hoffman) utilise a system whereby the units are loaded into a series of chambers that surround a system of ducts. The first chamber is fired and then the fire is then 'moved' from one end of the series of chambers to the other; while the units that are behind the fire cool, the units in front of the fire are heating up. One of the benefits of this kind of kiln is that the gases from one chamber, such as the cooling chamber, can be transferred to another, such as the drying chamber, which increases efficiency. Fletton bricks are well known for being fired in a Hoffman kiln. The advent of the Fletton brick was a revolution in brickmaking which occurred around the 1880s. As mentioned above, the firing process is responsible for much of the expense of the brick-making process. The lower Oxford clay used to manufacture Flettons naturally contains approximately 10% fuel-oil (carbon), which meant that it could be fired at much lower temperatures. Temperatures are in fact maintained at just over 900°C to remove any residual carbon and are controlled using cold air so that they never go above 1000°C. One final comment on the Fletton brick relates to the ‘kiss’ mark often observed on their stretcher faces. This is a result of the way they are stacked within the kiln.

Tunnel kilns operate on the opposite principle to Hoffman kilns as the fire remains in the same location and the bricks pass through the kiln slowly on a car or trolley. In a similar fashion to the chamber kiln, gases are recycled from the cooling zone to reduce the necessary consumption of fuel. Most stiff bricks and some soft bricks are now fired in tunnel kilns. Further reading regarding manufacturing and firing is available in Ilston (1996) and Jackson (1996).

Properties/specifications

As mentioned previously, there are prescriptive guidelines within the UK/EU for the specification and description of clay brick masonry products (BS EN 771-1: 2003). The code specifies a division of clay masonry units into two main categories (based on their density above 1000 kg/m^3 respectively) and whether they are destined for use in protected or unprotected conditions), namely low-density (LD) and high-density (HD) units.

There are several minimum parameters that must be specified when describing either LD or HD clay masonry units. These are the unit type, the dimensions and tolerances and in the case of LD units the gross dry density. HD units must also include a freeze–thaw designation (DD CEN/TS 772-22: 2006). Parts 1, 2, 3, 5, 7, 9, 13, 16 and 20 of BS EN 772 specifically refer to parameters required for clay masonry units. These are discussed only briefly below; the reader is recommended to consult the relevant parts of the code should more precise guidance be required.

The UK specification for clay masonry units, BS EN 771-1: 2003 states that the dimensions of clay bricks must be specified by the manufacturer (in terms of work size) as length × width × height. The most common size of bricks in the UK is 215 mm × 102.5 mm × 65 mm. The units must be assigned a tolerance category to indicate the maximum possible deviation from the work size, i.e. the stated dimensions. Both LD and HD units have specific tolerance categories that they can fit into, in addition to specified ranges between different units.

In the case of LD units, the gross dry density must be specified and should be less than 1000 kg/m^3. A tolerance category shall also be specified (typically 10 or 5%). For HD units, the density should be specified but there is no value stated in the code.

HD units which will be exposed to climatic conditions will be assigned a freeze–thaw category. The specification for the test to determine whether they are suitable for passive, moderate or severe classification is currently being drafted for development status (DD CEN/TS 772-2: 2006). The test involves the construction of a test panel and its exposure to 100 repeated freeze–thaw cycles between 20 and −15°C; the units are then assessed for damage and a category (passive, moderate or severe) is assigned. It should be pointed out however that the behaviour under freeze–thaw is as much a function of the masonry layout as it is the individual units.

The configuration of the clay units should include the shape and features such as frogs and perforations. The direction and extent of the void volume should be specified, as should the thickness of shells (the peripheral material between a perforation and the surface of a masonry unit and webs (the solid material between a perforation and the surface of a masonry unit (BS EN 771-1: 2003; BS EN 772-16: 2000)) and the frog size, which should not exceed 20% of the unit.

The compressive strengths of bricks will always be specified where the unit is intended for structural purposes. The guidelines for crushing specimens to failure are detailed in (BS EN 772-1: 2000), and the manufacturers must specify the method used. The capping or grinding of the units to achieve plane parallelism, if required, should be stated. Plywood of thickness no greater than 6 mm may also be used to ensure concentric loading.

The compressive strength test involves the application of a uniform load at a rate which depends on the expected failure load. When the load applied is equivalent to 50% of this expected failure load the rate is adjusted so that failure is then achieved in under a minute. The data should be reported as the mean of several units in N/mm^2, i.e. the peak load divided by the cross-sectional area of the units. The normalised compressive strength may also be reported, where relevant. Manufacturers will specify whether the unit is labelled category I or category II. A category I unit is one which has a specified compressive strength and the percentage chance that it will fail to reach that strength is lower than 5%. A category II unit does not comply with such specifications.

The initial rate of water absorption is relevant to clay units because it reflects their behaviour when combined with mortar to form clay masonry; units with high water absorption will require a mortar with a high water/cement ratio (see Chapter 32). The test method is specified in Part 11 (BS EN 772-11: 2000) and in short requires the drying of specimens to a constant mass followed by their immersion in water, which is 5 mm ± 1 mm deep, for 1 minute. The change in mass is measured, accounting for the surface water which is removed before weighing. The initial rate of water absorption value is expressed in $kg/m^2/min$.

Water absorption properties over longer periods of time are also important when considering exposed units and units which are to be used in damp-proof courses. In the case of units externally exposed, the test requires that the dried units are entirely submerged in water for 24 hours and the water absorption value is expressed as a percentage in terms of the change in mass of the unit after 24 hours. In the case of units to be used for damp-proof courses, the test is similar but it involves the boiling of the units for 5 hours (BS EN 772-7: 1998) and again measuring the change in mass (the boiling water test gives a higher value of absorption as the boiling action achieves greater penetration through pressure). In both tests, the bed face of the units should be clear of the base of the container in order to ensure good circulation of water about all faces of the specimen. This is achieved by the use of spacers.

The active soluble salt content of clay masonry units can be determined using BS EN 772-5: 2001. This is specified to indicate the potential for salt damage of mortars or to the bricks themselves (from cryptoflorescence (see Chapters 34 and 37)). It can also help guide the user on whether to expect the bricks to exhibit efflorescence (a whitish deposit on the surface of the units resulting from the crystallisation of sodium, potassium or magnesium salts which can occur following construction or after being exposed to the environment and a series of wetting and drying cycles). Efflorescence is mainly only an aesthetic problem (see Chapter 34 Defects in Masonry). Once determined, the units are assigned a category (S0, S1 or S2) which relates to the total percentage by mass of sodium, potassium and magnesium. The test to determine the active soluble salt content can involve either a wet chemistry titration or an instrumental, spectroscopic technique. In either case, the solid sample must be crushed to a specified size and the soluble salt content extracted for quantification.

For LD units, the moisture expansion can be determined using guidance provided in BS EN 772-19: 2000. This guidance only applies to horizontally perforated (perforations which are parallel to the stretcher face, i.e. in the length dimension) units which have at least one dimension greater than 400 mm and with a shell thickness of less than 12 mm. There is no guidance available for HD units in BS EN 771-1: 2003; however, BS 5628-3: 2005 does recommend that the maximum reversible expansion will not exceed 0.02% and that the irreversible expansion of the units depends on the age of the brick and the clay type. These comments on the expansion of clay units are based on guidance provided in CP121 Part 1: 1973 (now withdrawn). For example, bricks manufactured from Weald clay will expand up to 0.08% during the first 2 days after they are removed from the kiln. Units manufactured from Carboniferous Shale will expand up to 0.07% during the period from 3 days after removal from the kiln to 128 days. (The moisture movement of units and masonry is discussed in Chapter 37.)

Concrete masonry units

Introduction

Aggregate concrete units have to comply with BS EN 771-3: 2003 (dense and lightweight) and autoclaved aerated concrete units with BS EN 771-4: 2003 and the test methods to determine the required properties are listed in these documents. As mentioned previously, the units are now categorised according to their vertical void percentages (Groups 1 to 3) and horizontal percentages (Group 4). In the UK, Group 3 and 4 units are not commonly manufactured. Units currently described as ‘solid’ fall into Group 1 ($\leqslant 25\%$ formed vertical voids by volume). Cellular and hollow blocks will fall into Group 1 or Group 2 ($>25\% \leqslant 60\%$ formed vertical voids by volume), depending on their void content. Aerated concrete units all fall under Group 1 as do manufactured/reconstructed stone units which must comply with BS EN 771-5: 2003 (these can be considered ‘concrete’ units).

Aggregate concrete units are typically available in two standard face sizes: 440 mm × 215 mm (length × height) and 390 mm × 190 mm (length × height). To obtain the coordinating dimensions, the specified mortar joint thickness (normally 10 mm) is added to the length and height face dimensions. The width of each of these two sizes of unit ranges from 75 mm to 215 mm (440 × 215 face sizes) and 90 mm to 190 mm (390 × 190 face sizes). As before, the size of the unit must be specified in the order of: length: width: height. Hence a 440 mm × 215 mm face unit which is 100 mm wide will be specified as 440 mm × 100 mm × 215 mm. This is a change to the previous method of specifying units (pre BS EN 771-3: 2003) which was length × height × width.

Aggregate units are extremely versatile. They are suitable as the inner load-bearing leaf of a standard cavity wall, where clay units act as the facing leaf. They can also act as both leaves of the cavity wall where the external leaf is rendered or not. (If not rendered, units can be manufactured with significant architectural appeal to be left exposed.) Solid walls can also be built using hollow/

cellular/solid units laid flat or collar-jointed. They are also suitable as partitions and internal walls. All of these applications reflect the units' high level of performance in terms of their load-bearing, thermal, fire and acoustic properties. In many ways, aggregate blocks have now largely superseded clay units in most structural applications within the UK.

Manufacture of concrete units

Materials

Aggregate

The principal component of concrete units is the aggregate, typically plus 90% by volume of the unit. Aggregates mostly act as a chemically inert filler material but nonetheless have an effect on the density, strength and elastic properties of a unit. As a result, different aggregates will be selected depending upon the desired end use.

Grading the aggregate content is a crucial parameter to the properties of an aggregate concrete unit. Dense aggregate blocks tend to have well-graded constituents. This means that there are fewer voids in the matrix as the smaller particles fill the gaps. Lightweight aggregate blocks tend to utilise a 'gap' gradation whereby there are fewer particles of a lower size and the voids between the large particles are left unfilled. The shape of particles also clearly affects their ability to pack against one another. The aggregate size in these blocks can sometimes be up to 14 mm, however they are normally 10 mm down. Autoclaved aerated concrete units contain siliceous based fine aggregates or a pulverised fuel ash (PFA) slurry.

In the manufacture of concrete aggregate units it is usual that at least two different types of aggregate will be used. These aggregates will possess their own 'natural' grading and therefore by combining appropriate aggregates it is possible to achieve a reasonable approximation of the required grading (continuous or gap). However, they can also be separated into specific gradations artificially using sieves of different aperture size.

Aggregates can be grouped according to their density and specific gravities. Normal-density aggregates are often obtained from natural sources such as gravels and sands, or crushed from bulk sizes in the case of rocks. When bound with cement, they produce a matrix with densities in the range 1800–2100 kg/m^3 (dense aggregate block units). Some natural aggregates which are merely extracted, washed and graded tend to have more uniform, rounded shapes. This affects their packing behaviour within the cementitious matrix and also has an influence on the properties of the hardened material. Crushed aggregates have far more angular and irregular shapes.

Lightweight, low-density aggregates are the most widely used for aggregate concrete blocks. This is historically due to their ease of handling but there other benefits such as a reduction in self-weight and better insulating properties due to a larger amount of air voids. However, this comes at the expense of the overall strength of the concrete. Lightweight aggregates are usually artificial in origin although materials such as pumice could be used. Manufactured or 'by-product' aggregates include:

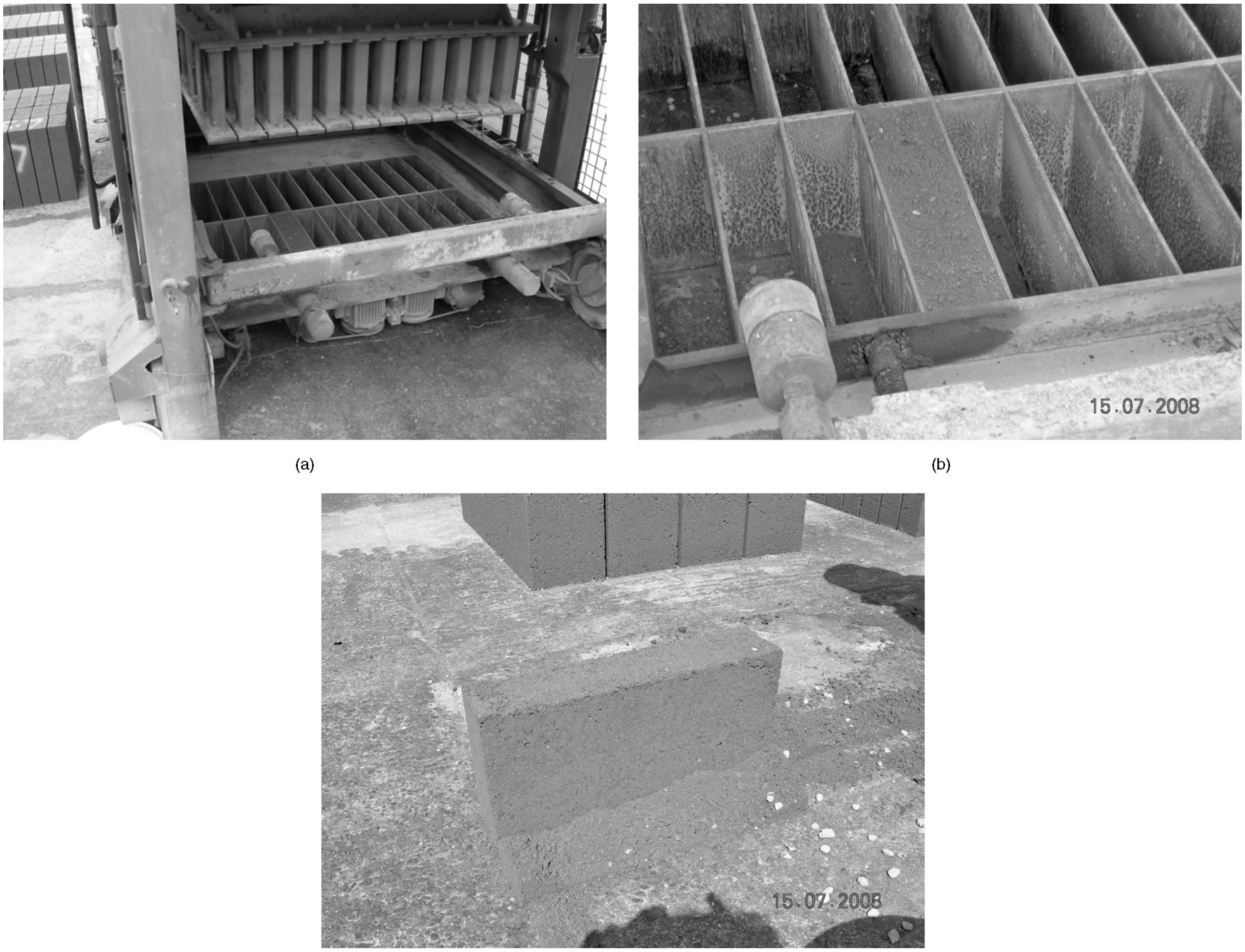
- furnace bottom ash
- sintered pulverised fuel ash (PFA), a processed by-product of coal fired power stations
- expanded clay or shale
- foamed blastfurnace slag.

Although all these artificial aggregate products are processed to have a lower density and specific gravity than dense aggregates, they do have different properties. Blocks produced using lightweight aggregates generally have a lower elastic modulus and increased creep and shrinkage behaviour. The current drive to make concrete units more sustainable has seen the wider use of secondary or recycled/waste aggregates, however furnace bottom ash and blastfurnace slag have been used for many years.

Cement

Cement is a mixture of inorganic materials which will harden when combined with water by way of a chemical reaction known as hydration. Cement is a binder, that when mixed with aggregates and water binds the aggregates together into a coherent matrix which will become stiffer with time once the cement has set. There are other materials that can be considered cements such as glues or organic cements which harden via alternative processes; these are not to be discussed here.

Portland cements are the most widely used cements for aggregate concrete blocks. They are produced from a mixture of clay and limestone raw materials, which contain the necessary minerals: calcium, silica, alumina and iron. Essentially, the manufacture of cement involves the grinding and mixing of these clay and limestone materials and the reaction between these materials at high temperatures (~1400°C) in a kiln to predominantly form, by way of the fusion of oxides found within the raw materials, a number of silicate, aluminate and aluminoferrite compounds which, upon cooling, are known as a 'clinker'. The clinker material is then ground to a very fine powder with a small proportion of gypsum, which acts as a control on the rate of hydration of the silicates and ensures that the aluminate (tricalcium aluminate) is unavailable for reaction. The key compounds involved in the hydration of the cement are the calcium aluminates and silicates that are formed during the clinker production. A complex series of both concurrent and subsequent reactions occurs, the most important of which for strength development is the formation of a calcium–silicate–hydrate (C–S–H) gel.


(a)

(b)

(c)

Figure 3 Example of an egg-layer: (a) 'Egg-layer'; (b) one mould 'cell' filled with material and compacted; and (c) unit de-moulded and left on the yard to cure

Cement chemistry is a complex and widely researched field – further reading is available in Bye (1999), Taylor (1997) and Ramachandran (2001).

Manufacturing process

Aggregate concrete blocks

The most widely used method for the production of aggregate concrete units is the use of static moulds into which the constituent materials are poured and then vibrated and pressed using a tamper. The material is then rapidly extruded on to pallets and removed on a conveyor belt for curing. The mould can be refilled automatically. Typically this kind of production involves the use of multiple moulds for batch processing of units.

An alternative technology used to process large amounts of aggregate concrete units is an 'egg-layer' (see **Figure 3**). These machines generally operate outside. They are fed with the cementitious mix and then used to press a large amount of units (either 22 or 44 units per batch). The 22/44 cell mould is then raised and the machine moves on to press another batch, leaving the just-formed units outside on the surface of the yard for up to five days, in order to gain sufficient strength to allow them to be mechanically handled and stacked ready for delivery.

Curing of aggregate concrete blocks can be achieved both at ambient (either externally as above or internally) and elevated temperatures. The use of steam chambers for elevated temperatures may be to shorten the duration of curing or to increase the strength of the units. Drying ovens with maximum temperatures of up to around 75°C may also be used depending on the manufacturing plant (the raised temperature is often due to the consequence of heating pipes which are positioned near to the internal drying areas).

Autoclaved aerated concrete blocks

The manufacture of autoclaved aerated concrete units utilises a chemical reaction between the constituent materials prior to and during high-pressure steam curing to form a cellular, aerated structure. The resulting products have a low density and good insulating properties due to a high content of air voids (at least 40% by volume).

The production of aerated blocks involves the casting of a slurry containing a siliceous material (i.e. ground sand) or PFA, a cementitious and/or lime binder and water into moulds. Aluminium powder is then added a short time after this initial mixing, which reacts with the hydrated lime and the free water to evolve hydrogen which causes the mix to foam and rise with a cellular structure. This hydrogen ultimately escapes to the atmosphere and is replaced by air. Once the mixture has been cured for a short period of time, the moulds can be removed and the aerated concrete material can be cut to size. The units are then heat treated (steam cured at high pressure) in an autoclave where the lime reacts with the fine ground aggregate to form calcium silicate, which accounts for a major portion of the material's strength as well as contributing to its other properties. Quicklime and gypsum may also be added to the mix prior to the introduction of the aluminium powder. The lime hydrates and produces a rise in the internal temperature of the mix (the reaction is exothermic). The unit density and strength are controlled by the strict management of the reaction rate of the quicklime with both the water and then the aluminium powder. Aerated autoclaved concrete blocks can be used immediately after curing as long as they have cooled to ambient temperatures.

Specifications/conformity

In the UK/EU, there are prescriptive guidelines for the properties that either aggregate (BS EN 771-3: 2003) or aerated (BS EN 771-23: 2003) concrete blocks must meet, and a general guide for the specification of both types (BS 6073-2: 2008). For conformity, the manufacturers must specify these properties and any relevant additional properties in the description of their product.

An exhaustive list of parameters exists in the aforementioned guidelines (Parts 1, 2, 6, 11, 13, 14, 16 and 20 of BS EN 772 apply to aggregate concrete units and Parts 1, 2, 10, 11, 13, 15, 16 and 20 to aerated concrete units). Manufacturers need to state that their product meets these guidelines and report some general basic characteristics, some of which are listed and discussed briefly below:

- unit type
- work size dimensions
- tolerance category
- configuration
- mean compressive strength
- dry density
- moisture movement.

Annex C of BS EN 771-3: 2003 contains a figure (Figure C.1) of the various aggregate concrete unit types. However, interestingly, none of the examples illustrate a UK block shape (they are more relevant to mainland EU).

As mentioned previously, the work size dimensions of concrete units may vary. **Table 2** shows the work size dimensions of aggregate concrete blocks.

The tolerance category of a unit is an indicator of how significant the deviation in actual sizes can be. Most aggregate concrete blocks produced in the UK conform to tolerance categories D1 (+3 and −5 to all dimensions) or D2 (+1 and −3 to length and width and ±2 for height). Other tolerance categories exist, however they are not used within the UK (tolerance D4 is intended solely for units which are to be laid with thin joint mortar, a technique which is uncommon in the UK). Aerated concrete blocks also have similar specific limits to deviations.

Previously, configuration was discussed in terms of the percentage vertical voids and the four groups which describe these ranges of percentages. However, although unlikely to be applicable to the UK market, configuration also covers flatness requirements for the faces of facing quality units and (in conjunction with the D4 tolerance category) for the bed surfaces of units to be used with thin layer mortar as well as a requirement for the parallelism of bed faces of units to be used with thin layer mortar.

Manufacturers must also specify compressive strengths for the units they produce. There are a range of compressive strengths suggested in the code (BS EN 771-3: 2003; BS EN 771-4: 2003). However, manufacturers will tend not to produce blocks of all the strengths specified. Aggregate concrete units can either be tested dry or saturated; the conditioning of the unit prior to testing must be specified. Compressive strength values have therefore been increased in the codes by about 4% and what was a '3.5 Newton' or '7 Newton' unit will now be a 3.6 N/mm^{-2} and a 7.3 N/mm^{-2} unit, respectively. Aerated concrete blocks will typically have lower specified compressive strengths. There is a geographical preference for compressive strength required by users, i.e. in Yorkshire the preferred strength for an aggregate block is a 7 N/mm^{-2} unit.

The gross dry density of the units should be specified in terms of kg/m^3. Aggregate blocks generally fall in the

Length: mm	Width: mm						Height: mm
390	n/a	90	100	140	190	n/a	190
440	75	90	100	140	190	215	215

Table 2 Work size dimensions of aggregate concrete blocks

range 650–2200 kg/m^3, and aerated blocks in the range 400–800 kg/m^3. Aggregate blocks have a permitted deviation of ±10%, and aerated blocks ± 50 kg/m^3.

The moisture movement (shrinkage) behaviour of both aggregate and aerated units may be specified. The tests for aggregate blocks (BS EN 772-14: 2002) and aerated blocks (BS EN 680: 2005) operate under the similar principle of examining dimensional changes in samples under controlled water-absorbing and drying conditions. However, the conditions and the measurements taken are different. The aggregate block test may use whole units or partial units, and it involves the soaking or drying of samples and the measurements are taken from gauges attached directly to the surfaces. The aerated blocks are not tested whole but instead prisms are cut from whole units and gauges are used to measure the length change. Previously, the maximum shrinkage of aggregate concrete units is specified as 0.06% and for aerated concrete units it is 0.09%. However, these limits no longer apply in the Eurocodes.

If the masonry units are to be used in structural applications, the shear strength should be specified. The test (BS EN 1052-3: 2002) involves the application of a four-point load to specimens (typically cut from larger samples) until failure. There are two main procedures, one of which involves the application of a determined pre-compression load and one where there is zero pre-compression. The test is actually a shear bond strength test. In practice, the results are of little use as the bond shear strength is affected by many factors which are site specific and therefore cannot be guaranteed by the manufacturer.

Calcium silicate masonry units

Calcium silicate masonry units are manufactured to BS EN 771-2: 2003 and must conform with Parts 1, 2, 9, 10, 13, 16, 18 and 20 of BS EN 772. The units are typically available with dimensions of 215 mm × 102.5 mm × 65 mm (length × width × height); they are predominantly frogged units and fall into either Group 1 or 2 (i.e. less than 60% vertical voids by volume). They are very versatile units being suitable for facing purposes, foundations and as common units and may be specified as either LD or HD units. As an HD unit, their freeze–thaw resistance must therefore be determined (BS EN 772-18: 2000).

Calcium silicate units are manufactured from a mixture of lime and siliceous aggregate with a minimal amount of water. Typically, the aggregate is fine sand. The materials are well mixed and compacted at high pressures to produce ‘green’ bricks with reasonable strength for handling and curing and with well-defined arrises and faces. The curing of calcium silicate bricks is similar to that of aerated concrete blocks, i.e. high-pressure steam autoclaving (for these units 8–16 bars pressure and temperatures of 160–203°C are used) that results in the formation of the hydrated calcium silicate species tobermorite, which is responsible for the strength of the unit.

Manufacturers must report at least the dimensions, tolerances, compressive strength, gross dry density and configuration of the calcium silicate masonry units (BS EN 771-2: 2003). The units typically have strengths higher than 5 N/mm^2 and up to approximately 60 N/mm^2 (i.e. equivalent to medium- to high-strength clay bricks (Class B Engineering bricks)). They also possess good acoustic properties.

Calcium silicate bricks are virtually free of soluble salts so efflorescence is not a problem. They are available in a range of colours (the use of pigments can easily be used to colour the brick which is otherwise white in its final manufactured state) and the light-reflective qualities of the light colours can make the unit very popular with architects.

Dimensioned natural stone masonry units

Stone masonry is historically extremely popular due to its durability and appearance. There are three main types of stone available for construction in the UK: igneous, metamorphic and sedimentary. Sedimentary stone, formed by the layering and compaction under self-weight of sediment material on the beds of lakes, seas or rivers, is by far the most commonly used in the UK. Limestone and sandstone are examples of widely used sedimentary stone.

There are various classes of stone masonry units, however of these only dimensioned natural stone units satisfy the code definition of a masonry unit (BS EN 1996-1: 2005) and these must comply with BS EN 771-6: 2003. The denomination of the stone masonry unit (i.e. the traditional name of the material, the petrological family, the typical colour and the place of origin) should be specified in accordance with the standard procedure (BS EN 12440: 2008). The description and designation requirements of natural stone units are in line with the other unit types discussed above; where water absorption is being determined and the stone is clearly anisotropic, the test should be performed by submerging the face which will be horizontal during use. If no arrangement of use is established by the manufacturer then two values, normal and parallel to the bedding plane, should be obtained.

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Chapter 32

Mortars

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This chapter covers the background and history of mortar; the constituents: cements, limes, sands and other minor components; mortar requirements, varieties, properties, testing and use; and the major elements of supply, production and manufacture and site practice.

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Scope

This chapter covers mortars used for laying masonry units, generally those composed of brick and block but which may also be of any other appropriate materials, e.g. stone, glass, etc.

For convenience, these mortars will be known by the generic name of masonry mortars, and are distinct from plastering, rendering and floor screeding mortars which are not covered in this chapter. Specialist polymeric mortars, generally based on epoxy resins, polyurethanes or other polymers are also excluded from the scope.

Introduction and historical background

Mortars are one of the most ancient of building materials, but one capable of use in the most up-to-date of structures. They are durable and economic yet flexible, both in design opportunity and above all in execution and use on the building site.

There exist examples of mortar dating back many millennia. Although there is no definite date agreed upon for their origin, examples are known dating back to about 8000 BC. That much of the early mortars endure today, albeit in ruined form, is a testament to their durability.

Today, as in past years, mortars are relatively easy to produce and use, and they utilise raw materials that in elemental form are most abundant in the earth's crust and geology.

Mortar types

Although within the UK the term mortars is generally taken to cover those based on Portland cement, a number of different binders and mixtures of binder are used in practice and these are briefly considered below.

Gypsum mortar

Gypsum is widely used as a binder in plastering mortars and in some Continental European countries internal blocks use a gypsum, rather than Portland cement binder. These then use gypsum-based mortars. At one time the latter were widely used, and some Paris dwellings still remain that were constructed with these mortars, both externally as well as internally, as do traces of similarly based mortars in the Great Pyramids, although gypsum-based mortars are not generally suited to external use. A few Georgian and early Victorian buildings remain in the UK that are rendered with gypsum-based materials.

However, gypsum is not stable in the presence of water and degrades; thus historic stucco render was always coated with oil-based paint to provide a protective layer. Gypsum masonry mortars may therefore be used in exceptionally arid climates and internally, in the latter case protected against wetting through condensation of atmospheric moisture vapour. These potential uses are so restricted that they will not be considered further in this chapter.

Lime mortar

There is a documented use of lime mortar which again dates back through millennia, but terminology in this field may be confusing, as the terms ‘lime’ and ‘lime mortar’ may have different meanings and interpretations. In order to address the issue accurately, the different types of lime need to be defined, as outlined in detail later in this chapter.

In general, however, it is important to distinguish between hydraulic lime, which reacts with the mixing water to form a weakly hydraulic (often known as cementitious) product, that is a binder with the properties of finite, measurable compressive, flexural and bond strength, and hydrated lime. The latter does not react with water to form a cementing compound, but over a long time, in the order of years, may gradually react with the carbon dioxide in the atmosphere, to form a weak compound, calcium carbonate.

This process, known as carbonation, is thus beneficial in mortars, although generally regarded as undesirable in, for example, concretes.

Towards the middle of the twentieth century it appeared that there was some confusion between the properties, uses and applicability of these fundamentally different types and, as they were becoming less used, a lack of real understanding became widespread. In addition, their slow rate of gain of strength, particularly in cold weather conditions, was not always successfully integrated into the construction programming, and in 1973 they were removed from the building codes. At the time of writing, however, the use of appropriate types of lime mortars is undergoing somewhat of a resurgence.

Pozzolanic mortar

The precise definition of a pozzolan or pozzolana is surprisingly complex, but in principle these materials generally rely on another reactive component, perhaps lime or Portland cement, in order to react and gain strength.

Pozzolanic mortars were widely used in ancient times, and many famous structures used them and are still in existence today, albeit ruined or restored; the Parthenon in Rome is one well known example, and the name derives from the town of Pozzuoli, near the volcano of Vesuvius. Similarly to lime-based mortars, pozzolans appeared to undergo a renaissance in the late twentieth and early twenty-first centuries, and a large amount of concretes and mortars produced in Europe today contain at least some pozzolan.

Ash was used together with lime to produce pozzolanic mortars, particularly during the industrial revolution in the UK, when large amounts of industrial ash were produced as a by-product of various furnace-based processes, particularly in industrialised city areas.

Twentieth century and later Portland cement-based mortars

Following on from the invention of Portland cement, patented by Josef Aspdin, a Leeds bricklayer, in 1824, these cements rapidly displaced other binders for use in mortars, until in the latter part of the twentieth century they were used in virtually all situations. However, cost and other issues were gradually responsible for an increasing use of pozzolans and limes, generally, but not always, in combination with Portland cement.

This meant that as well as mortars composed solely of ordinary Portland cement and sand, mixtures of cement, lime and sand became widely used, with these mixes often containing slag or ash in addition, as further reactive components.

The requirements of mortar

The requirements of mortar may be separated into two categories, each important to one category of user but often given insufficient thought by the other. These categories are the hardened properties, of clear and obvious concern to the specifier, and the wet, user or plastic properties, of importance to the operative. Unfortunately, the operative often fails to consider the requirements of the specifier, which will usually encompass those of the overall building client, while the specifier rarely considers the operative's needs in terms of adequate ease of use on the actual building site. Rationalising/integrating these two categories of need is essential to ensuring successful construction, and each category is considered separately below.

The hardened properties/requirements

The hardened properties are effectively those properties that enable structural performance, adequate durability and conformity to national/supranational building, environmental, health and safety and any other statutory instruments.

Taking as an example one national requirement, they are required to achieve adequacy in the areas of loading, ground movement, fire, sound and thermal properties. In effect, these may be listed as a series of properties and appropriate test methods, with that of satisfactory appearance an additional need. All of these properties are considered in detail later in this chapter.

Acceptable performance is also required in the area of durability, as an adequately strong mortar may lack long-term structural integrity and durability. Long-term plastic deformation or creep is also of interest, although mortars in general use in the Western world appear satisfactory in this respect.

The final hardened property, appearance, is usually achieved satisfactorily by a combination of careful selection of basic raw materials and the addition of pigments.

The plastic properties/requirements

These requirements are effectively those perceived by the bricklayer (hereinafter 'mason', as blocks, stone and other products, as well as bricks, are then covered in the generic term).

The mason requires something that he often refers to as workability, by which is meant the sum of the working properties, not, as in concrete, the slump, flow, consistence or similar. The different properties that together go to make up this property of 'workability' are considered later in the chapter.

Cements for mortar

Portland cement, so called originally by an early producer because it was said to resemble Portland stone, was widely used for mortars in the early and middle twentieth century in the UK, either on its own, or blended with hydrated lime to improve the working properties and perhaps to reduce the strength of a Portland-cement-only mortar, which may have been excessive when used with some weaker masonry units.

As the drive for European standardisation started in the late twentieth century, British Standards Institution (BSI) in the UK published a draft British Standard for cement, DD ENV 197, followed by the BS EN 197-1: 2000, which was the full UK version of the European Standard EN 197. After a short 21-month transition period, the old British Standard for cement, BS 12: 1996 was withdrawn.

The new standard permitted a much wider range of cements to be used, which is more in line with some Continental European practice. Overall, the range of permitted cement compositions and types has increased hugely, but with experience of many of these in the climatic conditions of the UK not being available. However, the Codes have, wisely, only initially permitted the use of a relatively small number of these, with properties that are not very dissimilar to those of the old BS 12 cements.

The key difference between the old and newer cements is that previously, in conformity with BS 12, cement was composed primarily, that is at least 95%, of cement clinker in the cement 'nucleus', which comprised cement clinker, other main constituents (invariably pozzolans) and minor additional constituents. Thus, the material was virtually pure cement clinker, although in addition a small percentage of calcium sulfate as gypsum was used to control set.

In the new and current Standard, BS EN 197-1: 2000, such a cement is known as CEM I. There are now, however, four other main types of cement included in the Standard, to give five in all, as follows:

CEM I	Portland cement
CEM II	Portland–composite cement
CEM III	Blastfurnace cement
CEM IV	Pozzolanic cement
CEM V	Composite cement.

When this standard was issued there was doubt on the part of some users as to the practicality of producing mortars with cements other than the known CEM I, for which experience of use over many years was available. However, within a few years of the inception of the standard the use of pure, unblended CEM I became almost non-existent, and the use of CEM II was widespread, with little or no reported problems.

CEM I cement

As discussed above, the use of this material in unblended form in the UK had virtually ceased by the year 2009, as it was no longer available in bags, and economic reasons had led the producers of factory-made mortars to use blends that conformed to the requirements of CEM II, as discussed later in this section.

Portland limestone cement

This is a form of blended cement composed of clinker and finely ground limestone, either mixed or ground together, and the standard contains two types, with two different maximum limestone contents of 20% and 35% respectively. The former had been used in substantial amounts in factory-made mortars by the year 2009 without any known reported problems having been ascribed to it.

Masonry cement

In the middle of the twentieth century, UK cement manufacturers produced materials known as masonry cements, which contained approximately 75% of cement clinker, so-called inert material, usually ground chalk or limestone, and an air-entraining admixture to enhance the working properties and the durability. Towards the end of that period, one UK producer followed the relatively common practice in North America and produced a masonry cement that used hydrated lime instead of inert filler and this was still in production at the time of writing.

Apart from the specialist masonry cement type outlined above, production of masonry cement in the UK was low, with little usage, partly occasioned by the unfavourable logistics associated with delivery and stockholding of a material in low amounts, and usage declined until by the early twenty-first century there was little or no production in the UK.

Improved cement

Although not standardised, this term refers to Portland cements that are modified in production for ease of use and supplied in bags for the small building and DIY markets. The cement contains an air-entraining agent for ease of use and is marketed for applications such as, for example, garden/patio concrete repairs and small new works, general minor repairs, small areas of brickwork, patching, etc.

The materials are not ideally suited for larger works, and certainly not for larger concrete pours.

Sulfate-resisting cement

This material was sometimes specified in the UK for concrete for use in sulfate-bearing soils, and for mortars either below damp-proof course in these locations or in other situations where it was thought that it might be exposed to deleteriously large amounts of sulfates. In terms of European practice, it was a uniquely British material and was not made in Continental Europe, where sulfate resistance was achieved by other means. At the time of writing, it is now no longer produced in the UK and it is probable that the UK standard, BS 4027: 1996, will be withdrawn although a new class of cements suitable for use in conditions where sulfation is thought likely is being developed for inclusion in the European standard.

White cement

This is produced by using specially selected raw materials that are low in the iron compounds that are responsible for the colour of normal cements. Their properties are no less adequate than those of grey cements, but if they are to be used in the UK they will have to be imported, currently often from either Spain or Norway.

Pozzolans

The ancient Romans used pozzolans, or pozzolanas as they are sometimes known, and their usage in both mortar and concrete has continued to the current time, yet an accurate definition of these materials is not easy to achieve. In general, they are thought of as materials that react with free lime to form a hydrate, and this definition usually suffices, although some pozzolans react with water, albeit very slowly, and developing little strength, without the necessity for an addition of lime or related compounds. Nevertheless, for all practical purposes, the definition stated above will suffice as the vast majority require free lime, and for those that hydrate without it the reaction is very slow.

Pozzolans result from a process of high temperature, hence, for the majority, the natural materials deriving from volcanic activity, and the artificial ones resulting from man-made activity associated with temperature. The latter include those commonly used in large amounts in the twenty-first century, as, for example, blastfurnace slag and pulverised fuel ash, and the general furnace ash widely used as a pozzolan in the previous century. Man-made pozzolans also existed, and were utilised, many years ago and crushed brick and tile, mixed with lime, are found in mortars within Roman remains.

Today, the majority of UK mortars contain slag or ash, either as a function of the cement used, or as a result of blending in a factory-made mortar plant. These blends have the potential to produce excellent mortars, with sulfate resistance that is better than that of the unblended cement and, in many cases, enhanced working properties.

The rather slower rate of strength development, when compared to the same cement unblended, is also quite appropriate for mortar.

There are many other materials that are known to exhibit pozzolanic properties, with some being used in specialist applications in concrete as well as mortar. Silica fume, a by-product of silicon or silicon alloy production, and metakaolin, a calcined clay, are examples that have established commercial application.

Limes for mortar

Until the invention of Portland cement in the 1800s, the mortar in most of the enduring masonry known to mankind was based on lime, pozzolan or a mixture of both. In the early twentieth century a large amount of masonry was based on lime mortars but by the end of the century much of the necessary knowledge appeared to have been lost and the UK codes of practice no longer included mortars based solely on lime as a binder.

In general, lime mortars have good working properties and develop strength more slowly than those based solely on Portland cement, which have gradually become faster in their rate of strength gain, primarily as a result of market pressures from the major users, ready-mixed and precast concrete producers. For mortars, a rapid gain in strength may be of assistance when the temperature shortly after placing the mortar falls to below freezing point, but it is not really recommended that bricklaying takes place in these conditions and, in other respects, a much slower rate of strength development is probably better suited to mortars. This is because some early life stresses may develop in masonry as a result of unit movement – expansion of bricks or contraction of concrete blocks. These are not necessarily harmful, and movement joints in masonry are used to accommodate the movements, but mortars that gain strength slowly are likely to be superior in these environments.

There is often confusion between the main different types of lime, quicklime, hydrated lime, lime putty and hydraulic lime, which a consideration of the production processes easily resolves. In the production of most lime, the basic raw material, which is usually chalk or limestone, is heated in a kiln. This drives off the chemically combined water to produce quicklime.

When a controlled amount of water is added to quicklime, a process known as slaking or hydrating, hydrated lime, sometimes known as dry hydrate, is formed. This slaking reaction is strongly exothermic and for that reason health and safety issues must be addressed in the process. (The intense heat generated is the reason why murderers of yesteryear were reputed to dispose of their victim's bodies in lime pits!) If the slaking water added is substantially in excess of that required to just hydrate the lime, then the product is lime putty or milk of lime, which produces better working properties than dry hydrated lime but is the same chemically.

Hydraulic lime

Notwithstanding the name, many hydraulic limes have more in common with cements than with hydrated limes, either dry hydrate or putty. While hydrated limes do not react with water to form a hardened compound, hydraulic lime contains other chemical species, in particular calcium silicates, which are very similar to those found in cement; indeed, cement contains a calcium silicate allotrope known as alite, whereas hydraulic limes contain belite, a very slightly different calcium silicate.

Portland cement contains a large proportion of alite, and much of its strength development occurs as a result of this compound hydrating. Hydraulic limes contain some belite but in many cases not a large proportion, with the result that they set more slowly and achieve a lower strength. Because of this lower rate of strength development, early stresses exerted by moisture and/or thermal movement are more easily absorbed. The materials also set to a more porous form, so that internal condensation and precipitation of soluble salts may be less problematic, and as they have greater creep, medium- and longer-term movement stresses are better absorbed.

There are clearly a number of benefits accruing to the use of these materials but their slower rate of setting and strength development can also be problematic, particularly in cold weather conditions. It is often overlooked that historically, when these materials were used in the majority of buildings, masonry construction did not take place during cold, winter periods.

Hydrated lime

The function of this material is primarily to aid the working properties of the mortar, but it also carbonates over a period of time and therefore contributes a very minor amount to the overall strength of the mortar in the case of mixes composed of cement, hydrated lime and sand.

The enhancement of working properties with lime is considerable, and it will enable cleaner, coarser sands to be used to achieve good hardened properties yet still provide a mortar with good working properties. Without the use of lime, sands of this type may prove difficult to use in practice, although their gradings may conform to current standards.

Magnesian lime

This material is chemically similar to the calcareous limes discussed above, but with the exception that the calcium atom is replaced by the similar magnesium atom in the final compound, thus giving, for example, magnesium hydroxide rather than calcium hydroxide. It is used for building applications in the USA but is not processed for that purpose in the UK.

Lime putty

As outlined above, this can be the product of hydration with an appropriate amount of water. There is a lot of anecdotal evidence that the longer lime putty is kept or matured, the greater its beneficial working properties. The putty is said to ‘fatten’. There is probably a technical explanation for this in that a dynamic equilibrium is set up in the saturated solution within the putty, with material constantly dissolving and re-precipitating in either a finer or a more beneficially formed shape.

Laboratory work confirms this increase in working properties, but the extent may not be quite that sometimes quoted subjectively by users.

Sands for mortar

The vast majority of mortar sands are naturally occurring, having been laid down as sand deposits in one geological form or another, or occasionally originating as stone or rock that is reduced to sand-size particles artificially by a process of crushing or similar; however, sands may also be manufactured, when they are often referred to perhaps inaccurately as artificial.

Some manufactured aggregates are used in mortars, generally because they possess an attribute that is required for a particular special purpose. Thermal insulating properties are probably the most called-up requirement, with inorganic lightweight aggregates manufactured from slag or ash sometimes being used for this purpose. Alternatively, organic aggregates, such as, for example, polystyrene, may be used.

Recycled or secondary aggregates may also be used in mortars. With the former, the normal caveats with respect to cleanliness and lack of contamination hold good. Some specialist secondary aggregates have found relatively widespread use, with by-products from the production of china clay by high-pressure water monitors having been especially widely used, particularly in the UK West Country.

Within the terminology of mortar sands there exists a great deal of subjectivity and confusion, with recent changes to the British Standard probably serving only to exacerbate matters, as sands are now to be known as ‘fine aggregate’, not sand; the days of ordering a load of mortar sand are thus said to be numbered, with the request now said to be required to be for a load of fine aggregate – perhaps not something that will be easily accepted by UK site personnel!

Even without this latest terminological problem, there is already a lot of confusion engendered by terms in common usage to describe mortar sands. People talk of ‘sharp sand’ when they probably mean coarse sand, and ‘soft sand’ when they mean fine sand. It is suggested that for accuracy and avoidance of confusion the correct terms should be as follows:

Coarse sand:	Sand with large particle size, perhaps up to 4 or even 5 mm.
Fine sand:	Sand lacking coarse particles, perhaps nominally less than 2 mm, or even 1 mm for specialist pointing applications.
Sharp sand:	Sand containing crushed and/or angular particles.
Soft sand:	Sand with predominately rounded particle shape.

In the terminology which follows from that of the new British standard for mortar sands (BS EN 13139: 2002), the sand is described by the pair of sieve sizes in millimetres between which the particle sizes nominally lie. Thus a 0–2 mm sand would have a *nominal* particle size of between 0 and 2 mm. It is important to emphasise the word nominal, because 100% must pass a sieve size of *double* this nominal size, and 95% a sieve of about one and a half times the nominal size.

This is different from the earlier British Standard, where the conformity to a sieve size was absolute, so that rather than 'most' of the sand having to pass a stated sieve size, it all did. Thus, where 100% was required to pass say a 5 mm sieve size, it was all so required, but in the new European standard, for example, a 0–4 mm sand may have some particles, in this case 1%, that are larger than the largest sieve, which in this case is the 4 mm size, but there is a requirement that 100% must pass the 8 mm sieve size.

Sand for mortar then is characterised primarily by particle size and also perhaps by particle shape, although the latter is not covered by the standard. The maximum particle size requirement is a function primarily of joint size and operative preference, the minimum size primarily of strength, durability and other hardened properties.

The maximum size for ordinary masonry mortar, as opposed to, for example, thin joint or other specialist mortars, is thus in general effectively a function of operator preference, as it is unlikely that a sand particle will be so large that it actually obstructs the formation of the joint, which will usually be a nominal 10 mm in size. (The exception is the occasional presence of grossly oversized particles, resulting from contamination by other, large aggregate, perhaps in storage or transit – these being rather charmingly known as 'spacers' in one part of the country.)

In addition to operative preference, the specifier may use some engineering judgement in terms of the maximum size, or perhaps some aesthetic judgement which would be influenced by the visual effect of the larger sand particles.

The minimum sieve size at which the particle size is normally determined is the 63 µm size, which is often said to be the diameter below which the material is clay. This is not really correct, below 63 µm there will exist silt, which is the same general chemical composition as the majority of the sand, and also, in the small particle size range below 10 µm finer silt and clay, with most clay particles probably lying at the finer end of that size range.

As well as maximum and minimum sizes, the distribution of particle sizes in the intermediate ranges is also important. In broad terms, an 'even' distribution is desirable, with no excess of particles in one, narrow size range. Sands with this characteristic are known as 'single sized'. Sands with a more even, better distributed grading range are often known as well graded but caution should be exercised in using this term as it may be misinterpreted; in addition many specifications use this term virtually in isolation and effectively abrogate responsibility for a more complex but objective specification, that can be accurately interpreted and met – thus believing that if they specify that the sand shall be 'well graded' their specification function is discharged.

Particle shape also plays a part, with spherical particles providing the best working properties. On the contrary, angular, elongated particles tend to be more difficult with which to work. They feel harsh on the trowel and in many joint types can 'tear' the surface, leading to problems in achieving an acceptable finish.

So long as the grading and cleanliness are acceptable, the actual chemical composition of the sand is not an issue, provided that it is reasonably chemically inert. This means that either siliceous or calcareous aggregates are widely used, with other compositions not being excluded so long as they remain substantially inert.

Most mortar sands contain some impurities, with clay being the most common.

An amount of silt and clay will enhance the working properties and have little deleterious effect on the hardened properties but an excess will reduce strength and durability and increase shrinkage. The maximum is set at between 3 and 8% for naturally occurring sands; for crushed rock it is much higher, on the premise that the majority of the fines in a crushed rock will comprise fine rock as opposed to clay, and it is only the latter that is particularly harmful.

A number of impurities may be present from time to time as contaminants in sand, and excessive amounts of these have the potential sometimes to cause problems in the resultant mortar. Coal and lignite, an immature form of coal, are relatively commonplace. These can give rise to brown staining and to pop-outs in the surface of the mortar.

Iron pyrites gives rise to very similar symptoms and it is sometimes only by examining the residual particle at the base of an eruption or the start of the stain that the causative material can be confirmed. Although similar in effect, the incidence and occurrence of these two materials is quite different. Many sand sources potentially contain pyrites but there is often no indication of this and, more importantly, no occurrence, as the pyrites tends to be present only at or below the lower bound of the deposit and is therefore completely avoided if the dig is carried out with care. The problem with these deposits, however, is that they may be effectively pyrites-free for many years, as the dig is carefully carried out, but may then suffer a sporadic outbreak that goes unguarded because there is no current reason to suspect the presence of the material.

Lignite may also appear sporadically, but is often reasonably constant, sometimes even being present in such large amounts, and so uniformly, that it is a contaminant in every load. In these cases, there may be instances where new equipment is installed that is said to be to remove the lignite, but this is often not totally effective and sometimes also washes away a lot of the sand fines, with the result that subsequent mortar is lacking in cohesion, plasticity and other related working properties that are seen as beneficial by the mason.

The actual acceptability of mortar that contains these impurities varies greatly geographically. In places where it is difficult to obtain completely lignite-free sand, for instance in parts of the north-west of England, minor lignite staining may not always be viewed too seriously, but in parts of the UK where it does not usually occur, for example in London, even a very small occurrence will be taken very seriously.

Chlorides may also be regarded as impurities, but the amount is important as they are present to some degree in all sea-dredged sands; most large, western UK cities, for example Cardiff, Bristol and Liverpool have a large number of masonry structures built in sea-dredged mortars and in some places it was the predominant sand. It is generally agreed that modest amounts of chloride are acceptable, but that large amounts may require more consideration. The mortar sand (aggregate) standard (BS EN 13139: 2002) refers the reader to the mortar standard (BS EN 998-2: 2003), and states, for guidance, that 'the chloride ion requirements of EN 998-2 are usually achieved when the chloride ion content of the aggregate does not exceed 0.15% for plain mortar and 0.06% for mortars with embedded metals'.

The mortar standard itself states 'The chloride ion content shall not exceed 0.1% Cl of the mortar by dry mass'. These values are in the same order.

The issue of potential efflorescence is often raised in the context of chlorides, but as alkali metal chlorides tend to be very water soluble, e.g. sodium chloride (common salt), calcium chloride, any efflorescence composed of these compounds would only be of a very transient nature and would readily disappear with weathering.

Mica is sometimes found in mortar sands, particularly in those produced in the south-west as by-products of the production of china clay.

Admixtures

The vast majority of mortars contain admixtures, and even where these have not been specified by the architect or engineer they invariably appear in the final mortar mix. In general this is because they are added on site or in the mortar factory. Within the UK, over three-quarters of mortars are believed to be factory made and all of these are air entrained, with the possible exception of a very small number of mixes that may from time to time be specially ordered without air entrainment, although the resulting mixes for this small minority will have reduced durability and inferior working properties. Ready-mixed set retarded mortars by definition contain cement set retarders, and are also air entrained.

Of the remaining site-made mortars, most have air entrainers added at the mixer and some use cements that already contain these materials.

Air-entraining admixtures

As discussed above, these are used in the majority of mortars. Several decades ago there was occasionally a view that the use of these materials was not always desirable, and there were instances of specifications which specifically excluded their use, but this is now rare if not non-existent, and it is clear that the view was based on erroneous opinion, although in gross excess they may be harmful (in common with many things).

Air-entraining agents provide several benefits. Freeze–thaw resistance is greatly increased, indeed, apart from the very rich and often excessively strong mortars that are rarely needed, it is doubtful if the majority of mortar mixes for use in exposed applications can achieve satisfactory durability to cycles of freezing and thawing without the inclusion of entrained air, a similar situation to that of concrete, where air entrainment is mandatory in, for example, pavement-quality concrete which is used externally in a saturated environment.

There is sometimes a view that air entrainment is inappropriate for lime-based mortars but examination of historic lime mortars very often reveals traces of natural air entrainment and of the presence of numerous entrained air voids.

As well as improving durability, air entrainment greatly aids the placing of the mortar and the bedding of units so that the contact area between the mortar and the units is more complete. The units are easier to place accurately and it is easier and quicker to make any small final adjustments to line and/or level. All of these matters are likely to lead to better bond, although from first principles, excess air at the mortar–unit interface will clearly tend to reduce bond. In general a maximum value of no more than about 20% offers a good compromise. Constraining air content unduly, or even specifying that no air entrainment be added thus carries the risk that due to the reduction in plasticity and ease of use the overall work of the mason will be affected, and the resulting work will be inferior; conversely, excessive air entrainment should also be avoided.

Non-air-entraining plasticisers

Because air-entraining agents increase the plasticity of the mix, they are sometimes known as plasticisers, but other materials also impart added plasticity, although generally not to the same extent as air entrainers. These materials often work by dispersing cement agglomerates, and/or affecting the residual surface charges to the small particles, primarily the cement, in the mix.

Retarders

These materials, more correctly cement set retarders, prolong the workable life of the plastic mortar by delaying the set of the cement. After preparation, the mortar exhibits no set whatsoever for the selected period of retardation time but then sets in a similar way to a normal, unretarded mortar. A wide variety of materials are used as retarders, and it is difficult to point to one particular type as being preferred.

If the cement set retarded mortar is used in the period of time prior to cessation of retardation, then the suction of the units being laid will cause the commencement of set to take place immediately following laying, in the same way as a normal mortar, thus truncating the retardation time.

Testing of the final, hardened mortar reveals no difference in properties from a ‘normal’, unretarded mortar, although it is often found that the final strength is actually a little higher.

Viscosity modifiers

These materials are relatively recently developed and enable the properties of a mortar to be modified in a more complex manner than simple plasticisers. They can affect the plastic viscosity and/or the shear value, which are the two key rheological values that dominate plasticity and workability; they have the ability to enhance plasticity at higher values of consistence, without segregation taking place.

Viscosity modifiers may be based on cellulose ethers or polysaccharides, both very long chain but relatively simple molecules, or on acrylic or related polymers.

Mortars that are heavily modified by the use of these materials may have the ability to retain their workability in demanding situations, as, for example, when used with very high suction units which may tend to suck the water out of a mortar rapidly, leading to a very short plastic working life and perhaps to difficulty in adjusting the final position of a masonry unit after laying. Again, these admixtures do not appear to have any deleterious effects on the mortar.

Waterproofers

So called waterproofers, more correctly water repellers or integral water repellers, generally act by lining the pores in the mortar that are accessible to water with a hydrophobic material, by blocking the pores or by a combination of these two processes. Their applicability and value however is not always as it may appear at first sight, because the vast majority of water ingress through masonry occurs not through the general matrix of the mortar but through major cracks or fissures and the pore blocking/ water repelling actions are only effective in the very small cracks. Adding waterproofers to materials that will form cracks or fissures in practice thus has no effect on water ingress in these cases, although it may be beneficial in the absence of cracking defects.

Waterproofers do have an application in rendering mortars, where inclusion in the first (backing) coat will reduce its suction to successive coats. This means that when the next or finishing coat is applied it remains workable and relatively fluid for longer, allowing it to be worked accurately to the desired finish. This then represents their useful application, in general they have no current application within the UK in making masonry less permeable to water if added to a masonry mortar.

Fibres

Fibres are sometimes added to specialist mortars, in particular to some renders. They have been found of use in some backing coats but if used in the final coat, while technically efficacious, mar the aesthetics as they are obvious on the surface, resembling a haired finish, and completely spoil the appearance.

Water

Water is not really an admixture but is placed in this section for convenience. Most water specifications either quote the new European standard for water for concrete (BS EN 1008: 2002), or specify ‘potable’ water. The standard is complicated, and in practice drinking water will always be suitable for making mortar. Naturally occurring water, particularly from ponds or lakes, should not be used without first arranging for tests for acceptability to be carried out.

Pigments

Much mortar is pigmented, from the red Roman mortar that owed its colour to the inclusion of crushed tiles and clay pots, and in which colour may or may not have been seen as an attribute, to contemporary mortars that are often specially pigmented, usually with iron oxide pigments, to produce coloured mortars. These are purpose-made for pigmenting cementitious and related building products and have little or no effect on the mortar when used in the amounts that are normal, and that suffice for visual effect. At these levels the effect on the hardened properties is so small that it usually cannot be measured with statistical certainty. A lot of specifications constrain the maximum amount of pigment, as in large amounts the added fines so represented will reduce strength, but at the much lower levels that suffice to obtain satisfactory colour there is little or no measurable effect.

Iron oxide pigments are supplied in a wide variety of shades, from yellows, through orange or 'marigold' to light and then dark reds, light and then dark browns, purples and blacks. In reality, yellows, reds and blacks are generally produced in the primary pigment production, with browns, purples etc. being produced by mixing red and black, perhaps also with yellow.

Carbon black pigment is a very intense black but is likely to be unstable and to result in colour loss. In general, dark pigments that are very intense should be viewed with suspicion as they may well contain carbon black mixtures.

Pigments based on other than iron oxide are available. Titanium dioxide produces a white, although if the basic sand colour is intense it will only be lightened, not made truly white, or even off-white in the case of very well-coloured sands. In these cases, if a white colour is desired, it is probably better to use a white sand in the first instance.

Stable blue and green pigments are also available, although these are probably aesthetically suited only to smaller, specialised, internal applications.

The properties of mortars

The mortar properties that first come to mind are almost always the hardened properties, with a large number of specifiers considering only compressive strength. There are of course a number of other hardened properties that are important, and if the wet or plastic properties are poor it may not be possible for the mason to lay the units effectively and in such a way as to achieve the potential of the desired hardened properties. The hardened properties and then the plastic properties are considered hereafter.

Hardened properties and testing

Compressive strength is both an important property and also one that is the most frequently measured, without excessive difficulty or apparent need for specialist facilities, as the methods appear to have much in common with those used for testing hardened concrete. Although the previous British Standard (BS 4721: 1981) allowed test specimens to be either cubes, of varying sizes, or prisms of 100 mm in length, the new test method standard (BS EN 1015-11: 1999) permits only 150 mm long prisms, which are not usually used for concrete testing.

The mortar standard (BS EN 998-2: 2003) is primarily a designed, as opposed to a prescribed standard, and suggests a range of compressive strengths, as shown in **Table 1** below, although these are only examples and any other strength may be declared by the manufacturer.

Class	M1	M2.5	M5	M10	M15	M20	Md*
Compressive strength: N/mm^2	1	2.5	5	10	15	20	d*

Note: * Where d is a compressive strength greater than 25 N/mm^2

Table 1 Mortar classes (adapted from BS EN 998-2: 2003)

Thus in this, the main body of the standard, the requirement is for a designed mortar, based on determination/declaration of compressive strength.

There is also a National Annex, and this takes account of prescribed mixes in addition to designed ones, by reference to a second table, as shown in **Table 2**.

The key wording in the standard attached to **Table 2** is that 'The following relationships, based on the prescriptive mix proportions historically used in the UK codes of practice and standards can be regarded as authoritative'.

Unfortunately, as may be seen from **Tables 1** and **2**, the compressive strengths in the main body of the standard, those shown in **Table 1**, are different from those in **Table 2** of the annex. Nevertheless, the new standard has been in use for several years at the time of writing, with only three real problems arising, as discussed below.

First, the strengths given in the annex to the new standard differ from those in the main body of the standard. Second, the assumed equivalence in strengths in the old standard and those of the new standard, as shown in **Table 2**, may not always be achievable, particularly with all of the modern cements. Third, and perhaps most importantly, the *assumed* **Table 2** equivalence may be taken, in error, to mean that when a prescriptive mortar in accordance with the table is tested in compression, then its strength is required to be no less than the values shown. This was not the intention of the table and, if this rigid interpretation is erroneously made, it effectively changes a prescribed requirement into a quasi-designed one, that is a prescribed mix but with designed requirements in addition.

It is generally considered undesirable to mix designed and prescribed requirements in the same document, as complexity and potential for lack of integration inevitably occur. In addition, there are reasons for specifying performance criteria, and other reasons, that may differ, for specifying prescription, and each concept and ideal is

BS 4721: 1981 – Mix designation	EN 998-2: 2003 – Mortar classes
(i)	12
(ii)	6
(iii)	4
(iv)	2

Table 2 Mortar mix designation (adapted from BS EN 998-2: 2003)

likely to be irrevocably undermined by confusion with the other concept.

In addition to compression, other mortar strengths are also of interest. Both standards provide for the determination of the flexural strength of mortar by spanning across two bearers and loading vertically downwards in the centre through a third member, followed by a subsequent double determination of compressive strength, once on each prism half.

In the current mortar standard (BS 998-2: 2003), bond strength is addressed as the initial shear, in accordance with BS EN 1015-3: 2002, and this is said to be in the present absence of a direct bond strength test, and the standard also has provision for a declaration of bond strength taken from a standard table which is in Annex C of the standard.

There is, however, a method long established in the UK for determination of the flexural strengths of masonry wallettes, in which strengths are determined parallel to and at right angles to the bed joints, but the specimens are of a much larger size than the small ones used for shear bond. As with all masonry bond strength testing, the method does have some variability which means that a number of samples are required for a result with some statistical reliability, and this is one of the drivers leading to the development of a method that uses smaller specimens and thus appears to be easier and more convenient. Unfortunately, however, it has so far been the case that these more convenient methods are less accurate, so that overall there is no benefit.

One possible exception to the observation that so far easier, cheaper methods for the determination of bond are more variable and/or less reliable, may be in the case of the bond wrench test, which has shown some promise and could perhaps be adopted more widely. In this test procedure, an arm is clamped to a bonded brick and a weight placed on the end of the arm to exert a bending moment through the lever arm, thus allowing a bond strength to be calculated. Currently, though, the approved EN/BS EN reference method for mortars is shear bond.

The modulus of elasticity and the long-term creep properties are of some importance, particularly with regard to movement and possibly longer-term absence of cracking and it may be that some lime-based materials are beneficial in these areas.

In addition to strength, the new standard has a requirement for water absorption due to capillary action of hardened mortar, which is appropriate to mortars for external use. This is simply determined in accordance with BS EN 1015-18: 2002 by immersing a hardened prism of mortar in water for a predetermined period of time and weighing the amount absorbed.

Water vapour permeability is also addressed for mortars for external use, and the standard requires a minimum value but there is no need for a test to be carried out as there is provision to 'declare' or quote a standard value, taken from a document that is referenced in BS EN 1015-19: 1999.

The thermal conductivity is dealt with in the same manner, for mortars for which it is relevant, by requiring a declaration to be made by reference to a specified document.

Density also has to be declared, but only 'where relevant for the use for which the mortar is placed on the market', and only in terms of a range, which effectively means that it is necessary for lightweight mortars.

Finally, with respect to hardened mortar, the durability is clearly of great importance, with resistance to deleterious reaction with soluble sulfates and freeze–thaw resistance being key issues.

Reaction with sulfates is dependant on the availability of sulfates, of water in which to dissolve them, and on potentially reactive cements.

Reactive sulfates generally derive from the masonry units, soil (groundwater), or some other extraneous source, so that avoidance by way of this aspect of the problem is related to specification of units with safe levels of sulfates, careful detailing and consideration of specification of materials in conditions that are unavoidably permanently damp or wet, for instance areas below the damp-proof course.

A compound called tricalcium aluminate, C_3A for short, which is present in Portland cements, is an important part of the reaction with sulfates to form tricalcium sulfoaluminate, which is a much larger molecule; the reaction is thus strongly expansive, leading to the generation of potentially disruptive forces. Sulfate-resisting cement, which has a low level of C_3A and is stable and effectively unreactive in the concentrations of sulfates found in practice, has traditionally been used where sulfate attack may be an issue, but this is now no longer produced in the UK.

Nevertheless, the inclusion of pozzolanic materials such as slag or ash, to produce CEM II cements, has proved to assist in resisting sulfation. In addition, the level of C_3A in the cements used in the UK has declined substantially in the recent two decades. These two factors are thought to be key factors in the decline in sulfation problems experienced in the UK in recent years.

A meaningful yet relatively simple and economic test for durability has been sought for many years, but unfortunately the search has so far proved elusive.

Simple tests based on freezing specimens of hardened mortar are frequently proposed but, although relatively inexpensive, they are difficult to relate accurately to the performance of 'real mortar' in practice. Casting mortars in moulds, removing these, and then subjecting them to cycles of freezing and thawing rarely duplicates the actual performance that is found to be obtained in practice. This is due to a variety of reasons, but means that the quest for a simple yet meaningful test, that may be accurately related to actual performance on site, is much more difficult than it is sometimes thought to be.

Current work showing great promise uses small brickwork or blockwork assemblages, often called wallettes, which are wetted and cooled from one face, thus simulating actual building environment conditions much more closely than specimens of mortar only, but the economics and timescale are still issues, although the test itself appears to show a very good relationship with on-site performance.

There is a further requirement for the specialist thin joint mortars that are manufactured for the purpose of laying autoclaved aerated concrete blocks. These are accurately sized and finished to an accurate plane surface and therefore may be laid with very thin joints, which confers structural benefits and also aids thermal performance if mortars of standard density, that is of non-lightweight nature, are used. For these mortars, the maximum aggregate size permitted is 1 mm, a recent reduction on the 2 mm originally specified in BS EN 998-2: 2003.

The plastic or wet properties and testing

The plastic properties of the mortar may generally be regarded as being those that are perceived by, and of interest to, the mason, and in the initial terminology there may perhaps be an issue of definitions. This is because in mortar technology the word workability refers to the sum of the plastic properties perceived by the mason, that is the sum of the flow or consistence (loosely speaking the ‘runniness’), the cohesion, the adhesion, the propensity or otherwise to segregate and bleed, the resistance to absorption into a high suction background and, perhaps most importantly, the ‘feel’ of the mortar ‘on the trowel’. In contrast, workability with respect to concrete refers to the flow or consistence.

Thus the workability of a mortar is really the sum of the mason’s perception of the material; this is of key importance, as the mason clearly places every trowel-full by hand, as opposed to, for example, most concrete where the operative contacts a much lesser amount of the bulk material.

The mason may therefore almost be said to be testing each trowel-full, and thus has the potential to form a view of the properties of the mix and potentially air that view. As many masons are paid in proportion to the amount of bricks or blocks that they lay, they are not usually inhibited from airing their views if they deem the mortar properties to be less than optimum, particularly if this causes their rate of laying, and thus their remuneration, to slow down.

Unfortunately, some masons also seek to modify the plastic properties of their mortars by adding unauthorised material to the mixes: domestic detergent, in the form of washing up liquid, being a common choice. This entrains air, which improves the working properties but, in excess, air is known to reduce strength, both in compression, tension and, probably most importantly in the case of mortar, in bond. In addition, it has been suggested that domestic air entrainers can cause white staining to completed mortar joints.

Of the plastic mortar properties, the most readily perceived and easily adjusted is therefore the flow, i.e. the *workability* in concrete, but not in mortar, terminology. The retention of workability when the mortar is in contact with a unit of high absorption/suction is known as the water retentivity. This test was a part of the old British Standard BS 4721: 1981, but when the new European Standard (BS EN 998-2: 2003) was introduced, the water retentivity test was omitted. However, because the UK deemed it still to be of value, and it was not in conflict with the new European Standard, the test was retained, and is now part of another British Standard (BS 4551: 2005).

There is also a requirement for workable life, given in BS EN 1015-9: 1999, which is based on a test that was part of the old BS 4551.

There is no requirement for a maximum air content in the new standard, unlike the old British Standard, and this reflects the designed, performance approach of the former and the prescribed approach of the latter; although there is a need for the air content range to be declared by the manufacturer.

The maximum chloride content is constrained, where relevant, which effectively means where there may be embedded metal and external conditions, but the permitted value is such that sea-dredged sands won from around the shores of the UK and relatively well drained to remove surplus water, for instance being stocked in reasonably large-sized stockpiles on hard standing, should have no difficulty in conforming.

While BS EN 998-2: 2003 is primarily a performance standard, with requirements for minimum compressive strength, it does, as discussed previously, also allow for prescriptive mixes and there is the statement that requires ‘publicly available’ references to be used to establish a relationship between mix proportions and compressive strength, which refers to **Table 2**.

There is also a requirement to declare the reaction to fire classification of the mortar; in general, that is for mortars with an organic content of less than 1%, they are classified as Class A1, without the need to test, but if they contain more than this amount they have to be tested.

For use only with thin joint mortars, there is a specialist test for workable life known as the correction time test, rather than the standard test for workable life.

Appearance

Although perhaps not always a matter of prime concern for the engineer, to many ultimate clients, those who commission or purchase buildings, appearance is of key importance. The mortar in a brickwork façade represents about 15%, in the order of one-sixth, of the total masonry area, but is often overlooked aesthetically until the masonry is being built, when it may belatedly become an issue.

The appearance of mortar is affected by four major issues – materials, workmanship, design and weathering – and these are considered below.

All of the major raw materials affect the colour, with the sand probably the predominant factor in most cases, and with mixes containing lime, although lighter in overall colour, a change in lime source, within one type, is unlikely to affect the colour greatly.

The cement colour does affect the overall colour of the mortar, with sulfate-resisting cements, where these are still available, being darker in colour. While there may be a substantial difference from works to works, within one cement works there is unlikely to be a substantial batch-to-batch variation.

Blended and/or CEM II cements tend to reflect the colour of their components. Limestone cement is generally a lighter colour, and the light, off-white colour of slag and the grey of pulverised fuel ash are reflected in blended cements that utilise these materials.

A further factor that may need to be considered is the increase in cross-border trading in cements, which means that materials from an increasing number of sources may now be imported into the UK.

Sand colour has a major effect on the final mortar colour. Some UK sand types, although characteristically coloured, remain quite consistent within one source, while others change markedly. Thus, oolites are often most consistent within one quarry, as are the vast and deep Permian deposits of the North-East, but some river gravels change markedly throughout one deposit; one river terrace deposit in Sussex changes from a deep red to a bright yellow within a few metres across the dig, which necessitates the production of small stocks of each material in the ‘as-dug’ state prior to blending at the washing and processing plant.

As the overall colour of the sand is often due predominately to the fines present, washing will often reduce the depth of colour and the within pit variation.

Within the UK, broad sand colour trends may be associated with geographical areas, although there are some exceptions. Thus, in general, sands in the Cheshire region are red, further north in Lancashire brown, yellow in the Permian beds of the north-east, red or red-brown in Scotland, yellow in London and the South East, and so on. Interestingly, in the local areas with distinctive colours, all of the existing mortars relate to the existing base sand colours and form the locally accepted genre, with different colours not always being readily accepted in that region.

Workmanship has a major effect on the sand colour. Accuracy of gauging is clearly important but the consistence at the time of laying, and of joint finishing, is also a major factor, as wetter mixes are lighter in colour.

The consistence of the mortar as mixed generally varies little throughout a contract, so long as the same units are being laid, as they dictate the consistence that the mason chooses to use. However, the joint is not finally finished or tooled until some time after the units are originally placed and the suction or rate of absorption and the ambient temperature and humidity affect the ‘stiffening’ or consistence loss of the mortar prior to tooling, and it is the final consistence at that time that has the predominant effect on the colour. It is thus sometimes the case that very dense and low-absorption bricks, which remove little water by suction from the mortar, are associated with lighter-coloured joints.

Thus continuity of tooling time throughout a contract is of importance and the unit moisture content should also be considered, with brick, block and stone stockpiles kept covered in accordance with recognised good practice.

The amount of laitance present in a typical joint type is also of relevance. A recessed joint will have all of the laitance struck off, but, for example, a smooth faced, worked joint will tend to contain a meaningful amount of surface laitance, and will therefore tend to a lighter colour than a recessed joint made with the same mortar and units, in the same overall conditions.

Saturation of mortars, particularly those that are newly laid, may cause soluble material to be dissolved and, on subsequent drying, a white surface stain of calcareous material may deposit on the joint surface. This surface stain, sometimes known when it occurs on concrete structures as efflorescence, but more correctly, with mortars, as bloom, will have a white or off-white appearance and thus will lighten the surface colour of the affected joint.

Following the advice given in the Code of practice (BS 5628-3: 2005) with respect to covering and protecting newly erected brickwork/masonry will avoid the early occurrence of bloom.

The weather and ambient climatic conditions after completion of construction and during the life of the structure is also of relevance to the appearance of the mortar. Constant saturation may lead to the appearance of bloom later in the life of the structure, and the aim of architectural detailing of elements such as roof overhangs, copings and similar protective details is to protect the structure from excessive saturation, which may be detrimental to the function and long-term durability as well as to the appearance. Thus the design as well as the local climatic conditions are of importance in this respect.

The durability of mortar

It is clear from the examples of historic mortar that still exist, even after the passage of thousands of years, that it has the potential for great durability and, viewed in the context of the current HM Government assumed design life of a masonry house of a mere 60 years, durability should never be an issue.

This is indeed generally the case, but there are occasions where longevity expectations are not fully achieved, perhaps with some degradation of properties taking place. Adequate design and material selection can always overcome these issues, which are associated with either incorrect design/specification and/or inadequate materials or execution.

The Code of Practice (BS 5628-3: 2005) should always be considered at design stage and this will provide design details appropriate to particular exposure conditions as well as advice on material specification. For mortar to exhibit adequate durability, the constituents and the mix proportions should be appropriate for the specific application.

In broad terms, the code suggests the use of stronger mixes, higher in binder, for applications requiring greater durability, but this advice is only general and represents somewhat of a simplification. In addition, there is anecdotal evidence, not yet codified, that some very weak lime-based mortars have excellent durability, but the overwhelming empirical weight of evidence currently seems to be that binder type and amount are critical, with the use of air entrainment being a very important factor.

Experience suggests that adequate air content is of great importance, particularly with cementitious mortars.

Mortar production and sourcing

Historically, mortars were mixed on the building site, but increasingly they are now partially or completely factory made. Site mixing still remains a practical possibility, but care should be taken to ensure that the product is of adequate quality and that workmanship and/or other labour-related issues do not result in an inadequate final product.

As a first step to ensuring the quality of mortar, the raw material storage should be considered. If the mortar is a factory-made material then this will be taken account of and will be monitored by the factory production control criteria in operation. With site-mixed mortars, sand should be stocked on a hard standing – a concrete base or similar, and sheeted to avoid rain ingress and subsequent segregation. Bagged materials, for example dry bagged mortar, cement and/or lime, should be stored in a dry place, a cement shed or similar, and used in rotation in the order of delivery, care being taken that newer loads are not stocked in the front of the storage and used first, with the result that 'dead' stock remains at the rear of the store.

Where site mixing together of separate constituents is necessary, these should always be properly measured, or 'gauged' to use the relevant terminology. Gauging by weight is to be greatly preferred but, in its absence, volume gauging is often acceptable, although purpose provided containers must be used. It is not an uncommon practice to gauge by volume using shovels full of material, perhaps one shovel full of cement to three, four or six of sand or lime:sand mortar. This practice is to be deprecated as different materials occupy different volumes, individual operatives may use differently filled shovels, perhaps of different sizes, and the moisture content or degree of compaction is also a major factor. Gauging should always use containers that are dedicated for the purpose but these need not necessarily be too expensive – stainless steel buckets are used with success.

Storage is much easier for factory-made mortars, the varieties of which are briefly considered hereafter.

Premixed lime sand mortar, often called premix and in past years rather erroneously known as ready-mixed mortar, should be stored on a hard standing and sheeted. It is advantageous if the surface of the stockpile is first smoothed down to minimise the area of the surface that is available to potentially react with the carbon dioxide in the atmosphere and to carbonate on the surface. It is especially important to sheet pigmented mortars to ensure that rain does not wash the pigments from the surface of the stockpile, thus causing a colour variation.

Lime:sand mortar, once very widely used in the UK is now, however, a minor part of the market, having been supplemented, or perhaps virtually superseded by, ready-mixed cement set-retarded mortar and silo mortar, which are both discussed below.

Cement set retarded, wet ready-to-use mortars are delivered by volume and discharged on the site into receptacles of accurately known capacity, generally 0.2, 0.25 or 0.33 m^3. These containers should be covered with an impermeable sheet of plastic or similar, to reduce drying by sun and subsequent evaporation.

In common with all other mortar types, extra water should not be gauged in to the mortar and the product remixed when it begins to set in the vessel; it should be discarded if this occurs.

Mortar may also be delivered in a silo and there are two distinct forms of these. Multi-compartment silos are split vertically so that in plan they have the appearance of a pie chart. There are usually two compartments, the largest for wet or damp sand, the smallest for cement. In some forms there is a third, small compartment for lime, which thus enables the production of a mix of cement, lime and sand, although the majority of split silos have only two compartments. Whatever the number of compartments, the lower silo outlet end has a water inlet line, a power supply connection and a small integral mixer so that when switched on the sand and binder feeds begin to operate, materials flow into the mixer which then mixes and, as it is in the form of a continuous screw, screws the final, mixed, mortar by way of a discharge chute into a convenient receptacle, commonly a wheelbarrow or similar.

The advantage of split silos is simplicity of operation and thus of cost, as the sand does not have to be first dried, thus avoiding the cost and complexity of this operation. The disadvantage is that the wet sand may not always flow smoothly and as reproducibly as desired, and the final mix ratios are achieved by volume batching, which is always less accurate than weight batching.

Single silo mortars are produced by drying sands and weigh batching with the binder(s) and any other constituents; the final, mixed mortar is used on site from the silo, which has an integral mixer and discharge screw at its base and a water inlet, in the same way as the split silo, but which clearly uses fully weigh batched and gauged materials.

Mortars may also be fully ready mixed and dried but delivered to site and stored in bags, which are then mixed, with the addition of water, and used. This system is popular in some continental European countries but its use in the UK is generally limited to smaller and DIY projects. It produces mortar that is as precise as that from single silos but with a lot of empty, torn paper packages as a by-product.

Site practice and mortar

As discussed above, the raw materials for mortar should be properly protected and stored, used in rotation, and gauged accurately, preferably by weight.

Unauthorised admixtures should never be added to the site mixer.

Mortar that has began to set should not be gauged with further water and remixed, a practice known as retempering.

Masonry should not be laid at very low temperatures, when there is a possibility of below freezing temperatures, and the Code (BS 5628-3: 2005) gives detailed advice in this area.

Newly erected masonry should be protected against saturation by rain by covering with polyethylene sheet or similar but this should be arranged to be clear of the face of the masonry, rather than in contact.

In conclusion, mortar requires attention to only a few areas in order to ensure successful execution and a long, maintenance-free life.

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Chapter 33

Masonry wall design

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CONTENTS

The aim of this chapter is to introduce the basic approach to the design of masonry walls which is adopted in Eurocode 6. It considers the three stress states of compression, shear and tension (from flexural behaviour) and illustrates how to determine the design value of resistance of the masonry to vertical, shear and lateral actions. The guidance is appropriate to the design of clay, concrete aggregate, calcium silicate, autoclaved aerated concrete, manufactured stone and natural stone masonry. However, for simplicity, guidance may occasionally be restricted to only the design of clay masonry.

Introduction

The governing codes for the design of masonry are Parts 1 to 3 of Eurocode 6, namely:

BS EN 1996-1-1: 2005 General Rules for Reinforced and Unreinforced Masonry Structures
BS EN 1996-1-2: 2005 Structural Fire Design
BS EN 1996-2: 2006 Design Considerations, Selection of Materials and Execution of Masonry
BS EN 1996-3: 2006 Simplified Calculation Methods for Unreinforced Masonry Structures.

As with all Eurocodes for the different structural materials, Eurocode 6 is intended to be used with Eurocode 0 Basis of Structural Design and Eurocode 1 Actions on Structures. It has been developed to enable the designer to use the six common unit types: clay, calcium silicate, aggregate concrete, autoclaved aerated concrete, manufactured stone and natural stone.

The most common forms of unreinforced masonry walls are load-bearing walls, non-load-bearing walls or infill walls to framed structures although the latter should be checked for any applied lateral loads such as wind and differential air pressure. Load-bearing walls are typically defined as continuous from foundation to roof and support loads from each floor level and the roof. Non-load-bearing walls can typically be the external leaf of a cavity wall system (i.e. the cladding to the inner load-bearing masonry leaf). When constructed as an infill panel, they are typically a solid/single leaf wall, providing compartmentation or acting structurally by providing additional shear capacity to the structural frame, see earlier comment regarding lateral loads. They may of course act in both ways and this highlights one of the benefits of masonry: it can provide a variety of functions. As a single-leaf wall it can also act as a separating wall in domestic structures or as a partition wall in industrial buildings; however, it is possibly most common today in the form of a cavity wall.

The structural design of masonry requires an understanding of the material under various stress conditions, namely compression, shear and tension. Compressive stresses are apparent in masonry subjected to vertical actions (or loads). In the UK, shear and tensile forces result from horizontal or lateral pressure from the wind. The design of masonry must also consider its performance in other areas as well, i.e. thermal, acoustic, fire, impact and durability. The intention of this chapter is to provide an introduction to the design of masonry only in terms of its resistance to vertical, shear and lateral loading. The other design requirements are beyond the scope of this chapter. (Reference should be made to Chapters 2 and 3, respectively, for more complete descriptions of masonry unit and mortar properties referred to in this chapter.)

Eurocode 6 is a limit state design code and therefore the basis of design when using this code is to ensure that the design value of the resistance, R_d, of the masonry is greater than or at least equal to the design value of the effect of the actions, E_d. In order to do this, it is first necessary to calculate the characteristic strength of the masonry. This characteristic strength is then divided by the appropriate material partial safety factor to give the design value of resistance which would then be compared by the designer with the design effects of the actions. However, the aim here is not to illustrate how to assess the actions (i.e. set of forces) that are acting on a structure and determine the applied actions (loads) (the 'design value of the effect of actions') on a typical masonry element, such as a wall, using the guidance provided in BS EN 1990: 2002 and BS EN 1991. Instead, it is to determine the 'design value of the *resistance*' of an element.

The general relationship is:

$$E_d \leqslant R_d \tag{1}$$

where:

E_d = design value of the effect of the actions
R_d = design value of the resistance.

$$R_d = R_k/\gamma_m \tag{2}$$

where:

R_k = characteristic value of resistance
γ_m = material partial safety factor.

The partial safety factors, γ_m, for use with masonry can be obtained from the National Annex to BS EN 1996-1-1: 2005. The factors are given in Table NA.1 of the National Annex and are reproduced here as **Table 1**. The following sections illustrate how to calculate respectively the compressive, shear and flexural characteristic values of resistance for a masonry wall and the subsequent design values of the resistance.

Characteristic values of resistance

Characteristic compressive strength of masonry

Guidance on how to experimentally determine the characteristic compressive strength of masonry is provided in BS EN 1052-1: 1999. It is no longer necessary to test storey-height panels. Instead, wallettes are used which are typically five courses high by at least two units wide (the wallettes are required to have at least one perpendicular joint in the middle of the width of the wallettes) for brick-sized units and three courses high by at least 1.5 units wide for block-sized units. A designer has the option of using values obtained from wallettes constructed from the units intended to be used on site or alternatively values determined from a large database available in the UK National Annex to BS EN 1996-1-1: 2005 can be used in Equation (3):

$$f_k = K f_b^{\alpha} f_m^{\beta} \tag{3}$$

where:

f_k = characteristic compressive strength of masonry, MPa
K = a constant, the value of which is given in the UK National Annex to 1996-1-1: 2005 (see **Table 2**)
f_b = normalised mean compressive strength of a masonry unit, tested in the direction of the applied actions, MPa
f_m = compressive strength of mortar and can be taken to be the same as the mortar strength class
α, β = exponents, the values of which can be obtained from the UK National Annex to 1996-1-1: 2005 and which are reproduced below.

Values for α and β for use in Equation (3) are as follows:

- $\alpha = 0.7$ and $\beta = 0.3$ for general-purpose and lightweight mortars
- $\alpha = 0.85$ and $\beta = 0$ for Group 1 clay masonry units, calcium silicate and aggregate concrete units of Group 1 and 2 and autoclaved concrete units of Group 1 in thin-layer mortar (bed joint thickness 0.5 mm to 3 mm)

	γ_M	
Class of execution control:	$1^{(A)}$	$2^{(A)}$
Material		
Masonry		
When in a state of direct or flexural compression		
Unreinforced masonry made with:		
units of category I	$2.3^{(B)}$	$2.7^{(B)}$
units of category II	$2.6^{(B)}$	$3.0^{(B)}$
Reinforced masonry made with:		
units of category I	$2.0^{(B)}$	(C)
units of category II	$2.3^{(B)}$	(C)
When in a state of flexural tension		
units of category I and II	$2.3^{(B)}$	$2.7^{(B)}$
When in a state of shear		
Unreinforced masonry made with:		
units of category I and II	$2.5^{(B)}$	$2.5^{(B)}$
Reinforced masonry made with:		
units of category I and II	$2.0^{(B)}$	(C)
Steel and other components		
Anchorage of reinforcing steel	$1.5^{(D)}$	(C)
Reinforcing steel and prestressing steel	$1.15^{(D)}$	(C)
Ancillary components – wall ties	$3.5^{(B)}$	$3.5^{(B)}$
Ancillary components – straps	$1.5^{(E)}$	$1.5^{(E)}$
Lintels in accordance with BS EN 845-2	See NA to BS EN 845-2	See NA to BS EN 845-2

(A) Class I of execution control should be assumed whenever the work is carried out following the recommendations for workmanship in ES EN 1996-2. inducing appropriate supervision and inspection, and in addition:

(a) the specification, supervision and control ensure that the construction is compatible with the use of the appropriate partial safety factors given in ES EN 1996-1-1;

(b) the mortar conforms to BS EN 998-2, if it is factory-made mortar, or if it is site-mixed mortar, preliminary compression strength tests carried out on the mortar to be used, in accordance with BS EN 1015-2 and BS EN 1015-11, indicate conformity to the strength requirements given in BS EN 1996-1-1 and regular testing of the mortar used on site, in accordance with BS EN 1015-2 and BS EN 1015-11, shows that the strength requirements of BS EN 1996-1-1 are being maintained.

Class 2 of execution control should be assumed whenever the work is carried out following the recommendations for workmanship in BS EN 1996-2. including appropriate supervision.

(B) When considering the effects of misuse or accident these values may be halved.

(C) Class 2 of execution control is not considered appropriate for reinforced masonry and should not be used. However, masonry wall panels reinforced with bed joint reinforcement used:

(a) to enhance the lateral strength of the masonry panel;

(b) to limit or control shrinkage or expansion of the masonry.

Can be considered to be unreinforced masonry for the purpose of class of execution control and the unreinforced masonry direct or flexural compression γ_M values are appropriate for use.

(D) When considering the effects of misuse or accident, these values should be taken as 1.0.

(E) For horizontal restraint straps. unless otherwise specified, the declared ultimate load capacity depends on there being a design compressive stress in the masonry of at least 0.4 N/mm^2. When a lower stress due to design loads may be acting, for example, when autoclaved aerated concrete or lightweight aggregate concrete masonry is used, the manufacturer's advice should be sought and a partial safety factor of 3 should be used.

Table 1 Ultimate limit-state partial safety factors for masonry materials (reproduced with kind permission from BSI)

Masonry unit	General-purpose mortar	Thin-layer mortar (bed joint $\geqslant 0.5\,\text{mm}$ and $\leqslant 3\,\text{mm}$)	Lightweight mortar of density	
			$600 \leqslant \rho_d \leqslant 800$: kg/m^3	$800 \leqslant \rho_d \leqslant 1300$: kg/m^3
Clay				
Group 1	0.50	0.75	0.30	0.40
Group 2	0.40	0.70	0.25	0.30
Group 3	A	A	A	A
Group 4	A	A	A	A
Calcium silicate				
Group 1	0.50	0.80	B	B
Group 2	0.40	0.70	B	B
Aggregate concrete				
Group 1	0.55	0.80	0.45	0.45
Group 1[C] units lad flat	0.50	0.70	0.40	0.40
Group 2	0.52	0.76	0.45	0.45
Group 3	A	A	A	A
Group 4	A	A	A	A
Autoclaved aerated concrete				
Group 1	0.55	0.80	0.45	0.45
Manufactured stone				
Group 1	0.45	0.75	B	B
Dimensioned natural stone				
Group 1	0.45	B	B	B

A Group 3 and 4 units have not traditionally been used in the UK, so no values are available.

B These masonry unit and mortar combinations have not traditionally been used in the UK, so no values are available.

C If Group 1 aggregate concrete units contained formed vertical voids, multiply *K* by (100 – *n*)/100, where n is the percentage of voids, maximum 25%.

Table 2 Values of *K* to be used with Equation (3) (reproduced with kind permission from BSI)

- $\alpha = 0.7$ and $\beta = 0$ for Group 2 clay masonry units in thin-layer mortar (bed joint thickness 0.5 mm to 3 mm).

A number of limitations on the use of Equation (3) are presented in the National Annex (2005):

- The masonry is detailed and constructed in accordance with BS EN 1996-1-1: 2005, Section 8.
- f_b should not be taken to be greater than 110 N/mm^2 for units laid in general-purpose mortar and not greater than 50 N/mm^2 in thin-layer mortar; the coefficient of variation of the strength of units should not be more than 25%.
- f_m should not be greater than $2f_b$ nor 12 N/mm^2 when units are laid in general-purpose mortar and 10 N/mm^2 when laid in thin layer mortar.

There are additional points in the National Annex that should be noted which relate to: (a) aggregate concrete units with the vertical cavities completely filled with concrete; (b) when the actions are applied parallel to the bed joints of the masonry; and (c) when the perpendicular joints are unfilled.

Also, for masonry made with general-purpose mortar, the *K* values must be modified if the wall is more than one masonry unit thick, i.e. a collar jointed wall (irrespective of whether or not the collar joint is filled) or an equivalent '8 inch' wall. In these cases, the appropriate *K* value obtained from **Table 2** above should be multiplied by 0.8.

Characteristic shear strength of masonry

The characteristic shear strength, f_{vk}, of masonry is a function of the characteristic initial shear strength, f_{vko} (BS EN 1052-3: 1999) and the design compressive stress perpendicular to the shear plane, σ_d, at the level under consideration (representing the compressed part of the wall that is providing the shear resistance):

$$f_{vk} = f_{vko} + 0.4\sigma_d \leqslant 0.065 f_b \quad \text{for fully filled perpendicular joints} \qquad (4)$$

$$f_{vk} = 0.5 f_{vko} + 0.4\sigma_d \leqslant 0.045 f_b \quad \text{for unfilled perpendicular joints} \qquad (5)$$

where:

f_b = the normalised compressive strength of the masonry units (BS EN 1996-1-1: 2005) for the direction of application of the load on the test specimens being

Masonry units	Strength class of general purpose mortar	f_{vko}: MPa		
		General-purpose mortar	Thin-layer mortar (bed joint $\leqslant 0.5$ mm and $\geqslant 3$ mm)	Lightweight mortar
Clay	M12	0.30	0.30	0.15
	M4 and M6	0.20		
	M2	0.10		
Calcium silicate	M12	0.20	0.40	0.15
	M4 and M6	0.15		
	M2	0.10		
Aggregate concrete, autoclaved aerated concrete, manufactured stone and dimensioned natural stone	M12	0.20	0.30	0.15
	M4 and M6	0.15		
	M2	0.10		

Table 3 Values of the initial shear strength of masonry, f_{vko} (reproduced with kind permission from BSI)

perpendicular to the bed face. The values of initial shear strength of masonry as given in the UK National Annex are reproduced in **Table 3**.

For walls subjected to shear forces, the design value of shear resistance is therefore given by:

$$V_{Rd} = f_{vd} t l_c \tag{6}$$

where:

V_{Rd} = the design value of shear resistance of the wall
f_{vd} = the design value of the shear strength of the masonry (the characteristic shear strength, f_{vk}, divided by the appropriate material partial factor for masonry, γ_m (see **Table 1**)
t = the thickness of the wall resisting the shear
l_c = the length of the compressed part of the wall (ignore any section of the wall that is in tension; assume a linear distribution of compressive stress and take into account all openings).

Characteristic flexural strength of masonry

The characteristic flexural strength of masonry is required to determine the ability of the masonry to resist lateral actions, i.e. loads perpendicular to the plane of the wall. The resistance of masonry to lateral actions is developed through a two-way action (except for free-standing walls, when the wall will act as a cantilever) and because of this behaviour the aspect ratio of a wall panel, its edge conditions (free, simply supported or continuous) and its resistance to bending in the two orthogonal directions of perpendicular and parallel to bed joints, all affect the magnitude of the bending moment carried in each direction.

This flexural strength approach, while being the most popular, is not the only approach. Another approach available is one based on arching and the assumption that a three-pinned arch will form within the wall.

As mentioned before, the magnitude of the bending moment carried by the panel depends on the edge condition and the correct assessment of this condition. While a free edge is easily identified, a degree of engineering judgement is required by the designer in deciding between simply supported and fixed conditions. For a wall containing a damp-proof course (dpc), full continuity across the dpc can only be assumed if there is sufficient vertical load on the dpc to ensure that the flexural strength capacity is not exceeded.

For a wall supported on three or four edges, the applied bending moment, M_{Edi}, can be taken as:

$M_{Ed1} = \mu \alpha_2 W_{Ed} l^2$ per unit length of wall for bending parallel to bed joints in f_{xk1} direction
$M_{Ed2} = \alpha_2 W_{Ed} l^2$ per unit height of wall for bending perpendicular to bed joints in f_{xk2} direction

where:

$\mu = f_{xd1}/f_{xd2}$;
α_2 = bending moment coefficient from the tables in Annex E of EN 1996-1-1: 2005
W_{ED} = design lateral wind load/unit area.

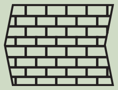
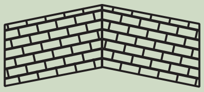
Any vertical load on the wall will enhance the bending resistance parallel to the bed joint to some degree and bed joint reinforcement will do the same for the resistance perpendicular to bed joints. These are included by way of assigning apparent values to flexural strengths:

$$f_{xd1,app} = f_{xd1} + \sigma_d \tag{7}$$

$$f_{xd2,app} = 6 A_s f_{yd} z / t^2 \tag{8}$$

where:

$f_{xd1,app}, f_{xd2,app}$ = apparent flexural strengths in the two orthogonal ratios
σ_d = design compressive stress on the wall, taken to be not greater than $0.2 f_d$
f_{yd} = design strength of bed joint reinforcement

Mortar strength class	M12	M6 and M4	M2	M12	M6 and M4	M2
Clay masonry units of Groups 1 and 2 having a water absorption of:		Value of f_{xk1} 			Value of f_{xk2} 	
<7%	0.7	0.5	0.4	2.0	1.5	1.2
7< and <12%	0.5	0.4	0.35	1.5	1.1	1.0
12% >	0.4	0.3	0.25	1.1	0.9	0.8
Calcium silicate brick sized* masonry units	0.3	0.3	0.2	0.9	0.9	0.6
Aggregate concrete brick sized* masonry units	0.3	0.3	0.2	0.9	0.9	0.6
Aggregate concrete masrony units and manufactured stone of Groups 1 and 2 and AAC masonry units used in walls of thickness up to 100 mm (see Notes 2 and 3) of declared compressive strength:						
2.9	0.25		0.2	0.4		0.4
3.6	0.25		0.2	0.45		0.4
7.3	0.25		0.2	0.6		0.5
Aggregate concrete masrony units and manufactured stone of Groups 1 and 2 and AAC masonry units used in walls of thickness of 250 mm or greater (see Notes 2 and 3) of declared compressive strength:						
2.9	0.15		0.1	0.25		0.2
3.6	0.15		0.1	0.25		0.2
7.3	0.15		0.1	0.35		0.3
Aggregate concrete masrony units and manufactured stone of Groups 1 and 2 and AAC masonry units used in walls of any thickness (see Note 2) of declared compressive strength:						
10.4	0.25		0.2	0.75		0.6
⩾17.5	0.25		0.2	0.9 (see Note 4)		0.7 (see Note 4)

Note 1. Tests to determine the water absorption of clay masonry units are to be conducted in accordance with BS EN 772-7.

Note 2. The thickness should be taken to be the thickness of the wall, for a single-leaf wall, or the thickness of the leaf, for a cavity wall.

Note 3. Linear interpolation may be used to obtain the values of f_{xk1} and f_{xk2} for: (a) wall thicknesses greater than 100 mm and less than 250 mm; (b) compressive strengths between 2.9 N/mm^2 and 7.3 N/mm^2 in a wall of given thickness.

Note 4. When used with flexural strength in the parallel direction, assume the orthogonal ratio $\mu = 0.3$.

* units not exceeding 337.5 mm × 225 mm × 112.5 mm

Note. See Table NA.6 of NA to BS EN 1996-1-1: 2005 for other masonry units.

Table 4 Characteristic flexural strength of masonry, f_{xk1} and f_{xk2}, MPa (reproduced with kind permission from BSI)

A_s = cross-sectional area/m of bed joint reinforcement in tension
t = wall thickness
z = lever arm. [$z = d(1 - 0.5A_s f_{yd}/bdf_d) \leqslant 0.95d$]

Note: the flexural resistance of clay masonry is affected by water absorption of the units which is determined in accordance with BS EN 772-7. This is illustrated in the values given in Table NA.6 of the UK National Annex to BS EN 1996-1-1: 2005.

The characteristic flexural strength of masonry values f_{xk1} (parallel to the bed joint) and f_{xk2} (perpendicular to the bed joint) shall be determined from tests on masonry which may be carried out in accordance with EN 1052-2: 1999 or from available results provided in a database. Values given in the UK National Annex for clay masonry are:

The design moment of resistance, M_{Rd}, per unit height of the wall is given by:

$$M_{Rd} = [(f_{xk1}/\gamma_m) + \sigma_d]Z \qquad (9)$$

where:

γ_m = material partial safety factor (**Table 4**)
σ_d = design vertical load per unit area ($<0.2\, f_k/\gamma_m$)
Z = section modulus of the plan shape of the wall
f_k = characteristic compressive strength of the masonry.

As masonry is accepted as an ideal method of construction to resist compressive forces, its capacity to resist lateral loads therefore depends on its ability to resist flexural tension. As mentioned above, this capacity is enhanced by the presence of a vertical action. The potential flexural strength is also greater if the plane of failure is perpendicular rather than parallel to the bed joint.

The flexural strength approach makes use of yield-line analysis to develop the bending moment coefficients developed above. Where wall panels contain openings (doors or windows, etc.) a degree of care is required and the panel should typically be subdivided into smaller panels (Roberts and Brooker, 2007).

Deformation properties of masonry

Modulus of elasticity

The modulus of elasticity of masonry shall be determined in accordance with EN 1052-1: 1999 or for practical purposes and in accordance with the UK National Annex:

$$E = K_E f_k \tag{10}$$

where E is the short-term secant modulus of elasticity of masonry; K_E is 1000 as given in the UK National Annex.

For calculations relating to serviceability limit state, the long-term value of E should be used.

$$E_{\text{long term}} = E/(1 + \varphi_\infty) \tag{11}$$

where φ_∞ is the final creep coefficient.

(Note: see Chapter 37 for a more in-depth study of the deformation properties of masonry.)

Creep, moisture expansion or shrinkage and thermal expansion

At present there is no European test method for the determination of creep or moisture expansion of masonry. However, values obtained from project-specific tests or a database may be used.

A range of values is given in EN 1996-1-1: 2005 but the UK National Annex values are shown in **Table 5**.

Type of masonry unit	Final creep coefficient$^{(A)}$: φ_∞	Long-term moisture expansion or shrinkage$^{(B)}$: mm/m	Coefficient of thermal expansion, α_t: 10^{-6}/K
Clay	1.5	0.5	6
Calcium silicate	1.5	−0.2	10
Dense aggregate concrete and manufactured stone	1.5	−0.2	10
Lightweight aggregate concrete	1.5	−0.4	10
Autoclaved aerated concrete	1.5	−0.2	10
Natural stone	Normally very low	0.1	10

(A) The final creep coefficient $\varphi_\infty = \varepsilon_{c\infty}/\varepsilon_{el}$, where $\varepsilon_{c\infty}$ is the final creep strain and $\varepsilon_{el} = \sigma/E$.

(B) Where the long-term value of moisture expansion or shrinkage is shown as a negative number it indicates shortening and as a positive number it indicates expansion.

Table 5 UK National Annex values (reproduced with kind permission from BSI)

Structural analysis of masonry

Introduction

Before carrying out structural analysis, the following information should be provided:

- A clear description of the structure, materials of construction and exposure conditions.
- Behaviour of the whole or part of the structure for different limit states.
- Actions on the structure or parts of it.

Adequate stability and robustness should be provided by careful consideration of layout and connection details.

Although EN 1996-1-1: 2005 recognises calculations based on a non-linear relationship between stresses and strains; a linear analysis is also acceptable and is more common.

Imperfections are allowed for by assuming that the structure has an inclination to the vertical, e.g. out of plumb. The value of the inclination angle to consider depends on the total height of the structure and is given as $\vartheta = 1/100\sqrt{h_{tot}}$ radians and the resulting horizontal action from this should be added to the other actions.

The structural layout should normally be such that adequate bracing against sway is provided.

Walls subject to vertical loading

The load-bearing capacity of vertically loaded walls is affected by the degree of fixity at its top and bottom, and vertical edges and the eccentricity of the applied loads. A number of 'end' conditions are provided in EN 1996-1-1: 2005.

Effective height

The relationship between the effective height and the degree of restraint is given as:

$$h_{ef} = \rho_n h \tag{12}$$

where h is the clear storey height of wall and ρ_n is a reduction factor appropriate to the degree of restraint: n = 2, 3 or 4.

$\rho_2 = 0.75$ for full top and bottom restraint offered by concrete floors and $\rho_2 = 1.0$ if the eccentricity of the load at the top of the wall is greater than 0.25 t.

$\rho_2 = 1.0$ where the restraint is offered by timber floors or roofs.

$\rho_3 = (1.5l/h) \geqslant 0.3$ for top and bottom restraint and one vertical edge stiffened, $h \geqslant 0.3$.

$\rho_4 = (1/(1 + (\rho_2 h/l)^2) \times \rho_2)$ for top and bottom restraint and both vertical edges stiffened, $h \leqslant 1.15l$.

$\rho_4 = 0.5l/h$ for top and bottom restraint and both vertical edge stiffened, $h > 1.15l$.

ρ_3 and ρ_4 values are shown graphically in Annex D of EN 1996-1-1: 2005.

Ratio of pier spacing (centre to centre) to pier width	Ratio of pier thickness to actual thickness of wall to which it is bonded		
	1	2	3
6	1.0	1.4	2.0
10	1.0	1.2	1.4
20	1.0	1.0	1.0

Note: Linear interpolation between the values give in Table 6 is permissible.

Table 6 Ratio of pier spacing (reproduced with kind permission from BSI)

Effective thickness

The effective thickness, t_{ef}, of a single-leaf wall, a double-leaf wall, a faced wall, a shell bedded wall and a grouted cavity wall should be taken as the actual thickness of the wall, t.

The effective thickness of a cavity wall is calculated from the equation below:

$$t_{ef} = \sqrt[3]{k_{tef} t_1^3 + t_2^3} \tag{13}$$

where t_1 and t_2 are the actual thickness of the leaves and $k_{tef} = E_1/E_2$, the E values of each leaf respectively. However, the UK National Annex recommends $k_{tef} = 1$.

The effective thickness of walls stiffened by piers is given by:

$$t_{ef} = \rho_t \times t \tag{14}$$

where ρ_t is given in **Table 6**.

Ultimate limit state design for vertical loads

The requirement is that the design vertical load on the wall, $N_{ED,}$ should be less than or equal to the design value of the vertical load resistance of the wall, N_{RD}. The design vertical load resistance of a single-leaf wall per unit length is given by:

$$N_{RD} = \Phi t f_d \tag{15}$$

Where:

Φ is the capacity reduction factor
f_d = design compressive strength of masonry = f_k/γ_m.

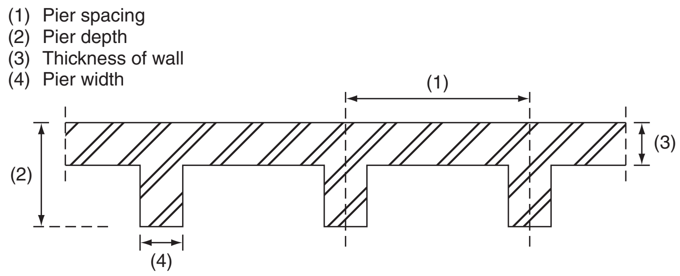


Figure 1 Pier spacings (reproduced with kind permission from BSI)

For wall cross-sections of less than 0.1 m^2, f_d should be multiplied by $(0.7 + 3A)$, the so-called 'small wall correction factor'. 'A' is the loaded horizontal gross cross-sectional area of the wall in m^2.

In cavity walls, each leaf should be verified separately but the slenderness ratio should be based on the effective thickness. There should be no less than 2.5 ties per unit area of the wall.

Faced walls should be designed as a single-leaf wall constructed entirely of the weaker units.

Double-leaf walls should be designed with the leaves tied together such that the number of ties is in accordance with:

$$n_t \geqslant W_{ED}/F_d \tag{16}$$

where:

n_t = the minimum number of ties, taken as 2.5 per m^2
W_{ED} = design value of the horizontal load per unit area of wall to be transferred
F_d = minimum (compressive or tensile) design resistance of one wall tie.

In selecting wall ties, consideration should be given to their appropriateness for allowing differential movement between the leaves without causing damage.

Chases and recesses should be limited to the values given in the UK National Annex as shown in **Table 7** for vertical chase.

- The maximum depth should include the depth of any hole reached during forming.
- Vertical chases should not be longer than 1/3 of the storey height above floor level.
- For walls 225 mm thick or greater, chases may be up to 80 mm deep and 120 mm wide.
- Distance between horizontal chases or a chase and a recess or opening should not be less than 225 mm.

Thickness of single leaf or loaded leaf of a cavity wall: mm	Size of chase formed after construction of masonry		Size of chase formed after construction of masonry	
	$t_{ch,v}$: mm	Maximum width: mm	Remaining wall thickness should be not less than: mm	Maximum width: mm
75–89	30	75	60	300
90–115	30	100	70	300
116–175	30	125	90	300
176–225	30	150	140	300
226–300	30	175	175	300
>300	30	200	215	300

Table 7 Vertical chases and recesses (reproduced with kind permission from BSI)

Thickness of single leaf or loaded leaf of a cavity wall: mm	$t_{ch,h}$ unlimited length: mm	Length $\leqslant$ 1250 mm
75–84	0	0
85–115	0	0
116–175	0	15
176–225	10	20
226–300	15	25
>300	20	30

Table 8 Horizontal or inclined chases (reproduced with kind permission from BSI)

- Distance between any two adjacent recesses, whether or not on the same side, or between a recess and an opening should not be less than twice the width of the wider recess.
- Sum of the widths of vertical chases and recesses should not be greater than 0.13 times the wall length for horizontal chases.

For horizontal chase

- Maximum depth should include the depth of any hole reached during forming.
- Horizontal distance between the end of a chase and an opening should not be greater than 500 mm.
- Horizontal distance between adjacent chases, whether or not on the same side, should not be less than twice the length of the longest chase.
- In walls of thickness greater than 175 mm the permitted depth of chase may be increased by 10 mm if the chase is machine cut accurately to the required depth. If machine cut chases of up to 10 mm deep may be cut on both sides of wall not less than 225 mm.
- Width of chase should not exceed half the residual thickness of the wall.

Calculation of capacity reduction factor, Φ

For the top or the bottom of the wall:

$$\Phi_i = 1 - 2e_i/t \qquad (17)$$

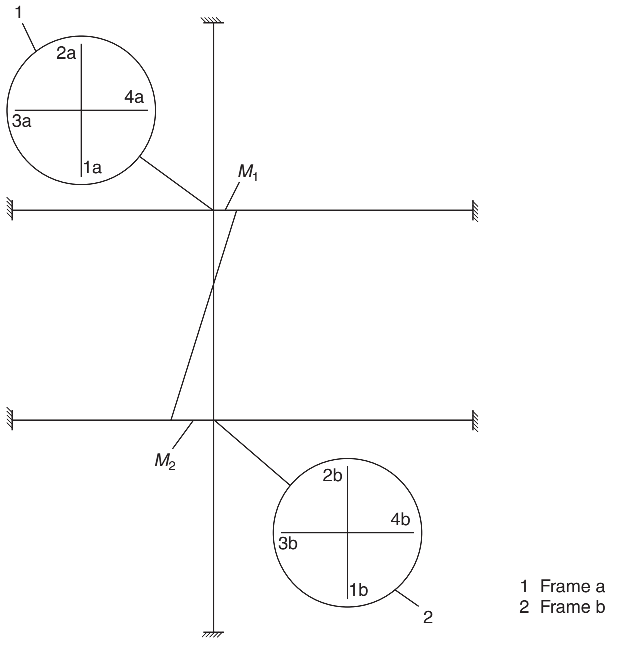
where e_i is the eccentricity at the top or bottom of the wall calculated from

$$e_i = M_{id}/N_{id} + e_{he} + e_{init} \geqslant 0.05t \qquad (18)$$

where:

M_{id} = design moment at the top or bottom of the wall due to load eccentricity
N_{id} = design vertical load at the top or bottom of the wall
e_{he} = eccentricity at the top or bottom of the wall from horizontal loads
e_{init} = initial eccentricity taken as $h_{ef}/450$.

A sub-frame analysis is suggested in EN 1996-1-1: 2005 and the model is shown in **Figure 2**.



Moment M_1 is found from frame a and moment M_2 from frame b

Figure 2 Sub-frame analysis (from BS EN 1996-1-1: 2005) (reproduced with kind permission from BSI)

The suggested equation in Annex C of EN 1996-1-1: 2005 is:

$$M_1 = \frac{\dfrac{n_1 E_1 I_1}{h_1}}{\dfrac{n_1 E_1 I_1}{h_1} + \dfrac{n_2 E_2 I_2}{h_2} + \dfrac{n_3 E_3 I_3}{l_3} + \dfrac{n_4 E_4 I_4}{l_4}} \times \left[\frac{w_3 l_3^2}{4(n_3 - 1)} - \frac{w_4 l_4^2}{4(n_4 - 1)} \right]$$

where:

n_i = the stiffness factor for members taken as 4 for members fixed at both ends and otherwise 3
E_i = the modulus of elasticity of member i, where i = 1, 2, 3 or 4.

Note: It will normally be sufficient to take the values of E as $1000f_k$ for all masonry units.

I_i = the second moment of area of member j, where j = 1, 2, 3 or 4 (in the case of a cavity wall in which only one leaf is loadbearing. I_i should be taken as that of the loadbearing leaf only)
h_1 = the clear height of member 1
h_2 = the clear height of member 2
l_3 = the clear span of member 3
l_4 = the clear span of member 4
w_3 = the design uniformly distributed load on member 3, using the partial factors from EN 1990, unfavourable effect
w_4 = the design uniformly distributed load on member 4.

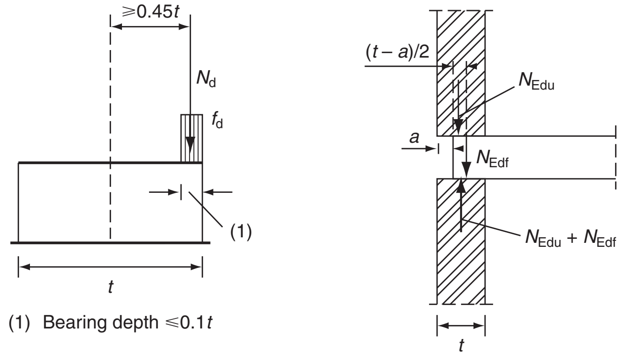


Figure 3 Partial bearing model (reproduced with kind permission from BSI)

Note: this method is not considered suitable for timber floor joists.

It is likely that the results of such calculations are conservative and it is suggested in the Annex that the following reduction factor can be applied to eccentricities:

$$\eta = (1 - k_m/4) \tag{19}$$

where:

$$k_m = \frac{n_3\dfrac{E_3I_3}{l_3} + n_4\dfrac{E_4I_4}{l_4}}{n_1\dfrac{E_1I_1}{h_1} + n_2\dfrac{E_2I_2}{h_2}} \leqslant 2$$

If the calculated eccentricity is greater than 0.45t then a partial bearing model, shown in **Figure 3**, should be adopted.

Moment above the floor, $M_{Edu} = N_{Edu}(t - 3a)/4$ and the moment below the floor, $M_{Edf} = N_{Edf}(a/2) + N_{Edu}(t + \mathrm{a})/4$ where a is the distance from the face of the wall to the edge of the floor, N_{Edu} and N_{Edf} are the design loads in the upper and lower walls respectively.

For the middle of the wall, Φ_m can be calculated from the EN 1996-1-1: 2005 equations in Annex G or from the graphs given in the same Annex and reproduced in **Figure 4** for $E = 1000f_k$.

The middle of the wall eccentricity has a term relating to creep eccentricity. However, for slenderness ratios of 27 or less, creep eccentricity may be taken to be zero.

Therefore:

$$e_m = M_{md}/N_{md} + e_{hm} \pm e_{init} \tag{20}$$

where:

$$e_m = \frac{M_{md}}{N_{md}} + e_{hm} \pm e_{init}$$

e_m = the eccentricity due to loads
M_{md} = the design value of the greatest moment at the middle of the height of the wall resulting from the moments at the top and bottom of the wall including any load applied eccentrically to the face of the wall (e.g. brackets)
N_{md} = the design value of the vertical load at the middle height of the wall including any load applied eccentrically to the face of the wall (e.g. brackets)
e_{hm} = the eccentricity at mid-height resulting from horizontal loads (for example, wind).

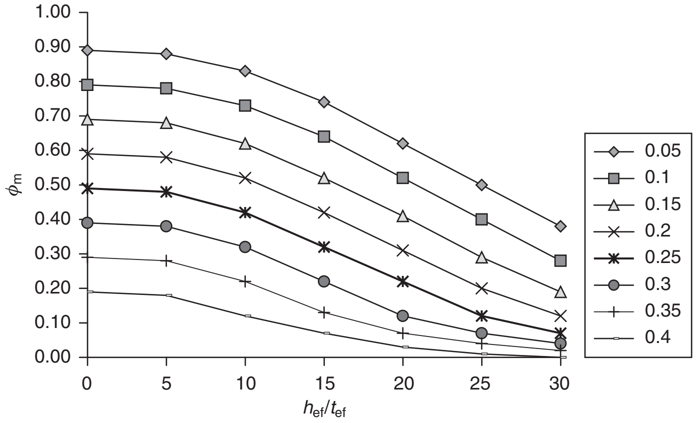


Figure 4 Capacity reduction factors

Note: The inclusion of e_{hm} depends on the load combination being used for the verification; its sign relative to that of M_{md}/N_{md} should be taken into account.

e_{init} = the initial eccentricity.

Concentrated loads

There are different 'enhancements' associated with different masonry groups. For Group 1 masonry units, other than shell bedded walls, vertical load resistance is:

$$N_{Rdc} = \beta A_b f_d \tag{21}$$

where $\beta = (1 + 0.3a_1/h_c)(1.5 - 1.1A_b/A_{ef})$ which should not be less than 1.0 nor greater than the lesser of $1.25 + a_1/2h_c$ or 1.5.

Where:

β = an enhancement factor for concentration loads
a_1 = the distance from the end of the wall to the nearer edge of the loaded area
h_c = the height of the wall to the level of the load
A_b = the loaded area
A_{ef} = the effective area of bearing, i.e. $l_{efm} \times t$
l_{efm} = the effective length of the bearing as determined at the mid height of the wall or pier
t = the thickness of the wall, taking into account the depth of recesses in joints greater than 5 mm
A_b/A_{ef} = not to be taken greater than 0.45.

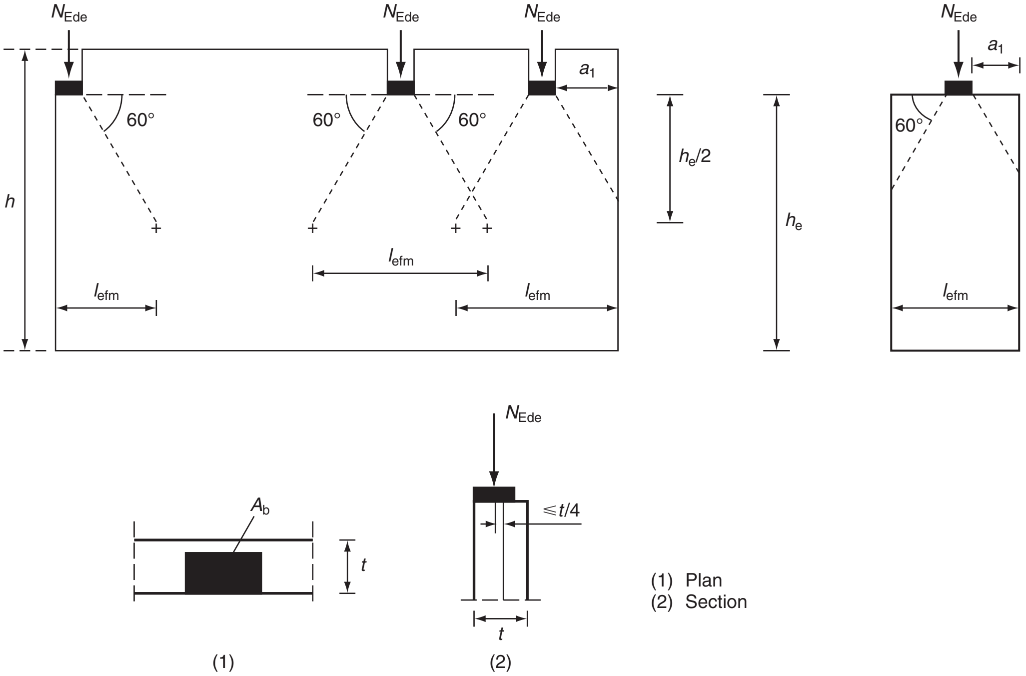


Figure 5 Walls subjected to concentrated load (reproduced with kind permission from BSI)

For walls constructed with Groups 2, 3 and 4 masonry units with shell bedding, $\beta = 1.0$. The concentrated load should bear on a Group 1 unit or other solid material of length equal to the required bearing length plus a length on each side, except for end bearings where the additional length is considered on one side only of the bearing based on a 60° spread of load to the base of the solid material.

If a spreader of adequate stiffness is used it must be the same width as the thickness of the wall and have a height greater than three times the bearing length of the load. The design compressive strength below the concentrated load should not exceed $1.5f_d$.

The design vertical load capacity of the wall must be checked at mid-height against the applied loads and spread of concentrated loads on a 60° basis.

Lateral loads

The following should be taken into consideration when designing masonry walls subjected to lateral loading:

- Effect of damp proof course.
- Support condition and continuity over supports.
- Faced walls should be analysed as single-leaf walls constructed in units with the lower flexural resistance.
- Movement joints should be considered as free edges unless provisions for shear transfer are made.
- Support reactions may be considered as uniformly distributed along the edges.
- Continuity can be assumed in cavity walls with only one continuous leaf provided the leaves are tied together and the ties can be shown to transfer the lateral loads from one leaf to the other. Openings and vertical edges require special consideration.
- Panel size should be limited to the following provided that the minimum thickness of the wall or leaf of a cavity wall is 100 mm. For cavity walls t_{ef} replaces t in **Figures 6–8**.

Masonry panels can be designed as one- or two-way spanning panels depending on the degree of fixity along their side and their aspect ratio, i.e. h/l.

The moment of resistance of the wall is given by:

$$M_{RD} = f_{xd}Z \tag{22}$$

where:

f_{xd} = design flexural strength appropriate to the plane of bending
Z = elastic section modulus of height or length of wall depending on the mode of resistance.

The presence of a vertical load has a favourable effect and can be included by modifying f_{xd} to an apparent value given by:

$$f_{xd1,app} = f_{xd} + \sigma_d \tag{23}$$

where σ_d is the design compressive stress on the wall $\leqslant 0.2f_d$.

For calculating the section modulus of a wall with piers, the outstanding flange from the face of pier is the lesser of:

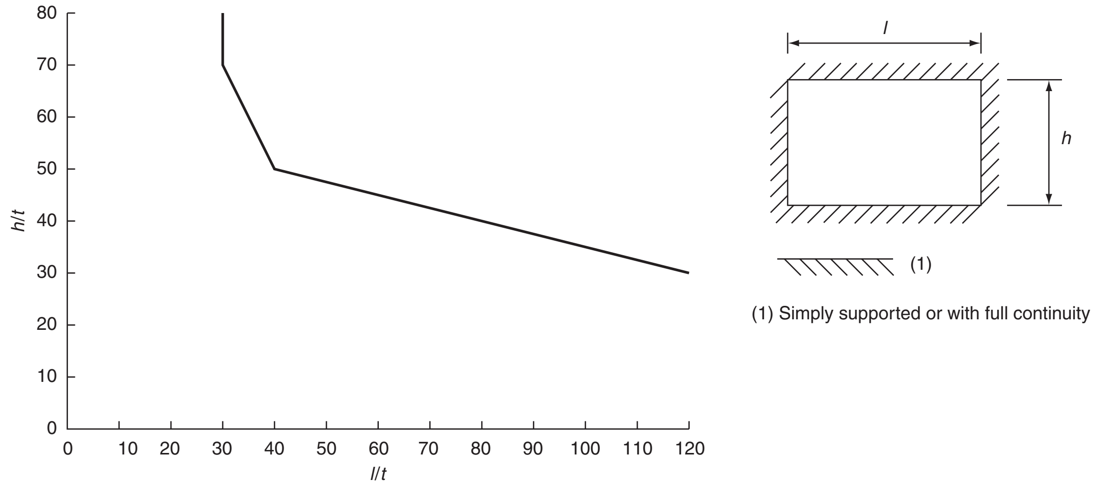


Figure 6 Limiting height and length to thickness ratios of walls restrained on all four edges (reproduced with kind permission from BSI)

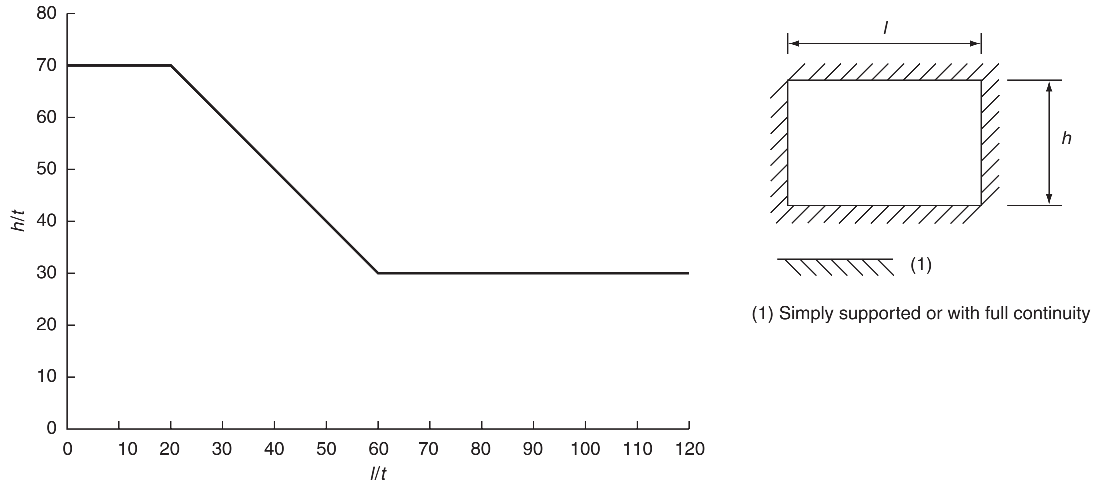


Figure 7 Limiting height and length to thickness ratios of walls restrained at the bottom, top and one vertical edge (reproduced with kind permission from BSI)

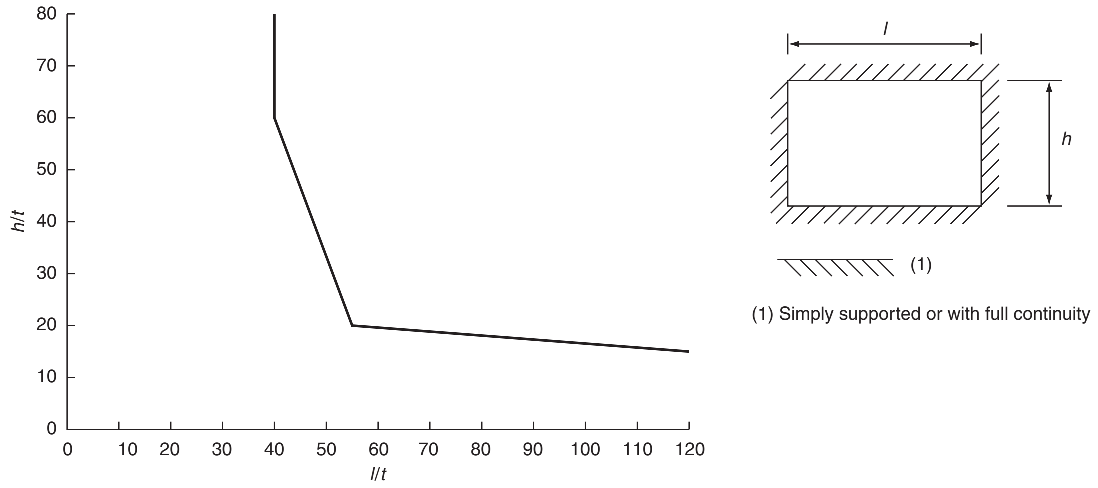


Figure 8 Limiting height and length to thickness ratios of walls restrained at the edges, the bottom but not the top (reproduced with kind permission from BSI)

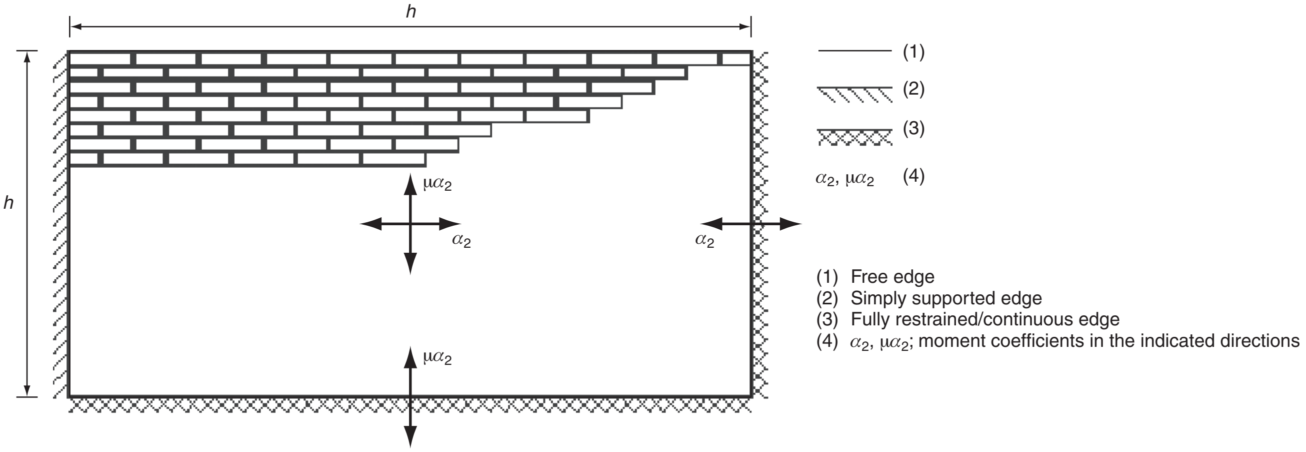


Figure 9 Bending moment coefficient, α_1, in single-leaf laterally loaded wall panels of thickness $\leqslant$250 mm (reproduced with kind permission from BSI)

- $h/10$ for vertically spanning walls
- $h/5$ for cantilever walls, e.g. free-standing walls
- half the clear distance between piers

where h is the clear height of the wall.

For a two-way spanning wall the orthogonal ratio μ is taken as the ratio of the design flexural strength parallel to the bed joint to the design flexural strength perpendicular to bedding, $f_{\text{xd1}}/f_{\text{xd2}}$. If there is a vertical load on the wall, f_{xd1} is modified to $f_{\text{xd1,app}}$ as shown before.

The moment of resistance is calculated from:

$$M_{\text{ED1}} = \alpha_2 W_{\text{ED}} l^2 \text{ per unit height of the wall} \qquad (24)$$

$$M_{\text{ED2}} = \mu\alpha_2 W_{\text{ED}} l^2 \text{ per unit height of the wall} \qquad (25)$$

For cavity walls, the applied design lateral load may be apportioned between the two leaves in accordance with their relative strengths or stiffness provided that the two leaves are properly tied together as described previously.

If the wall possesses chases or recesses, the weakening effect of these should be considered in the verification.

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Further reading

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CONTENTS

Chapter 34

Developments in masonry

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Masonry has developed continually over many years and in recent years changes in regulations have led to changes which have become part of modern practice. In 1997 the Masonry Industry Alliance developed a strategy for innovation and research for the subsequent 10 years. In 2007 the progress against that strategy was reviewed. Since that date, some further developments have taken place perhaps most significantly those in relation to the revisions to British Standard Codes of Practice and the Eurocode which will shortly replace the British Code. Some developments have stayed at the research phase, for example the design of pocket-type retaining walls with wide pocket spacings; some have been overtaken such as Traditional Plus; and some have continually evolved such as the development of ties. The development of the WI-beam system is a classic solution to a construction problem which requires a purpose drafted design guide. Masonry has developed not only in terms of new build but also the sympathetic treatment and change of use of existing buildings has transformed the centres of many of our cities. Masonry has moved forward in a variety of ways: through research-based projects, practical solutions, materials use and development and understanding of existing masonry. All are relevant and are underpinned by the development of Codes and Best Practice Guides.

Introduction

The title ‘Developments in masonry’ can be interpreted in a wide number of ways. Masonry has been developing since Roman times and before, and much has been written about the history. This interpretation will focus on more recent developments, The twentieth century saw some interesting developments: a general move from solid walls to cavity walls; a more widespread use of autoclaved aerated concrete blocks; an increased use of perforated bricks and a drift upwards in the level of perforation; and an improved quality in concrete masonry units as we moved away, if not in name, from the use of breeze blocks. In 1997 the Masonry Industry Alliance published *Masonry 2007*, an innovation and research strategy to take the industry into the twenty-first century. In 2007 I took a backward look at what had taken place in the decade and mapped what had happened against the priority areas identified in 1997 (Edgell, 2007). In this look at developments I will start with the outcomes of that review and add or amend comments and discussion points in the light of more recent thoughts and developments.

The needs that were identified by the Masonry Industry Alliance were grouped under the following headings:

- craft skills base
- education and training
- codes and standards
- sustainable construction
- innovative masonry systems
- improved masonry process
- technology of material interfaces.

The Masonry Industry Alliance deliberately took a very broad view in establishing what was required under the heading *innovation*, including, for example, education and training initiatives. Some observations have been made regarding progress in each area.

Craft skills base

- To investigate the extent of any projected shortfalls in the availability of skilled craftsmen to install and fix masonry products.
- To develop practices and means to expand the availability and quality of skilled craftsmen to meet the needs of industry’s customers.

It is probably fair to say that throughout the period more has been done to expand the availability of skilled craftsmen than to measure shortfalls; this is no doubt fuelled by the anecdotes of absurdly high wages from time to time in areas of shortages. The Better Brickwork Alliance, a group of manufacturers, contractors, trainers and Guilds drawn together by the Brick Development Association (BDA), has tried to encourage young people into the craft. The Alliance, either working together or as individual organisations, has provided resources to all ages from schoolchildren on the National Curriculum, through students at colleges twinned with material suppliers to competitions for both students and apprentices. The Modern Masonry Alliance now claims that there are sufficient trained craftsmen to lay all of the masonry units being produced.

Education and training

- To investigate the extent and content and projected education and training of construction professionals for the design of masonry and to identify any additional training and education needed to encourage the efficient use of masonry.
- To develop processes and practices to improve the education and training of undergraduates to meet the industry needs.
- To provide support, possibly through information technology, for the design and specification of masonry construction.

It seems to be general knowledge that the training of construction professionals in the design of masonry falls far behind that in any of the other main construction materials. The content of engineering courses is extremely low, that in schools of architecture a little better. Consequently, the focus has been on provision. The trade associations in particular have provided support in their particular areas of interest, for example the BDA to Schools of Architecture and the Aircrete industry in areas such as Thermal Insulation Regulations as continual professional development (CPD) activity. The trade associations also support the professional institutions in their efforts to provide lectures or other educational training material. However, the overall volume is relatively small, probably because the allocation by the institutions represents the already perceived importance in relation to other construction materials. In this way the status quo gets perpetuated. There have been a limited number of universities that have taken an interest, notably Kingston for consistency and various others usually related closely to the duration of a research grant. There are some sponsored academics and that is to be welcomed, but the overall effort is still below par.

The BDA has taken an initiative in the area of IT which should aid specification: it has software which will produce dimensioned drawings of structures designed in multiples of brick dimensions.

Codes and standards

- To develop and update data on the technical performance of masonry in support of regulatory and non-regulatory issues.
- To provide guidance on the use of European Standards.
- To maintain the existing UK codes (e.g. BS 5628: 2005) to provide a satisfactory transition arrangement within the Eurocodes.
- To support and enhance the Application Rules of Eurocode 6, where necessary.

Here the industry has done well in regard to its technical input to code and standard development. This has largely come from the manufacturing industry, with some notable exceptions. For manufacturers this is vital, as an odd class boundary, performance requirement or design rule can open up or close down an element of the market. Hence, self-interest has kept this fuelled. In terms of guidance on the use of European Standards, manufacturing industry has disseminated information among its own staff, the British Masonry Society has published relevant papers and run Technical Sessions but there was, in 2007, no great appetite for knowledge among the specifier community. In a sense the same is true of the Eurocodes but there was little interest from the design community.

In the author's view, no look at developments in masonry should miss a look at the technical developments in codes and standards, and I will spend a little time on them.

Both are important to the development of the masonry industry. The main Codes of Practice which are of relevance are:

BS 5628: 2005 Code of Practice for Use of Masonry
  Part 1 Structural Use of Unreinforced Masonry
  Part 2 Structural Use of Reinforced and Prestressed Masonry
  Part 3 Materials and Components, Design and Workmanship

All of which were revised and republished in 2005. The main aim of the revision being to refer to the suite of European Standards for Masonry units, mortars and ancillary components.

The European Code of Practice was published in 2005 and 2006, and it is intended that this replace BS 5628: 2005 in its entirety in 2010. The series is:

BS EN 1996-1-1: 2005 General Rules for Reinforced and Unreinforced Masonry Structures
BS EN 1996-1-2: 2005 General Rules – Structural Fire Design
BS EN 1996-2: 2006 Design Considerations, Selection of Materials and Execution of Masonry
BS EN 1996-3: 2005 Simplified Calculation Methods for Unreinforced Masonry Structures

In 2009, design professionals are beginning to become familiar with the Codes, a number of guidance documents (handbooks) have been produced or are in the process of publication and the International Masonry Society and the Institution of Civil Engineers have run successful seminars. The key interest at the moment is, of course, the differences in any design outcomes between the Eurocode approach and the existing BS approach to see whether supplementary considerations are needed to retain or improve the economy of masonry.

One can of course agree as to whether practice drives Code development or the other way around. In reality both occur. For example, when it became necessary to reduce the thickness of mortar joints in autoclaved aerated concrete blockwork, as when included in thermal insulation calculations, they proved to be something of a cold bridge; it became essential to develop the technique and then make sure it was covered in Codes. This gave practising designers the confidence to use them and insurers the confidence to enable them to. On the other hand when in the 1980s, especially following the publication of BS 5628 Part 2: 2005, the clay brick industry developed designs for pocket-type retaining walls that began to be cost competitive, with reinforced concrete walls at significant heights, sometimes practice drove the Code development and sometimes the other way around. The Code encompassed the design concepts but industry needed to have accepted the improved designs, especially for shear, that drove the economics. It is a great shame that such productive interactions are relatively rare. Pocket-type walls 3 m high typically have pocket spacing of 1.2–1.4 m; however research, confirmed by testing, has shown that pocket spacings could safely be as much as 3 m with all the added economy (Tellet and Edgell, 1983). The appropriate guidance has not yet been developed. One example where guidance has led the way and opened up opportunities is in the area of wall ties. The UK led the development of the first performance-based standard for wall ties and introduced a classification system that helps the designer select which ties may be used in which situation. Since the introduction of the system in the early 1980s, coupled with the move towards the use of stainless steel, the tie industry has continually developed. The aim has been, especially in times of soaring prices for stainless steel, to produce ties made from the lightest gauge of steel but with sufficiently clever structural features to meet performance requirements. The same type of logic has extended to the straps we use to tie walls to floors and roofs. For many years the traditional 30 mm × 5 mm steel strap was referred to in our Approved Document to the Building Regulations but that became ‘challenged’ by the introduction of lighter straps which incorporated similar safety. Now, with the introduction of harmonised European Standards, we have a fully performance-based approach.

Code guidance regarding the design of natural stone masonry has, until recently, been relatively limited and for good reason. Most buildings made from natural stone have been relatively massive and hence stresses relatively low. Consequently, the guidance that it could be treated as concrete blockwork was quite reasonable. However, when the New Parliamentary Building, Portcullis House, was designed, the engineers realised that the stresses would be relatively high and more relevant data and guidance were needed. As a result, an extensive programme of testing was carried out and the design was therefore better informed. Fortunately, as the work was publicly funded, the results could be made available for wider use and BS 5628: 2005 could be revised to give greater relevant guidance for more highly stressed stonework.

The 2005 editions of the UK’s Code of Practice, BS 5628: 2005 takes us so far forward, for example it incorporates data for thin layer mortar, concrete blocks laid flat, and performance-based ties. However, some of what we need is not in BS EN 1996-1-1: 2006, Eurocode 6 or its National Application Document, which gives values of those safety-related items, e.g. partial factors, that are relevant for the UK. The material will of course be published separately in a non-contradictory and complementary way. However, it cannot be too long before someone wants to use an approach which may be covered somewhat sketchily in Europe and UK guidance and with which we have no experience in the UK. Perhaps we will see a bid to use, for example, confined masonry: panels built within framing elements possibly in reinforced masonry using national guidance from Portugal. Who knows, we are already seeing a version of it to resist wind load in the WI beam approach (see under systems), perhaps it has a role in resisting shear in areas relatively free of seismic effects.

There is no doubt that the introduction of Eurocode 6 will be profound. There are others who have a lot to learn from our ‘additional’ material. The Eurocode is weak on design of prestressed masonry but the UK has good complimentary material. We also have a long history of use of prestressed brickwork: tall slender brickwork columns were used in a London school by F. J. Samuelly in the 1950s, and Donald Foster designed the slenderest piece of post-tensioned brickwork in the world for the Stoke-on-Trent Garden Festival in 1986. We have an even longer history in reinforced brickwork, going back to Sir Mark Brunel. It seems that some of our mainland colleagues may utilise this information: after all, masonry design in Holland is now being taught in the English language. The level of ingenuity and consequent development could just increase.

Sustainable construction

- To prepare the necessary information regarding the whole-life benefits of masonry – such as whole-life costs, recycling, energy use, maintenance, and waste – and to cooperate with other researchers.
- To ensure that the correct information regarding the sustainable use of masonry is included in research, and to publicise and propagate the results of this work.

Sustainability is certainly at the heart of government thinking at the moment and there have been a lot of developments in this area, not least the Code for Sustainable Homes.

In relation to the needs identified in *Masonry 2007*, the industry has worked with the Building Research

Establishment to try to ensure that correct information is used in the development of Ecohomes. Issues that are under manufacturers' control are easily dealt with and, for example, verified data exist for energy use in the manufacture of clay bricks and, from their cement contents, concrete blocks as the climate change levy requires regular submission of data to the Department of Environment, Food and Rural Affairs (DEFRA). However, whole-life costs are not as simple. The British Masonry Society Annual General Meeting in 2006 included studies of 860 buildings carried out by Leeds Metropolitan University and showed that the average period for replacement of clay bricks was over 150 years. We also now know from the Arup study into lightweight vs heavyweight construction that, due to climate change, lightweight structures will enable unacceptably high internal temperatures to be reached as early as 2020. In heavyweight construction such a situation might arise some 50 years later. Clearly these facts need to be absorbed into building rating schemes and hopefully the Europe-wide approach now being developed will take due account of these facts.

A good example of the promulgation of sustainability information is the BDA response to the government's Sustainable Development Strategy. 'A Sustainable Strategy for the Brick Industry' sets out the case well and through the inclusion of key performance indicators, which are regularly reported, has the means to track industry performance against published objectives.

The indicators are:

- occupational health and safety of industry employees
- employee development through relevant and useful vocational training
- working with the supply chain and developing the Egan agenda
- liaison and support for local communities
- extending environmental management systems
- reducing the impact of emissions and process
- minimising waste disposal to landfill
- improved quality of effluent discharged
- increased contribution to ecological site management
- reduced use of treated water
- improved contribution to ecological site management
- reduced use of treated water
- improved energy efficiency
- increased efficiency of transport
- greater use of secondary materials (MARSS – materials from alternative, recycled and secondary sources)
- profitability, investment, employment
- manufacturing efficiency and environmental performance.

Since 2007 the momentum to improve sustainability has increased. European Codes are being developed which create rating systems against which buildings can be scored. It is however not that simple, as the amount of data needed to make such a scheme reasonably believable is quite significant and goes far beyond what is needed to successfully market products. Information 'before the factory gate' is relatively easy to obtain: that outside is very difficult and in a very fragmented industry is difficult to obtain reliably.

Another effect of the focus on sustainability is an increase in interest in a number of novel products. Lime mortar is of course not novel, but it has been dormant in the UK for a number of years and now is re-emerging as a product of interest. One of the arguments in favour of it is the easier cleaning of mortar from units and hence the greater reusability of units. The validity of this argument is difficult to verify, as certainly a fully carbonated lime-based mortar can provide excellent adhesion to masonry units. Reduced energy in production is also a more difficult argument as the use of waste materials such a chopped up car tyres to fuel cement kilns does distort the argument. What is not arguable is that lime-based mortars take longer to set, possibly reaching close to full strength after 56 or even 90 days. This can have implications for the construction sequence. However, the 'softer' nature of the mortar (more elasticity) is claimed to improve tolerance to movements before any cracking occurs.

Lime is also used in other novel if relatively rarely used forms of construction such as lime renders to straw bale construction, lime binders to fibrous materials, e.g. hemp in blocks.

The drive for sustainability has also stimulated interest in the use of unfired clay blocks, sometimes stabilised with lime, especially for internal partitions where it is claimed that the passive control of temperature is enhanced by a moderating influence on humidity variation. Several experimental products are in development, e.g. blocks made with bituminous binders to try to improve the general sustainability of masonry products.

Improved masonry systems

- To produce innovative systems of masonry construction to improve its performance, quality and economy, and to publicise and propagate the results of this work.

There has in recent years been a lot of pressure from government to use modern methods of construction (MMC) which have generally been interpreted as involving greater levels of offsite prefabrication. One could well feel that little has happened by way of innovative systems in what is a fairly traditional masonry industry. However, that is not the case. The introduction of thin layer mortars in aircrete has helped meet new thermal insulation

Figure 1 Thin-joint aircrete

Figure 2 Traditional plus novel hangers

requirements and also provides enhanced structural performance. Research is also in hand on the use of lower-density aircrete. The ancillary component industry has played its part as the thin joint system (**Figure 1**) needed new designs of ties, bed joint reinforcement and anchors to suit the new joints. Another development is the introduction of larger units made from aircrete or calcium silicate which together with thin-layer mortars offer the possibility of increased production rates.

Offsite prefabricated construction came in a variety of guises and there is a role for brickwork of a relatively traditional appearance but which is constructed offsite in what is analogous to a concrete casting yard. It can be particularly effective, as in the Inland Revenue Building in Nottingham where there is a lot of repetition of certain structures, e.g. walls or columns, and especially on sites where access is restricted. Elements can be built in reasonably protected conditions on something of a production line basis and either reinforced or either temporarily or permanently prestressed before transportation to site and hoisting into place.

Larger format clay units are also being made where together with bed joint reinforcement the use of stack bonded construction offers architects a new aesthetic in a way that can be engineer designed. Larger and wider clay units have also been made for the new housing system Traditional Plus, whereby well-insulated walls are produced from single-leaf construction on pile and ground beam foundations with timber floors supported on novel joist hangers (see **Figure 2**). This form is, however, unlikely to succeed commercially as the greater interest in sustainability drives an increased use of internal thermal mass which this system currently does not offer.

In aggregate concrete, blockwork retaining walls can now be built from hollow blocks which are filled with flowing concrete that cascades across the internal webs so as to fully fill all of the voids. There has also been an explosion in the development of lightweight manufactured stone and its use as quoins, cills, mullions, etc. makes the detailing of modern masonry interesting and cost-effective.

One of the more interesting developments is that of the WI beam. This is an attempt to break up the spans of large unreinforced blockwork walls by the introduction of reinforced beams. The focus has been on 140 mm blockwork walls where a typical wall for testing purposes is 8 m wide and 5 m high. The walls are tied into frames at either side. At approximately one-third and two-thirds of the height, a bond beam is introduced. This is a course of trough-shaped units; reinforcing bars are introduced and fitted into cleats at the columns (see **Figure 3**). The course is concreted and vertical shear connectors ensure that the course above and below act compositely with the beam (see **Figure 4**). The effect is that a large wall can be broken down into three which span vertically over relatively low heights (see **Figure 5**). The walls resist very high lateral loads and avoid the use of wind posts often considered to cause lots of practical problems. The bond (WI) beam is well researched and its use is being developed further with walls containing openings where they can provide lintels and cills; however, because of the transition of existing code guidance from BS to the Eurocode, the opportunity to cover the system in these official documents does not currently exist. Consequently a purpose produced Design Guide will be available in 2009 to try to gain wide acceptability for the system.

Regulatory changes in relation to acoustic testing, air leakage testing and the use of robust standard details have arguably improved the quality of all types of system. Small improvements, for example sealing the ends of built-in joists to restrict air leakage, are incrementally improving performance, now often tested on whole buildings.

Improved masonry process

- To produce improved and innovative techniques of masonry construction to improve its quality and economy, and to propagate the results of this work.

Figure 3 WI beam construction cleat detail

Figure 5 WI beam construction test wall

Clearly some of what has been said about innovative systems does relate to improved or innovative techniques, for example thin-joint masonry involves the use of a new type of applicator rather than a trowel. Perhaps two of the main improvements in technique in the period have been in the field of mortars. The rapid increase in the use of silo mortars and ready-to-use retarded mortars have led to improvements in the accuracy of mix proportions over that of site-batched mixes.

Figure 4 WI beam construction shear connectors

Technology of material interfaces

- To cooperate with other material sectors within the construction industry in examining the technology and practice at the interface between different elements of construction; to develop appropriate design solutions for these interfaces; and to publicise and propagate the results of this work.

Progress has been made here, for example on new knowledge of the interaction of multi-storey timber and concrete constructions when clad in brickwork. There is even one exciting practical steel frame example in Winterton House, East London.

At Winterton House a steel-frame building needed to be reclad. The cladding was failing and the opportunity was to be taken to include additional concrete in the floors so as to provide better acoustic isolation between apartments.

Figure 6 Winterton House general view

Figure 7 Winterton House support to transfer structure

However, the steel frame could not sustain further load: it had been in its day a most economical award-winning design. The engineers decided to build brickwork separate to the frame and surrounding the frame up the complete 26-storey height (see **Figure 6**). At roof level a steel transfer structure sat onto concrete supports at the top of the brickwork. The transfer structure was pushed vertically upwards off the brickwork using jacks. The transfer structure then transferred force out of ties connected to the columns of the original frame and hence taking any additional force out of the frame, through the transfer structure and into the brickwork (see **Figures 7** and **8**). This novel approach was highly dependent on knowledge of the likely relative movements due to load, creep, moisture and thermal effects in order that any jacking movement was not, for example, eliminated. Extensive research was carried out into what are all interdependent effects so that any adjustments to the locked-off movements would be minimal and not frequent.

Overall, however, the current preoccupation with prefabrication, although not a major influence in the marketplace, is reducing interest in the understanding of interactions. Clearly a structural insulated panel system (SIPS) panel or otherwise, even if clad in brick or block slips, does not require such fundamental engineering understanding.

Figure 8 Winterton House transfer structure

One area not really covered by the review process, and for fairly obvious reasons, is the area of refurbishment and reuse of existing masonry. This is an area where we have improved significantly. There can hardly be a town or city in the UK where some existing masonry buildings have not been refurbished and reused even after standing derelict in many cases for a number of years. Not everywhere can claim the spectacular success of a St Pancras but there are many examples of mills and warehouses which now form smart waterside dwellings and rejuvenate run-down areas of city centres. We seem to be learning increasingly that we need to understand the design and the skills of the craftsmen that built the original in order to be able to justify new loading regimes. We are better at sympathetic repairs and cleaning and at taking a view about likely longevity going forward, the answer to whether the investment is sound. These are skills not to be overlooked; sustainability is not all about reuse of materials into new construction – we have the opportunity to recycle the workmanship and labour of bygone times with staggering effect and the most sustainable credentials.

This chapter has shown that there have been, in recent times, many developments in masonry that will hold it in good stead for the future of the industry. The review of *Masonry 2007* showed that on most fronts identified by the Masonry Industry Alliance some progress has been made. The additional material since that review demonstrates that the rate of development has not slackened, whether it be due to pressures for sustainable buildings or practically inspired ideas such as the WI beam. The move to the Eurocodes will not be that simple but for those that embrace them there will be opportunities. For my part I hope that the increased knowledge that we have developed since the introduction in 1978 of BS 5628 and the enhanced research database enables us to justify some reductions in what are still high global safety factors and a general improvement in the economy of masonry.

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CONTENTS

Chapter 35

Defects in masonry walls

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Bricks and blocks, either clay or cement based, are manufactured products, and therefore by their nature require raw materials to be won, processed, put through a manufacturing process and then shipped to site, and finally used in construction projects. At each stage of this process there is the potential for problems to occur resulting in defects and failures of the bricks and blocks themselves or the brickwork and blockwork masonry structure. These vary in significance and range from serious structural implications through to just an impact upon the aesthetic quality of the finished structure. Examples of some of the commonly occurring defects that affect brickwork and blockwork (inclusive of cement-bonded reconstituted (manufactured) stone, aggregate concrete and aerated autoclaved concrete) masonry are presented here.

Material defects

Raw material derived

As with all manufactured construction products, raw materials are the first ingredient in the process. It is at this stage that some of the defects encountered in the finished product or buildings can stem.

Many of the raw materials that go into clay- and cement-based bricks and blocks are ‘naturally occurring’ geological deposits. These materials are therefore naturally of a variable nature, either through chemical or mineralogical composition. It is this variability that needs to be understood and controlled if the manufacturer is to produce a product of known properties and to a predetermined quality. Control of the raw materials and continual monitoring is vital if defects are to be avoided.

Construction product manufacturing is like life: subject to the odd period of turmoil, and from time to time all manufacturing processes have their ‘bad days’. When raw materials are involved, the cause of a defect is often the result of the inclusion of a reactive mineral that is unexpected under normal circumstances. The following examples highlight some of these and show how even a relatively small amount of a ‘contaminant’ can cause significant problems with the finished product.

Iron and lime ‘blows’ or ‘pop outs’

During the manufacturing process for clay bricks, the principal raw materials are silts and clays from sedimentary deposits. Each of these has its own specific characteristics, mineral composition and therefore chemistry. Occasionally, due to geological processes, minerals that can cause problems within the manufacturing process can become incorporated within the clay raw material feed and end up in the product.

Lime, or more correctly calcium carbonate ($CaCO_3$), in the form of limestone or chalk is a naturally occurring mineral. It can be found within clay deposits, as well as from external sources; in either case the result without correct raw material preparation and processing can lead to problems post firing.

The reaction starts during the firing process when the calcium carbonate breaks down.

Within the kiln the following reaction occurs:

$$\underset{(\text{Heat}\rightarrow 600^\circ\text{C})}{CaCO_3} \rightarrow \underset{(\text{Solid})}{CaO} + \underset{(\text{Gas})}{CO_2}$$

When the bricks come out of the kiln the CaO (lime) absorbs moisture from the atmosphere and converts to $Ca(OH)_2$ (calcium hydroxide). This reaction results in a rapid expansion of the mineral grain causing a build-up of expansive pressure.

$$CaO + \underset{(\text{Moisture})}{H_2O} \rightarrow \underset{(\text{Expansive reaction})}{Ca(OH)_2}$$

If, as a result, the expansive force is greater than the confining strength of the brick, the ceramic matrix of the brick will fail and a ‘pop out’ generated. A portion of the face of the brick is ‘blown’ off, often revealing the culprit mineral grain as a white or grey grain at the bottom of the crater (**Figure 1**).

Iron blows are very similar in appearance, however they appear as black or brown mineral grains at the bottom of the crater. The reaction resulting in an ‘iron blow’ (**Figure 2**) is similar, however the original mineralogy is often an iron carbonate, namely siderite.

Siderite breaks down during the firing process, releasing carbon dioxide as a result. The iron oxide remaining again hydrates post firing, resulting in an expansive reaction.

Figure 1 Example of 'lime blowing' or lime 'pop out'. Note the white mineral grain at the bottom of the 'crater'. This is the remains of the expansive (reactive) grain. It is common for this grain to be either powdery or hard in nature

The solution to this problem is either avoidance of the expansive mineral grains in the raw materials, when sourced, or, more commonly, the design and installation of specific clay preparation equipment in the brick factory.

In general terms, grains of >1 mm are the main cause of the loss of significant portions of the brick faces. The potential expansive forces are proportional to the size of the reactive grain. Reactive expansive grains >5 mm ∅ can result in craters 20–25 mm deep and 35–50 mm across. In extreme cases, bricks with numerous reactive expansive grains can 'fall apart' in a relatively short period of time after production.

One technique used to avoid this problem is to grind the clay raw material feed to a particle size below 1 mm. Brickworks, where calcite is a common mineral type within the deposit, will tend to grind to below 0.5 mm.

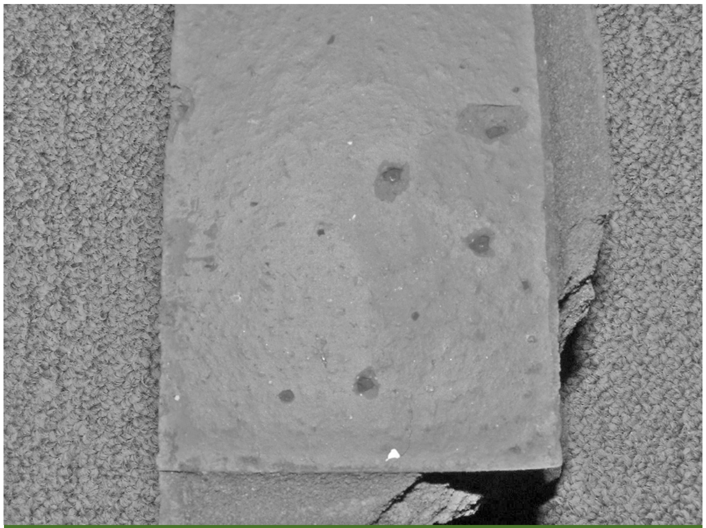


Figure 2 Iron 'blow' or iron 'pop outs' resulting from the hydration and expansion of a reactive iron-based mineral grain within the clay body

Alternatively, it is possible to reduce the reactive expansion post firing by rapidly saturating the bricks by 'dunking' them in a water tank. The rapid hydration of the expansive grains reduces the expansive forces, therefore minimising the damage.

As well as naturally occurring calcite in limestones and chalk, and aragonite (the shelly form of calcium carbonate) in sedimentary deposits dug for brick making, it is possible to have contamination of clay raw materials from external sources. Often stocking grounds are floored using recovered road scalpings or crushed concrete, both of which commonly contain large amounts of limestone aggregate. This can then be mixed with the clay when the loading shovel works the bottom of the stockpile. In addition, aggregate left in the back of lorries when transporting clays to a brickworks can also be the root cause of problems.

The 'mundic' problem

The same issues arise from the production of cement-bound aggregate concrete blocks, where often the inclusion of 'reactive mineral' has caused aesthetic and structural problems.

One such issue is referred to as 'mundic concrete'. This was an issue of particular concern in the south-west of England in Cornwall and some parts of Devon. The problem arose as a result of the mineral spoil heaps that litter the landscape in this part of the UK. The spoil was the waste from the mining and quarrying activities of the nineteenth century and the first half of the twentieth. This spoil was used as a ready alternative to quarried aggregates and used to produce concrete and concrete products.

Unfortunately the spoil, along with good aggregate, also contained 'gangue' minerals such as pyrite, or 'fools' gold' from where the term '*mundic*' comes, and other metallic sulphides associated with the tin and other metal ore. The problem came about from these minerals being unstable in the concrete and, over time, reacting in an expansive way, breaking the cement bonding and reducing the structural strength of the concrete and blocks made from the concrete.

The problem is caused by moisture-enhanced breakdown through oxidation of the sulphide minerals. In pyrite this reaction results in the formation of an iron hydroxide mineral, limonite and sulphuric acid. Both volumetric change and the formation of a corrosive acid within the body of the concrete blocks result in a breakdown of the bonding and loss of strength along with the formation of secondary reaction minerals such as gypsum, ettringite and thaumasite – classic sulfate attack reactant products. Both ettringite and thaumasite are well-documented, expansive minerals that can generate extreme expansive forces that can physically break the cement bonding and thus structural competence of the concrete (Bromley and Pettifer, 1997).

As a result, significant amounts of properties and structures built with this type of concrete and concrete products from the late 1800s through to the 1950s became blighted.

Concrete blocks manufactured through the 1920s and 1930s have shown such a significant deterioration that specialist surveys by RICS-registered surveyors are now required in order to obtain mortgages.

Aesthetic issues

In addition to reactive minerals that can cause a mechanical defect within the structure of a brick or block, there are times when the same types of minerals, for example pyrite and iron sulphide, which can cause aesthetic problems.

Figure 3 shows such a case where an aggregate particle rich in pyrite has been incorporated within the aggregate used. While being a naturally occurring component of the gravel deposit used for the production of this product, the sporadic and random distribution of the pyritic gravel aggregate in the gravel deposit makes this an unavoidable consequence. In other deposits, it is possible to avoid such mineralised zones, thus avoiding the problem.

The cause of the discolouration is due to the oxidation of the pyrite and conversion from an iron sulphide mineral to an iron oxide.

$$2Fe_2 + 7O_2 + 2H_2O \rightarrow 2Fe^{2+} + 4SO_4^{2-} + 4H^+$$

The Fe^{2+} ferrous form of iron is relatively unstable and therefore quickly converts to Fe^{3+} ferric state and oxidises. The formation of both the sulfate ion and free hydrogen when in an aqueous process often results in the formation of sulphuric acid, which then reacts with other minerals and forms additional secondary reaction products; in cement-based products these are often gypsum or other calcium sulfate compounds.

Figure 3 Pyrite oxidation resulting is a significant iron oxide stain on a concrete block. The problem has arisen due to the inclusion of crushed pyritic bearing aggregate in the concrete

Manufacturing process derived

Problems with bricks and blocks can also occur as a result of problems with the actual manufacturing process. In both clay- and cement-based products, chemical or thermal reactions take place that result in changes to both mineralogy and structure of the products.

Cement-based products

The basis for the use of nearly all cement-based construction products revolves around the hydration of the 'clinker' phases produced during the cement manufacturing process. Typically four 'clinker' phases are produced in the production of 'ordinary Portland cement' (OPC):

1. dicalcium silicate (C2S)
2. tricalcium silicate (C3S)
3. tricalcium aluminate (C3A)
4. tetracalcium alumino ferrite (C4AF).

The addition of water to all of these powdered 'clinker' phases when in 'cement' results in the formation of new cement hydrate mineral phases. It is this process that creates the bonding that binds the aggregate grains together for this 'artificial rock'. (For further information on the hydration processes of cement see Neville, 2008.)

Concrete and concrete products are therefore a combination of a binder (cement) and aggregates to 'bulk' out the mix. It is often issues, with regard to the correct proportions of binder to aggregate or the correct ratio of water to binder, that can result in problems with the finished product.

Weak concrete blocks are rare in this day and age of automated batching plants; however, occasionally, concrete blocks are produced following a problem with the correct batch mixing process. If the mix is deficient with cement there is often not enough to bind the aggregate together and form a concrete of the desired strength.

Similarly, if too much or too little water is added to the 'dry' components, this too can have an adverse impact upon the properties of the finished product. Too little water and there is not enough to get the desired hydration of the cement, therefore the bonding between the aggregates is poor, resulting in soft and friable blocks, or when too much water is added the mix becomes too wet and there is separation of the aggregate in the moulds resulting in a non-uniform block.

Fired clay products

With fired clay products, rather than a chemical reaction through hydration forming the bonds that hold the aggregate particles together, as in concrete, it is the formation

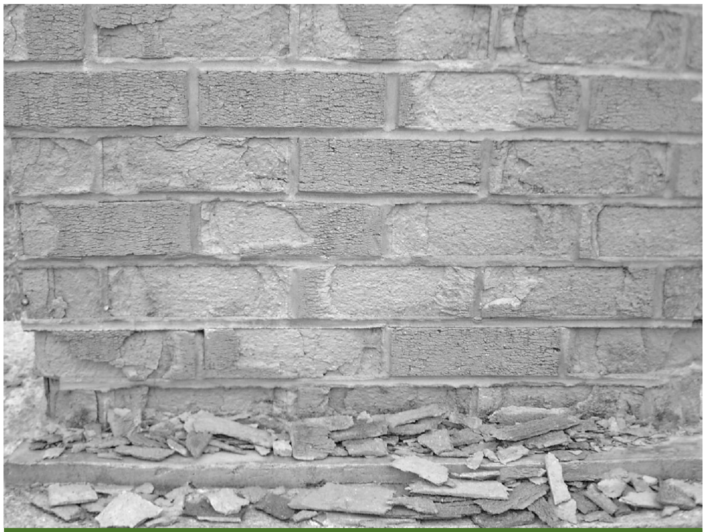


Figure 4 An example of the 'classic' delamination failure due to cyclical frost action on a free-standing wall. In extreme cases the face of the brick comes off as a single 'brick slip'

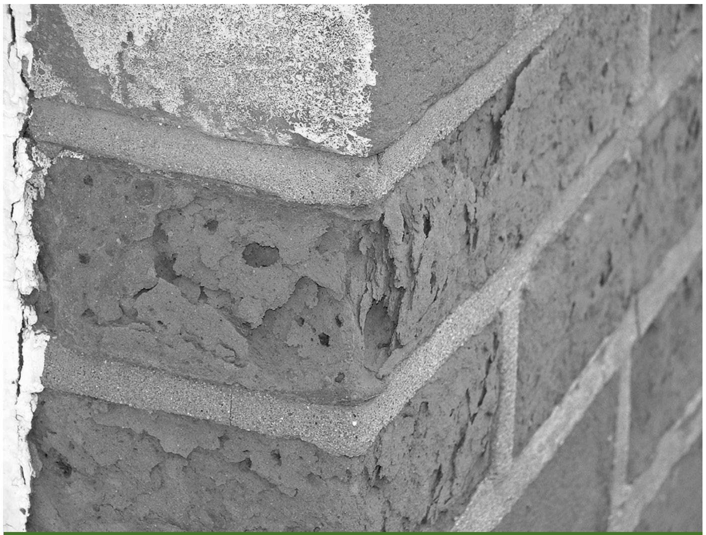


Figure 5 An example of the damage resulting from frost action on a low-temperature (soft fired) soft mud 'stock brick'. Note the scaling and crumbly nature of the brick, indicative of a general breakdown of the ceramic bonding by frost action

of a glassy melt (ceramic bond) that forms through the thermal transformations of the minerals within the kiln.

Most clay construction products are fired between 950°C and 1200°C, depending upon the clay type and mineralogy of the raw materials. At these temperatures a ceramic flux is formed which essentially is the 'glue' that holds the final fired product together. During the firing process the amount of ceramic flux formed is related to mineralogy and chemistry, as well as firing temperature, often referred to as 'soak temperature'. The thermal processes that form these fluxes need both time and temperature to develop; if either are not achieved, the finished fired product can show significant problems with its physical properties, such as durability, strength, size, water absorption and aesthetic characteristics.

One commonly occurring problem with fired brick masonry is that of 'frost failure'. Fired clay masonry products are rarely totally impermeable to water, therefore when water does get into the pore structure of a brick and freezes, the formation, expansion and pressure generated by the ice can at times result in damage to the bricks.

In general, pores below 1 μ (micron) are thought to be the source of the highest expansive forces generated during a freezing event. If the ceramic bonding (the glass formed from the cooling of the ceramic flux) is not strong enough to withstand the expansive forces of the ice crystal growth, small micro-cracks are generated within the body of the brick, typically parallel to the exposed face of the brick. Over time, repeated freezing and thawing cycles while the bricks are saturated, result in the propagation of these cracks until they form planes of weakness that result in the brick disintegrating by crumbling, spalling or delamination (a series of flakes of brick coming off the face).

Bricks within Europe are classified as being either F2 (fully frost resistant) or F1 (moderately frost resistant). It is therefore important to check the level of exposure and the likelihood of the brickwork becoming saturated for long periods of time when specifying the brick type.

Figure 4 shows a common form of frost damage found in extruded clay bricks that are frost susceptible, either through a problem in firing or as a result of the wrong frost resistance class of bricks being used in a particular exposure situation.

Figure 5 shows an example of a relatively weak brick, in compressive strength terms, that has been slightly underfired, either too low a top temperature, or not 'soaked' for long enough. The result is a general crumbling of the face of the brick and the production of 'scales' which when touched turn to sand. This example is indicative of a general loss of ceramic bonding due to the action of frost cycling when the bricks are wet.

Figure 6 shows an example of this with very localised conditions, often within an element of a structure, resulting in elevated saturation levels and thus a higher degree of risk to frost action. In this case the wall is a part of an external flight of stairs and the damage corresponds to the points where the treads of the steps are to be found on the other side of this wall. Here, 'ponding' of water on the steps appears to be soaking into the brickwork at a specific level and results in bricks that would on the whole be suitable for this level of exposure, as evidenced by the lack of damage in the rest of the wall, but clearly failing as a result of their exposure to higher saturation levels.

Figure 6 Frost damage due to the localised conditions of use. The damaged bricks here follow the position of the flight of steps on the other side of the wall. The bricks are therefore exposed to a higher level of saturation than the rest of the wall

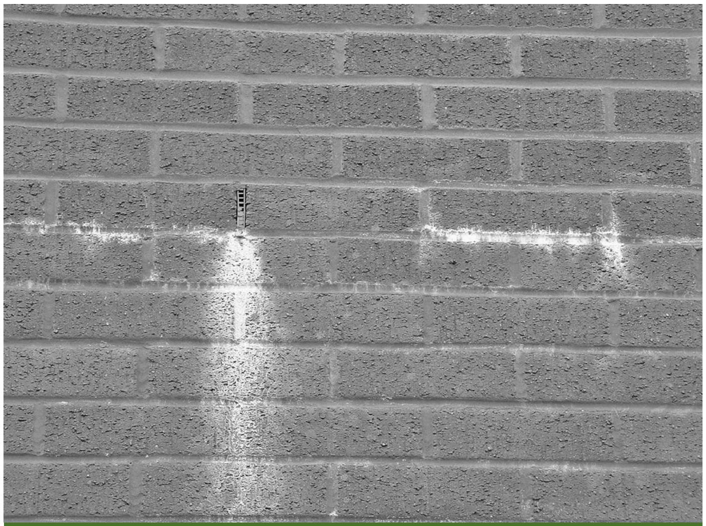


Figure 7 Deposits of calcium carbonate below a weep hole, as a result of the movement of water-soluble calcium hydroxide from the internal aggregate concrete blockwork being deposited over the brickwork

Surface staining

Surface staining in work is a problem that can affect both bricks and blocks. Staining often is the result of the dissolution of a compound, transportation and deposition on the surface of the masonry. Often the dissolution takes place in the presence of rainwater. Even without the added impact of industrial pollution, rainwater is generally slightly acidic due to the presence of dissolved carbon dioxide; therefore, strictly speaking, acid rain is common, although often the acidity is very slight and in the form of 'carbonic acid'.

Therefore, most staining issues are generally related to the reaction of certain compounds found within concrete or clay products being mobilised and redeposited where the water evaporates, normally the surface of the masonry.

The most common mobile compound in cement-based concrete products is calcium hydroxide ($Ca(OH)_2$). Commonly referred to as 'lime leach', this often appears associated with either blockwork or concrete-based products that have become saturated during the construction period, therefore any excess moisture runs over the surface rather than soaking into the blocks. This can also happen with mortar and low water absorption bricks, or when brickwork has not been protected from the rain during construction. The results are the deposition of a white material onto the face of the blocks or bricks. Commonly seen below vertical mortar joints in bricks or where the movement of moisture through a wall (earth-retaining wall) when no impermeable 'tanking' has been used.

Initially the deposit is calcium hydroxide, however this rapidly converts to calcium carbonate. The problem can be easily rectified with the use of a weak solution of hydrochloric acid, commonly sold in builders merchants and DIY stores as 'brick acid' or 'brickwork cleaners'.

Figure 7 shows an example where internal aggregate concrete blockwork has become wet during construction and the movement of moisture out through the weep hole at a floor level cavity tray has resulted in the deposition of calcium carbonate 'lime leach'. This aesthetic issue was resolved by professional brick-cleaning specialists; however, careful application of a dilute brick acid solution would also have been appropriate in this case.

With clay bricks, the mobile compounds tend to be slightly more exotic in nature, reflecting the variability of the chemistry of the clay deposits and the firing process used.

Mobile water-soluble compounds of iron, manganese and vanadium are examples of such stains, and relate specifically to the clay types and or pigmentations and firing conditions, and as to how and why these deposits are created.

Iron and manganese stains are often found associated with bricks manufactured from clays in the Weald area of the UK (East and West Sussex and Kent). The Weald clay when fired often forms a 'black reduction core' within the bricks, while the outer rim of the bricks are oxidised and generally produce red-, brown- and purple-coloured bricks.

The significance of the black core is that this is generated under reduction conditions starved of oxygen. As a result many of the iron and manganese compounds are relatively unstable and can, under slightly acidic conditions that are generated when water permeates the brick, become very soluble and mobile.

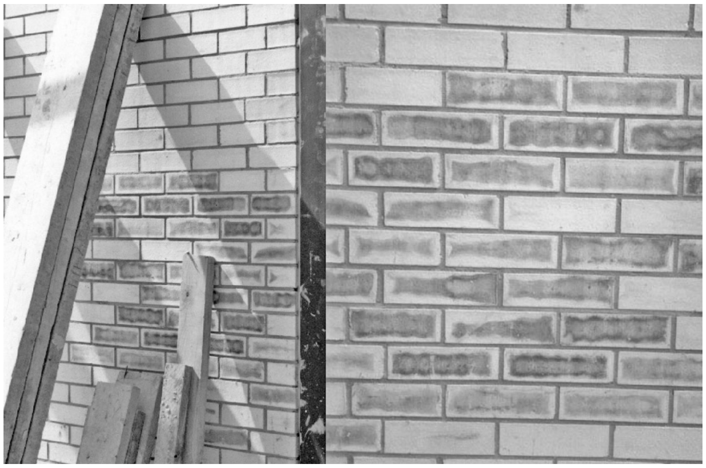
Figure 8 shows an example of this type of staining, but in this case associated with a brick type manufactured in

Figure 8 Bricks from mainland Europe which have a similar firing characteristic as the Weald bricks in the UK. In this case the iron and manganese compounds have become mobile and have been deposited on the face of the bricks, mortar and stone string course

Europe, but of a similar nature to Weald clays. In this case the iron and manganese compounds have become very mobile and have been deposited both on the face of the bricks and mortar but also have washed over the stone string course. This problem was rectified by professional brickwork cleaning using a combination of weak acids.

Figure 9 is a Weald soft mud brick where the iron staining has not affected the brick, but has stained the pale cream-coloured mortar. Here deposition tends to be very rapid as the pH of the mortar results in the iron being precipitated only on the mortar joint.

Figure 9 Deposition of iron staining on mortar joints as a result of a pH change in the transportation solution



(a)

(b)

Figure 10 (a) and (b) Vanadium staining. Once seen never forgotten. Typified by vivid yellows and green staining on buff and pale cream fireclay bricks

Vanadium staining of buff or pale cream fireclay bricks are probably the most striking mineral staining that is to be found in clay bricks. Vanadium compounds are commonly found within some fireclays and can become mobile under the right conditions. The deposition on the surface of the buff bricks is spectacular, and can result in vivid 'dayglow' greens and yellow stains. Over time these darken to deep yellows and greens (**Figures 10(a)** and **10(b)**). Dissolution, transportation and deposition of these vanadium compounds are all controlled by the oxidation state of the vanadium. After firing the vanadium compounds are highly soluble but once on the surface of the brick, react with the air and over time change to become stable and difficult to remove.

The standard method for vanadium stain removal is the use of oxalic acid; however, if a hydrochloric acid based cleaner is used by mistake, the stains can be permanent. It therefore pays to know your stains before cleaning begins!

Efflorescence

Efflorescence is again another type of soluble salt deposition; however, this term is most often used for soluble sulfate salts that are deposited on the surface of brickwork.

There are four main sulfate salts: calcium, sodium, potassium and magnesium. The most common type of efflorescence salt is that of potassium sulfate. These are naturally occurring salts that form within the brick during the firing process. These salts are all soluble; however, potassium, sodium and magnesium sulfates are significantly more soluble than calcium sulfate.

These salts are transported to the surface of the bricks by the capillary action of the drying of the brick surface and thus become deposited on the brick face as it dries out. All these soluble sulfates produce typically white 'fluffy' deposits that are easily removed with brushing; however, some twin salts and the calcium sulfate salts (gypsum) are more difficult to remove.

Traditionally, brickwork built in the autumn and winter period would exhibit a 'spring bloom' where the brickwork could literally turn white over a couple of days when the

Figure 11 Efflorescence, commonly associated with the movement to the surface of the brickwork of water-soluble sulfate salts, in this case potassium sulfate

weather got better in the spring and the sun started to dry out the building, drawing the moisture and salts to the surface (**Figure 11**). If potassium or sodium salts were present, these would disappear during the next shower, only to return when the brickwork dried out again. This would happen for a few years and then disappear totally. This was, of course, due to all the mobile salts eventually being drawn to the surface, and the repeated drying and wetting would wash the salts off the brickwork and into the surrounding soil.

More persistent salts such as calcium sulfate often require chemical or mechanical cleaning to remove them successfully.

One of the main causes of excessive efflorescence is the formation of a 'black core' within the brick when fired. As with issues with iron and manganese previously described, black cores are also the source of a number of the water-soluble sulfate salts. **Figure 12** shows 'coal measure shale' common bricks manufactured in Staffordshire during the late 1940s. Note the white salt deposits around the black 'hearting' colouration on the surface of the bricks. These areas are where bricks touched each other during firing, thus not allowing the oxygen to get to the surface, and thus resulting in reduction markings. This is a common location for sulfate salts to be deposited as the levels in the black cores are significantly higher than in the surrounding oxidised rims. Even after 60 years' service, these bricks still show efflorescence following periods of wet weather, when the brickwork starts to dry out.

Figure 12 Efflorescence resulting from the formation of black cores and surface reduction markings (black areas) on 60-year-old Staffordshire Coal Measure Shale common bricks

Cement- and aerated autoclaved concrete (AAC)-based blocks, if exposed to wet conditions, can also develop efflorescence. This commonly occurs as a result of the brick and blockwork not being suitably protected during the construction phase of any 'new build'. Once wet, the blockwork dries out slowly, and it is not unknown for blockwork to still be wet even after the building has become weathertight (roof and windows/doors). **Figure 13** shows an example of AAC insulation blockwork where efflorescence has developed as a result of the wall becoming wet during construction and then drying out slowly. The efflorescence here is typically of a similar composition to 'lime leach', i.e. calcium carbonate.

Fire cracks

Fire cracks are cracks that often occur as a result of drying the bricks too quickly, and differential shrinkage occurs between the outside and inside of the bricks. When the bricks enter the kiln, the cracks open up as the firing process progresses. As the cracks are present during the firing

Figure 13 An example of internal insulation blockwork showing the deposition of water-soluble salts during the construction phase

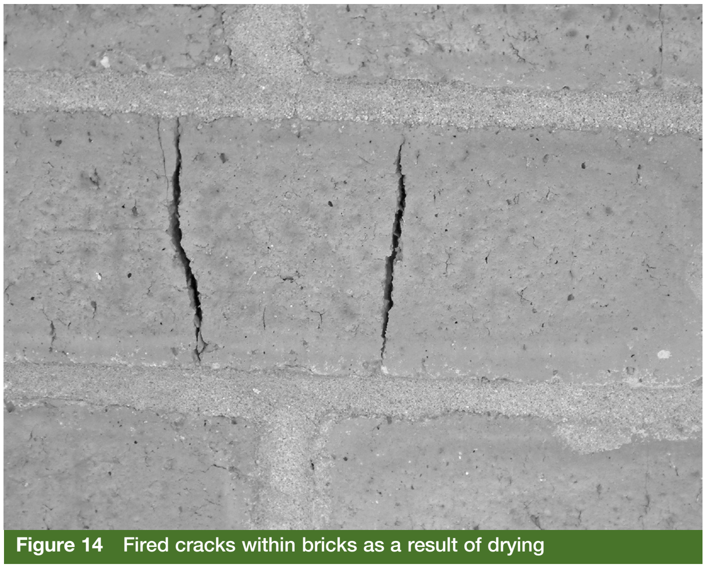


Figure 14 Fired cracks within bricks as a result of drying

process, the surface of the cracks tends to be as well fired as the outer surface of the brick. Cracks present before walling, that run through the whole height of the brick, often have a wedge of mortar in them.

Figure 14 shows 'fired cracks' where the cracks do not have a significant impact upon the edge of the brick, but are significant within the face of the brick. The sigmoidal shape is typical of this type of cracking. Unless the cracks result in the ingress of water, through the brick, these bricks are likely to be as durable as similar ones without the cracks of the same brick type. **Figure 15** shows an example of multiple-fired cracked bricks built into a house wall.

Dunting

'Dunting' is a term used throughout the ceramics industries and refers to a crack that is generated as a result of the sudden release of stress within a fired ceramic body while still in the kiln and typically cooling between 700°C and 600°C.

Figure 15 A wall built using fired cracked bricks

During the firing process silica-based minerals change their structural state, the most common of these being quartz in clays used to manufacture bricks, pipes, roof tiles and pavers. Quartz mineral grains, during the firing process, increase in volume by approximately 1% at the $\alpha - \beta$ inversion temperature of 573°C. This remains stable while the brick body temperature is over this threshold temperature; however, during the cooling stage, when passing through the inversion temperature again, the process is reversed. Care is needed, therefore, during the cooling stage of the firing process to allow time for the localised stress build-up in the bricks to be dissipated, rather than change suddenly.

Should a rapid cooling event take place through the temperature-sensitive zone, typically between 650°C and 500°C, it is possible to generate 'dunting'. These are best described as cooling cracks that result in a very smooth glassy crack surface, often resembling a highly polished surface, rather than the more typical dull fracture surface generated by a mechanical action. Rather than the crack following a plane of weakness, typically the interface between grains, 'dunting' results in a crack that runs through mineral grains and the glassy matrix.

This problem is also known when the silica mineral cristobalite also undergoes a similar inversion but at lower temperatures. This can often be recognised when the doors of the kiln are being opened to remove a batch of bricks. Sometimes 'pinging', a metallic ringing sound, can be heard as the sudden rush of cooler air enters the end of the kiln and some bricks 'dunt' instantaneously.

Post-manufacture problems

Chipping

Surface damage of any masonry product can occur during the handling, storage and transportation of the products to site. Cracking and chipping are the most common, and they result from a mechanical impact. While broken masonry units are easy to spot in the packs delivered, ones that have hairline cracks that result in chipping can on occasions not be spotted during construction, or 'a blind eye turned' by the operatives.

Surface chipping/spalling as a result of hairline cracks in bricks can be brought to notice following the 'building in' of these products and a period of time in service. Hairline cracks offer a route in for water, and such cracks can fill with water and then, during the winter, frost action can propagate the cracks resulting in the appearance of the chips or spalling. An example of such a chip is shown in **Figure 16**.

Chipping and spalling can be used to create a 'distressed' aesthetic appearance to the bricks (**Figure 17**), and it has become routine in the last 15 years for bricks to be

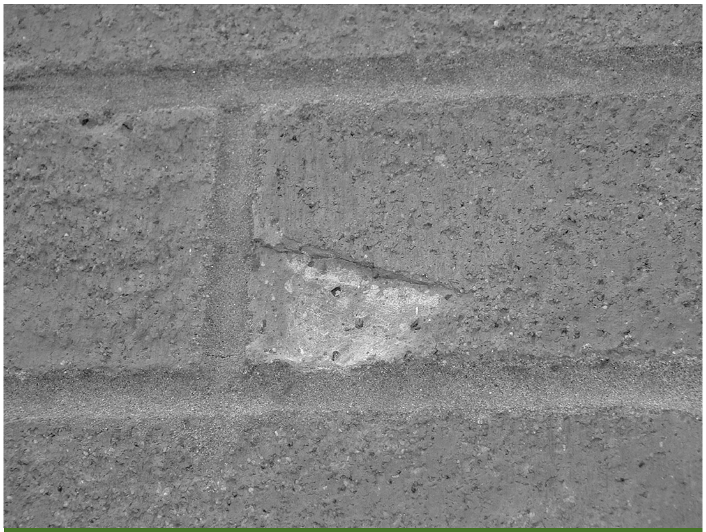


Figure 16 Spalling/chipping as a result of the action of water ingress into a hairline crack and subsequent frost action propagating the crack to the point of the total loss of bond. The original hairline crack was more than likely a result of mechanical impact

deliberately distressed by 'tumbling' in a rotating pipe to generate an 'old world' look. When done well with the right brick type, the look can be attractive.

Salt fretting

Salt fretting is a term commonly used in the USA and Australia to describe the breakdown of masonry structures exposed to the movement of salt-rich groundwaters being drawn up into the brick/blockwork. As the water evaporates, the salts are deposited in the pore structure of the masonry products and an expansive force similar in nature to that of ice crystal growth in frost damage occurs.

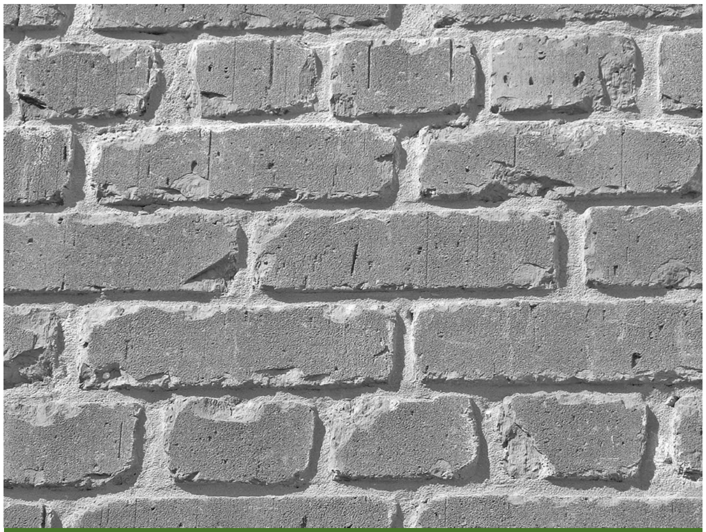


Figure 17 Deliberate mechanical damage through 'tumbling' used to generate a 'distressed or old world' look

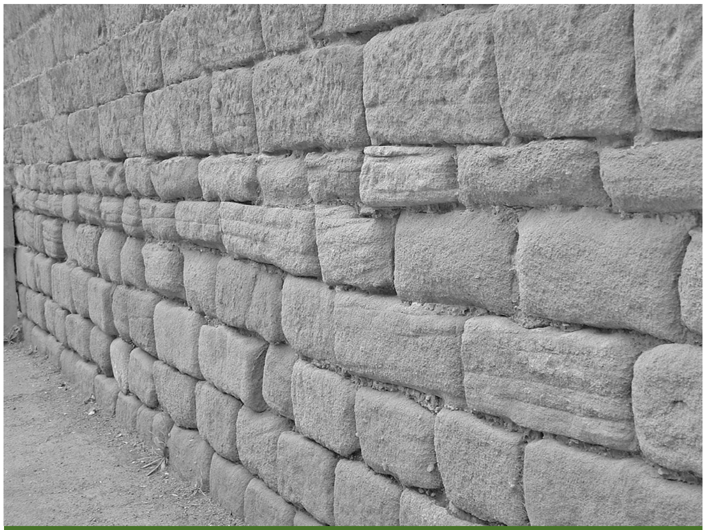


Figure 18 Salt erosion (fretting) of sandstone masonry blocks as a result of the action of de-icing salt spray since the 1950s

In the UK we can see very similar features, however not usually as a result of naturally high-salinity groundwaters, typically sodium chloride rich in the USA and Australia. In the UK and Europe, salt fretting can occur as a result of de-icing salt use on highways in the winter. The resulting 'salt spray' typically has a higher salinity than seawater, which when deposited onto brickwork, blockwork and, in some parts of the UK, stone masonry: the effects are dramatic.

In general, salt fretting can be distinguished from frost action on the basis that frost action tends to work progressively into the face of the brick/block; however, salt fretting typically has an impact upon the whole brick/block and the units generally just crumble away.

In such cases, the two processes of salt fretting and frost action work hand in hand, the salt fretting 'softening' up the brick/block, and the ingress of water and freezing action results in a rapid breakdown of the unit.

Figure 18 shows an example of a natural stone masonry wall adjacent to a highway in Derbyshire. The building itself is part of the first industrial water-powered cotton mill complex built by Richard Arkwright in 1771. It is only since the 1950s that the widespread use of de-icing salt in the UK has resulted in the erosion of masonry structures adjacent to the road network.

Salt fretting can also be found in 'earth-retaining walls' where the movement of moisture from the soil through the brickwork is not inhibited by the use of an impermeable membrane. This can result in both surface deposits of salts as seen in **Figure 19**, and sub-surface deposition resulting in blisters, in **Figure 20**.

In both cases the main culprit is calcium sulfate (gypsum) salts. Note the abrupt change in appearance in **Figure 19** between the levels, where the soil is in contact

Figure 19 Salt fretting of an earth-retaining wall. The bricks below the ground level on the rear of the wall are heavily encrusted with calcium sulfate (gypsum) deposits, while above ground level the higher moisture content has resulted in frost damage

Figure 20 Example of gypsum (calcium sulfate) 'salt' crystal growth resulting in the 'blistering' of the brick face. Commonly seen on old earth-retaining walls and where infill behind walls, to allow drainage, are composed of furnace bottom ash and clinker/conders. Both being naturally high in soluble sulfates

with the wall (to the rear of the wall), and above, where frost action due to elevated moisture levels occurs just above soil level. Analysis of the bricks in the bottom four courses shows very high calcium sulfate and potassium sulfate levels whereas the fifth course and above have normal soluble salt levels.

Figure 20 shows an example often associated with historic buildings, more specifically garden walls and earth-retaining walls in the grounds of historic buildings. It was common to use ash and clinker from coal-fired boilers and furnaces to provide drainage behind such walls. Unfortunately, such ash and clinker was often rich in soluble sulfates which combined with the calcium in the mortar and formed gypsum blisters on the surface of the bricks.

Expansion and shrinkage

Problems can occur as a result of movement of masonry structures. Often this is attributed to problems with the foundations, settlement or similar 'external' processes; however, both clay- and cement-based masonry units can also exhibit movement as a result of moisture or thermal interaction.

There are three principal types of movement of masonry units: reversible moisture, irreversible moisture and thermal related movement. It should be noted that this movement can be shrinkage as well as expansion.

Table 1 shows typical ranges (%) movement for common masonry units, as a result of moisture; the values are commonly used in structural design, although where critical, the advice is to contact the manufacturer for actual values for the masonry units being used.

As can be seen for **Table 1**, cement-based blockwork, concrete aggregate blocks and AAC blocks have a tendency, post production, to slightly shrink. This can result in the formation of tensile stresses within masonry constructed from these products. In the extreme, and if provision for such shrinkage has not been designed for, cracking can result.

Figure 21 shows such a situation but with concrete bricks. The vertical tension crack has developed because the band

Masonry unit	Irreversible moisture expansion: %	Reversible moisture expansion: %	Thermal movement
	(+) indicates expansion, (–) indicates shrinkage		Coefficient of thermal expansion per °C × 10^{-6}
Clay bricks	+0.02 to 0.07	±0.02	5 to 8
Calcium silicate bricks	–0.01 to 0.04	±0.01 to 0.05	8 to 14
AAC light-weight insulation blocks	–0.05 to 0.09	±0.02 to 0.03	8
Dense aggregate concrete blocks	–0.02 to 0.06	±0.02 to 0.04	6 to 12

Table 1 Typical examples of masonry unit movement due to moisture and thermal conditions

Figure 21 Structural cracking of concrete brick masonry resulting from the lack of design detail (expansion joint) in a long 'run' of brickwork. Cement-based masonry products typically shrink during the first few months of use, and the design of masonry structures needs to take this into account

of brickwork of the front elevation of this house between the top of the front door and the window opening above is too great for the forces generated by the shrinkage of the concrete bricks to accommodate. The end result is a structural failure in the midline at a convenient point of weakness. Initially the crack will probably have formed in the vertical mortar (perpendicular) joints, and then propagated through the bricks as the stress is relieved.

Fired clay masonry products in general expand with time in service, thus generating a mismatch with concrete masonry products; often both are used together in cavity wall construction. Care in understanding the differential movement and the impact upon the use of wall ties and other ancillary components needs to be taken into account at the time of design.

While all masonry units exhibit reversible moisture movement, clay bricks in general exhibit some irreversible moisture expansion, depending upon the fired nature of the clay and the amount of glassy ceramic bond formed. It is the uptake of water into the glassy matrix that results in the expansion. The expansion of brickwork, in long expanses of uninterrupted walling, requires special attention and the insertion of expansion joints to avoid structural failure of the walls. Fireclay bricks can have the largest irreversible moisture expansion characteristics and it is known for there to be a need to be able to accommodate 0.7 mm expansion per m of walling.

Design guidance for masonry walls in BS 5628-3: 2005 works on movement of clay masonry at 1 mm per m, therefore recommends that expansion joints of 16 mm width should be used for every 12 m of linear continuous unrestrained walling, or every 15 m of continuous restrained walling. These expansion joints allow for the periodic movement through both moisture and thermal processes.

Thermal expansion, as the term suggests, results from the physical action of heat upon a masonry unit. In general, warmth results in an expansion, and cooling a contraction. This can operate on a diurnal scale (day–night) or a seasonal scale (summer–winter). The values in **Table 1** represent the coefficient of thermal expansion per degree centigrade, 0.001 mm per metre. Being reversible, when the heat source is removed, the units contract back to the original starting point assuming the temperature is similar.

Mortar

The link between masonry units, bricks and block, and masonry is commonly the use of a binder, mortar, that bonds the units together to form the structural element, the wall.

It is therefore important to look at some of the issues which affect mortar that can result in structural defects in the masonry wall. Mortar is not without its own specific group of technical problems. Like concrete, most mortars used today are derived from the use of a manufactured binder, cement, and an aggregate, sand. Modern cements are rarely pure cement, and in recent years the development of CEM II blended cements that use a base of ordinary Portland cement (OPC) blended with other cementitious materials such as pulverised fuel ash (pfa) or ground granulated blastfurnace slag (ggbfs), to create a binder that hydrates and forms a cementiceous bond.

As with any blended material, the final properties are dependent upon the correct quantities of the components being added at the right time. As with other concrete-based masonry products, critical to this is the correct amount of cement. Too little and the mortar will not bind together and form a coherent durable material. Weak mortars are very susceptible to normal weathering processes, such as wind rain and frost, that can result in a friable sandy mortar, with poor structural properties.

Sulfate attack

Mortars are also susceptible to chemical attack, as is seen in 'sulfate attack'. Sulfate attack is a chemical reaction between the tricalcium aluminate in cement (C3A) and the water-soluble sulfates in the bricks or soil waters. The resulting reaction forms a product known as ettringite, a calcium aluminosulfate hydrate that upon contact with water expands. Ettringite, once formed, is often crystalline; however, in the presence of free water (liquid) ettringite swells, generating expansive forces that can break the cementitious bonding structure of the mortar.

There are a number of 'telltale' features that can be used to identify sulfate attack on site, without the need for chemical analysis. Some of these can be seen in **Figure 22**

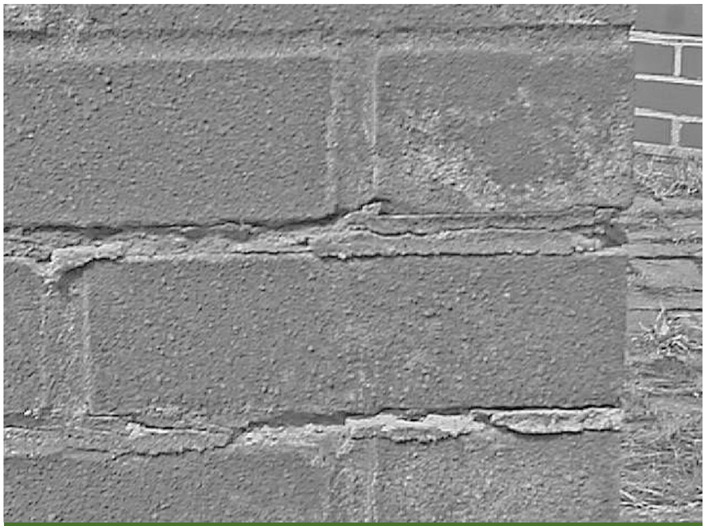


Figure 22 The effects of sulfate attack on cement-based mortar. The 'tooled' joint is often cracked along the centreline and behind the mortar is very soft and 'mushy'. Expansion of the mortar commonly results in the deformation of the masonry structure and, when seen in chimneys and boundary walls, often results in the need to rebuild the structure

Figure 23 Railway bridge masonry faced abutment with major structural cracking resulting from the formation of sulfate attack and formation of thaumasite. The source of the sulfate, in this case, is almost certainly from the use of 'cinders' and furnace bottom ash/clinker

which shows the central crack running down the length of the mortar joint and the soft 'mushy' mortar.

The expansion of the ettringite results in not only the breakdown of the mortar bond but can also have a dramatic effect upon the structure. Sulfate attack is often seen on boundary walls and chimneys, and the localised expansion on the face exposed to the wind-driven rain, often expands and causes the wall to bow.

Occasionally this can happen in structural walls within houses and can result in the need to remove all the external brickwork and outer leaf, and rebuild (reskin) the property.

The source of the sulfate is often hotly debated, normally as this results in blame and the cost of repair. As previously described, bricks can and do have a naturally occurring level of soluble sulfates that can cause sulfate attack, especially in very weak mortars (1:7 or 8, cement: sand ratio); however other, external, sources of sulfates can result in significant sulfate attack of the mortar without careful consideration and preventative measures.

One example of this is with 'earth-retaining walls' or in this case railway bridge abutments. **Figure 23** shows such a brickwork bridge abutment where sulfate attack of the mortar has resulted in structural cracking of the facing brickwork. In this case, though, rather than ettringite being the reactive by-product of the chemical attack, this is an example of the much rarer thaumasite attack being the root cause.

In the same way that ettringite formation, hydration and subsequent expansion causes the breakdown of the mortar, thaumasite formation has a very similar genesis.

Figure 24 shows a cored portion of the brickwork, with the presence of the blue-white thaumasite within the mortar joint. The source of the elevated reactive water-soluble sulfates in this case was from the use of 'cinders' to backfill the embankment. Over many years, the sulfate-rich waters, at relatively low temperatures, typical of thaumasite formation, have reacted with the mortar in the facing brickwork and resulted in the breakdown of the structural integrity of the masonry.

Structural defects in masonry walls

Engineers concerned with the structural design of masonry concern themselves with the usual principles. These are

Figure 24 Masonry core of the railway bridge abutment shown in Figure 23. The blue-white crystalline deposit in the mortar joints is the reaction by-product thaumasite

perhaps emphasised by the current focus on limit state design where the ultimate limit states of failure by compression, tension, shear and flexure are considered independently and through limits on slenderness the serviceability limit states of deflection and cracking are deemed not to be reached. In practice where structural defects occur in masonry walls it is rarely related to the concerns that we design against. In my career I have only once seen a case of compression failure in a building, that being due to an inappropriate change of use. That is not to say failures have never occurred, far from it – in the early days of infrastructure development and church building much was learnt from collapses. Scant attention was paid to health and safety, life was cheap, and the lessons learned were, in today's values unacceptably expensive. The development of the use of flying buttresses to resist the spread of arches and iron tie bars to restrict the bulging of walls demonstrate lessons learned.

Today, however, we incorporate quite massive safety factors against reaching the ultimate limit state, generally 4 to 5. Also, despite our generally accepted view that we build relatively slender masonry structures as opposed to the more massive wall constructions in central and Southern Europe, our structural walls are quite thick. Nevertheless our traditional housing construction where the 'sizing rules' that we follow in the Approval Document which supports our Building Regulations cannot be justified by structural calculations. However, tradition has demonstrated, along with some test work, that even with the solid, 'nine-inch' thick walls, through to cavity walling, the 'rules' devised for the former are still appropriate. In addition, many of our walls provide limited strength or stability to our buildings, they are simply hung on to outside as architectural features such that the fundamental structural competence is not relied upon.

The reality is that where structural defects occur, these are largely due to factors beyond those for which the masonry has been designed to resist, e.g. settlement or corrosion. That is not to say failures do not occur; they are, however, thankfully rare and usually in very specific circumstances. These range from wall collapses during demolition where, if an engineer had given some thought, it would have been obvious that an impossibly slender wall was being created by removing its support, to the much less well understood time-dependent effects causing the collapse of the historic masonry tower at Pavia in Italy (Binda *et al.*, 1992).

Overall in the UK, these modes of failure that we design against do not occur; that is not to say that cracking caused by, for example, tension or shear does not happen but that it has been caused by other mechanisms. The same is not true all over the world; especially in seismic areas the shear wave caused by earthquakes causes collapse of structures which have not been protected by tensile reinforcement. Distress in masonry is not uncommon but failure is perhaps one exception, being that of the triangle of masonry at the top of gable walls which do occasionally get 'sucked out' by wind forces. Inevitably this is due to inadequate tying back of the wall to the roof structure by steel straps as recommended in our Codes of Practice and Approved Documents. Laboratory tests have shown that it is relatively easy to cause such a triangle to crack along a bed joint at its base and then rotate about this joint (**Figures 25** and **26**). These tests included traditional (30 mm × 5 mm) steel straps from the roof structure restraining a concrete blockwork inner leaf to a brickwork outer leaf. The tests showed that a much improved detail was to fix the strap beneath the nearest rafter to the wall and for it to pass through, not over the blockwork (Templeton *et al.*, 1986a).

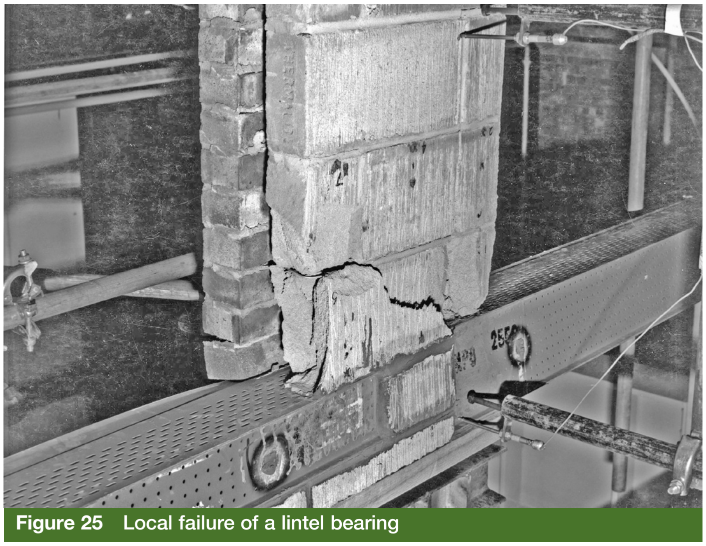


Figure 25 Local failure of a lintel bearing

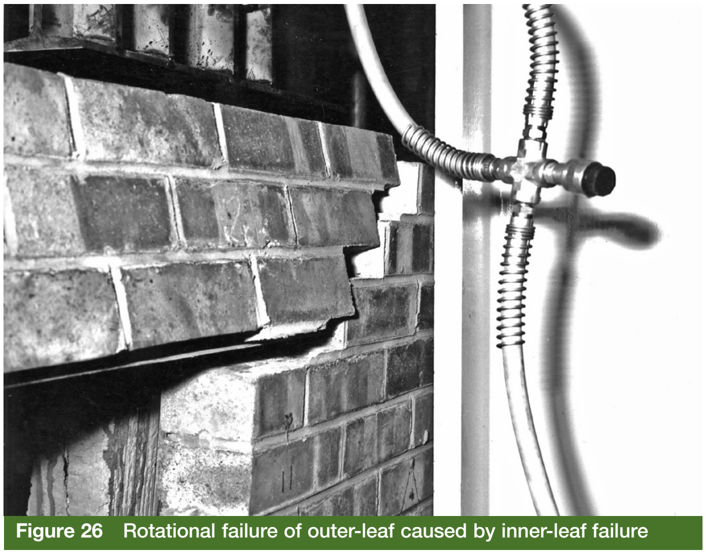


Figure 26 Rotational failure of outer-leaf caused by inner-leaf failure

Compression failures

As has been said, compression failures are rare. A good illustration as to why this is so is the experimental work on 'the Robustness of the Domestic House' in the 1980s (Templeton *et al.*, 1986b). In this case those responsible for Building Regulations in England and Wales became acutely aware that the sizing rules referred to earlier for, for example, maximum window openings had been developed from experience with solid brickwork walls and rafter and purlin roof construction. They were, however, in widespread use for cavity walls with trussed rafter roof construction.

In the solid brickwall case the roof load, and that from any snow, was transferred by the purlins into the gable and party walls, i.e. those with minimal or no openings. In the cavity wall case, the load was transferred onto the inner leaf of the front and rear walls, i.e. those containing the window and door openings and indeed often onto an aerated autoclaved concrete blockwork leaf with relatively low compressive strength.

Compressive strength tests on the front and rear walls to the house showed that the factor of safety against compressive failure was at least 3.5 over and above the weight of a tiled roof with a metre of snow on it (Edgell and de Vekey, 1985). More to the point, the failures were localised compression failures of lintel bearings.

Another relevant point is that we often concern ourselves with remediation of walls built with mortars made with relatively little cement. Where unscrupulous builders have tried to cut costs, mortars of mix proportions of 1:10 or 12 mixes of binder:sand are not that rare. However, we accept that for structural masonry the compressive strength is not strongly dependent on mortar strength, in fact the formula recommend in the National Application Document to the European Code of Practice (BS EN 1996-1-1 (Eurocode 6)) is:

$$f_{\mathrm{k}} = k f_{\mathrm{b}}^{\alpha} f_{\mathrm{m}}^{\beta}$$

where

f_{k} is the characteristic strength of masonry
f_{b} is the normalised strength of the masonry unit
f_{m} is the mortar strength, and
k is a constant
α and β are normally taken as 0.7 and 0.3 (BS 5628-3: 2005a and 2005b).

In some simple trials, the compressive strength of both brickwork and blockwork made with a 1:1:14 cement: lime:sand mortar was compared with that made with a 1:1:6 mix. The mortar strengths were 0.33 N/mm^2 and 2.23 N/mm^2 but the influence on the masonry strength was low. The blockwork strength was 2.6 N/mm^2 compared with 3.1 N/mm^2 and the brickwork 7.2 N/mm^2 compared with 9.4 N/mm^2. These tests suggest that at such low strength mortar, the dependency on mortar strength is less than indicated by the EC6 formula.

Tension failures

Tension failures caused by the application of an external load are rare. However, there are circumstances whereby roofs removed by hurricane forces can cause tensile failures at the points where the roof was tied down to the walls. These are somewhat special circumstances. Settlement can cause tensile failures. If settlement occurs under the centre of a long façade, the whole face tends to sag and it is possible for tension cracks to appear at the base of the wall and to progress up the height should the movement persist. If the settlement occurs at the ends of the wall or indeed the ground heaves in the centre, the façade will tend to hog and tension cracks may occur at the top of the façade and run down the wall.

A more common effect of settlement is where one wall or corner of a building settles causing diagonal tension to be established and diagonal cracking to occur. In many of these cases the settlement is relatively short-lived and the solution is to simply point up the cracks in order to prevent water ingress.

Tensile cracking can also be introduced due to movement in materials. For example, concrete and calcium silicate masonry are shrinkable materials, and in long lines of uninterrupted masonry, especially between rigid points such as intersecting walls, the restrained movement may be sufficient to exceed the capacity of the wall to resist the tension. The same may happen in clay brickwork, which tends to expand with time, where compression introduced by expansion can cause movement along a slip plane such as a damp-proof course and any subsequent contraction, due for example to thermal movement, can set up tension and cause cracking, (**Figures 27** and **28**).

Tensile cracking due to settlement or inherent movement of materials can be restricted by the use of prefabricated

Figure 27 Movement causing failure in a long run of clay brickwork

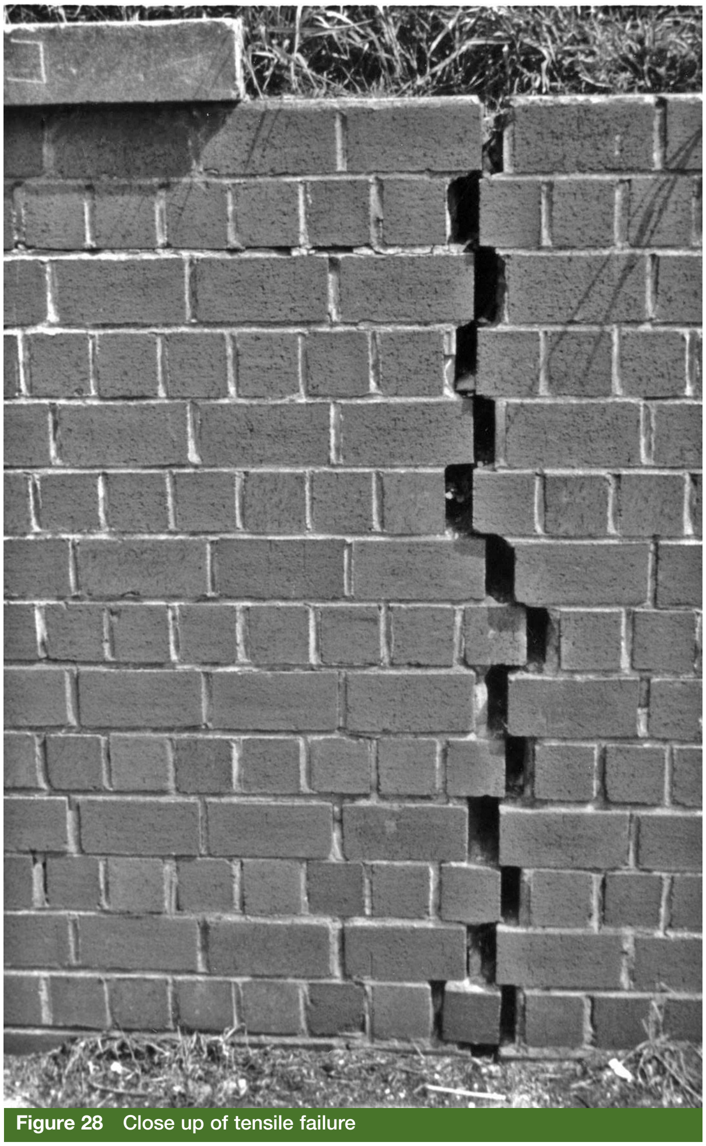


Figure 28 Close up of tensile failure

bed joint reinforcement – even the relatively light woven wire types can be effective. The usual effect is to spread cracks out over a number of joints, so the effects are far less noticeable than the fewer larger cracks which may appear if reinforcement were not to be used.

The main way to avoid cracking due to inherent movement is by the introduction of joints, filled with compressible material, and sealed with water-proofing material such as mastic at intervals along the work. There is inevitably a loss of continuity at such a joint but some can be retained by using a flat steel tie across the movement joint, one end of which is sheathed in plastic and hence continues to allow in-plane movement.

Horizontal movement joints are required, in particular, in high-rise concrete-frame buildings which are clad in brickwork. In time the frame tends to creep and shrink, hence the distance between a floor level and the soffit above can decrease. If the masonry walls are infilling the frame, it is possible that this downward movement coincides with long-term expansion of the brickwork. Consequently, it is possible for the external leaf to be subjected to a load for which it was not designed. One symptom where brick slips have been used to face the floor slab is that they are subjected to vertical load and buckle off the face of the slab (**Figure 29**). The correct design would include a soft joint below the slab, as this is the remedial measure which can accommodate the differential vertical movement.

Figure 29 Brick slips failed at face of concrete floor

Non-masonry factors

Tree roots

Disruption of masonry walls also occurs due to trees, more specifically tree roots. This may be directly or indirectly. In some situations tree roots are directly disrupting masonry walls, often boundary walls. The forces involved can be very great, and relatively thick walls can be seriously cracked, in some cases leading to concerns for the structural safety of the whole wall. Indirect effects are usually associated with a tree which is too close to a building causing a change in the moisture conditions underneath foundations leading to movement and cracking. The movement can be either due to the subsoil shrinking, due to excessive removal of water by the tree, or expanding, possibility following the felling of a tree leading to an increase in available water and ground heave.

The method of avoidance is obvious: do not build close to trees; however, that is of little help in a developing situation. Solutions can involve underpinning or reinforcing foundations with reinforced concrete beams. In the case of direct disruption it may be possible to relieve the effect by the use of a lintel over a root. The key characteristic of problems associated with trees is that they can be progressive.

Wind

Failures due to overloading by wind loads are rare and, besides the gable situation described earlier where they have occurred, it has usually been due to a lack of tying back to the rest of the structure. Nevertheless, our large safety factors against ultimate failure must contribute to this success story. It is also rare to find cracking of panel walls due to wind loading, as this occurs very close to ultimate collapse, so the safety factors guard against cracking and failure to a similar extent.

Soil and hydrostatic pressure

Other lateral loads can cause defects such as the bulging of retaining walls, sometimes to a spectacular extent. It is not unusual to see such a wall apparently supported by a nearby lamp-post. However it is quite common to see such walls where, obviously, some rotation or outward bulging has occurred but for them to be left alone apparently getting no worse over several years. Clearly the effect of movement can relieve the earth pressure load onto a wall and that can be a reason. Similarly, if blocked weep holes or an inadequate drainage layer behind the wall are contributing to the problem, hydrostatic pressure may fall if movement enables a new drainage path. Such movements do constitute a defect in the wall; whether the view is taken that it is so defective as to warrant repair is rather subjective. There is, however, little alternative to partial or total rebuilding of walls in this circumstance.

Impact

Failures caused by vehicle impact often lead to damaged walls which need rebuilding. However, there have been efforts to upgrade impact resistance of, for example, parapet walls by reinforcing them. This can be achieved by drilling vertical holes in them and resin bonding reinforcing bars into the foundation, usually concrete and the masonry above.

Corrosion of ancillary components

Corrosion of metal components in walling is much less common in modern walls than in earlier ones, due to the extensive use of stainless steel. However, during the twentieth century the extensive use of cavity construction together with inadequately galvanised ties, has caused problems. This has largely been of two types. Where light wire, typically butterfly wire ties, was used, a relatively common defect was for the wire to corrode often at the inner face of the outer leaf, sometimes to the point of failure. In many cases this would not be noticed as there may be no visible rust staining and only when some other defect appeared such as bulging did it become clear that an outer lead had lost its restraint from the ties.

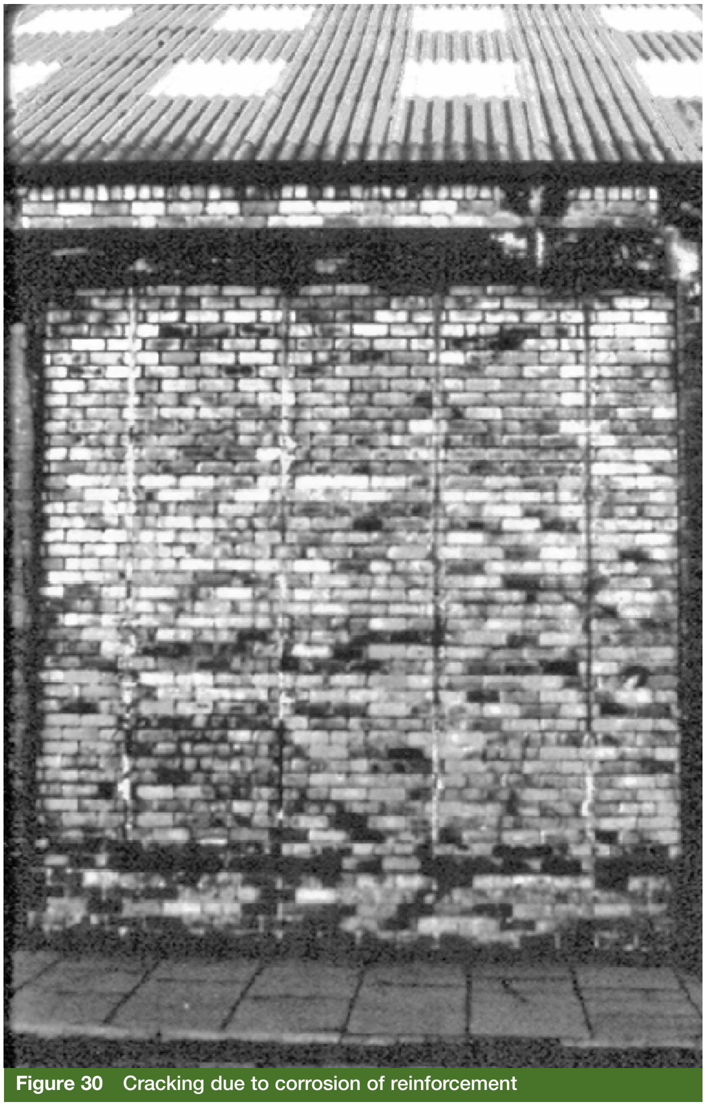


Figure 30 Cracking due to corrosion of reinforcement

Butterfly wire ties are, however, unusual components, as the wire knot could have various degrees of 'tightness' and under lateral load some ties would be effective in transmitting load and others not until any slack was taken out of the knot. Hence it may well be true that many cavity walls with failed wire ties could be functioning properly due to some overdesign and a remainder of working ties.

The second type of defect has occurred with larger-gauge metal ties such as the vertical twist type (**Figure 30**). In this case there is a significant amount of steel available to corrode and, as rust forms, seven times the volume of the original steel from which it came, this can often be too much to be accommodated in a bed joint and cracking has occurred. Even if there is no cracking there can be rust staining, giving a clue as to the problem. In both cases the addition of remedial ties, as those placed in holes, drilled into both leaves from the outside, are used. There is a multitude of designs but they can have, for

Figure 31 Gwyn Hall, Neath, South Wales

Figure 32 Fire damaged structure leaving unsupported walls

example, expanding fixings into the inner leaf and be resin bonded into the outer leaf or resin bonded into both leaves. In the case of the wire tie this may be all that is needed. In the case of a more substantial tie it may be desirable to render it ineffective by cropping it, if it can be accessed, for example if it slopes down to the inside and allows water to track across. However, it will probably always be necessary to isolate it from the masonry in the outer leaf by removing the mortar around it. This can be replaced by foam rubber and the face repointed. Hopefully any further corrosion does not cause local stress on the brickwork.

The Code of Practice for reinforced and prestressed masonry (BS 5628 Part 2: 2005) was introduced in 1985 and, at that time, corrosion protection was a major discussion point. What was a major issue was the degree to which masonry, grout infill and mortar would provide protective 'cover' to mild steel reinforcement. Although quite a lot of research was done into the longevity of reinforced masonry structure worldwide, a lot of the places where there were such structures were in relatively benign environments, e.g. California. There were also odd horror stories and, as a result, a cautious approach was adopted and perhaps restricted the development of the medium; however, the subsequent years have been relatively problem free.

Fire and frost

Two other actions which lead to there being a structural issue are fire and frost damage. The effects of both are usually to cause spalling and a loss of structural section in clay brickwork. However, the massive fire at the Gwyn Hall in Neath, **Figure 31**, did very little damage to the local Pennant sandstone, a very dense and strong material. Where spalling has occurred, either local replacement may be possible or alternatively we may be faced with rebuilding. In the case of fire damage, especially significant damage, there may be a loss of support from internal walls (**Figure 32**), floors or roofs, and hence a serious view needs to be taken about the stability of walls in a temporary situation before decisions can be taken as to reuse.

Explosion

The final action I will consider is explosive loading. In the 1960s, following the collapse at Ronan Point, experimental work on load-bearing brickwork structures (**Figure 33**) showed that in most cases windows were blown out before masonry walls cracked (Astbury *et al.*, 1970). However, where more confined explosions take place damage can be extensive (**Figure 34**). Although the load-bearing brickwork structures did not collapse, a more contained explosion did cause cracking. In most cases, where simple venting does not occur, damage can be extensive and demolition and repair the only option.

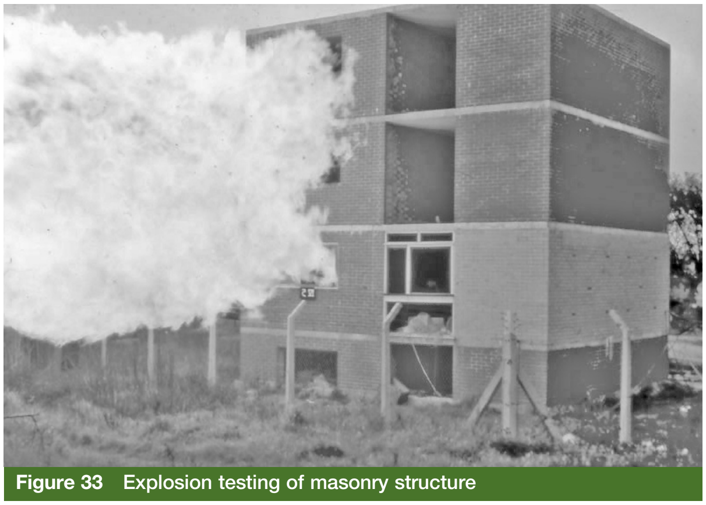


Figure 33 Explosion testing of masonry structure

Figure 34 Extensive damage due to explosion

Remedials

One final point in relation to a masonry structure showing structural defects is that the first consideration must be of the structure, as it is as a structure. The defects, e.g. cracks, loss of tying action, bulges, etc., all require consideration. Is the structure stable as it is? Will stitching across the cracks with light reinforcement return the wall to its previous condition structurally? Will replacement ties work and, if so, how many are needed? All are valid questions. Although such defects do have certain similar characteristics, where repairs are possible most are bespoke and require some original thought and understanding.

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Chapter 36

Sustainability and recycling of masonry materials

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CONTENTS

Sustainability in the construction products sector is generally regarded as a process of manufacturing that takes into account the impacts upon the social and environmental well-being of mankind, locally, nationally and globally. Many of the brick and block manufacturers strive to use resources as efficiently as possible, to reduce waste, reduce the energy used in manufacture, encourage the reuse of materials and to design products with the longest possible life span. One major factor in this goal is the use of alternative, recycled, secondary raw materials, as an alternative to virgin primary raw materials, thus preserving the resources while reducing waste to landfill or low-value applications. For this to happen it is important to understand the selection and assessment process in the selection of these alternative materials, in order to achieve product quality and performance. However, sustainability should also take into account the 'in use' phase of the life-cycle analysis of a product. With many brick and block products this can be in excess of 100 years. Therefore a manufacturing process with a high initial 'impact' of either raw materials usage, energy, emissions, etc. can be reduced considerably as a result of that product having a very long and functional service life. It is for this reason that sustainability and impact analyses should be viewed over the whole life of a product, 'cradle to grave', rather than just at the point of manufacture, 'cradle to gate'.

What is 'sustainability' in manufacturing bricks and blocks?

The last 15 years has seen a step change in the acceptance of 'alternative' raw materials in the manufacturing process for masonry construction units. This has had a number of effects including reduction in the use of primary raw materials (virgin raw materials) and substitution.

Sustainability is now a term that is commonly used, however it is often regarded as a term that has a very wide definition, therefore it can 'mean all things to all men' depending on how the term is used.

In the context of a general understanding of what 'sustainability' means, the United Nations' Brundtland Commission Report in 1987 (UN, 1987) defined sustainable development as:

> *Meeting the needs of the present without compromising the ability for future generations to meet their own needs.*

Many of the construction-sector manufacturing associations have looked at what sustainability means to them and in general come to the same conclusion. By way of example, the Brick Development Association, the trade association of clay and calcium silicate bricks in the UK, defined sustainability as:

> *Sustainability is about the responsible use of resources. Our task, therefore, is to use our resources as efficiently as possible – to reduce waste, reduce the energy used in manufacture, encourage the re-use of materials and to design products with the longest possible life span.*
>
> (BDA, 2001)

The emphasis is therefore in the use of virgin raw materials only when alternative, recycled, materials are no longer available or detrimental to the performance characteristics of the end product. This has resulted in a wide range of recycled material uptake into brick and block manufacturing. It has also brought to the fore, issues with some of the national and international regulation and legislation that, on the one hand, promotes the use of waste and recycled materials and, on the other, inflicts such a burden of bureaucracy that the cost of management of an alternative raw material overshadows the benefits, and therefore becomes unattractive.

The background and case for the use of recycled materials in masonry brick and block manufacturing

Since the mid-1990s there have been major step changes in the business drivers for UK brick and block industries, not least the introduction of national, European and global environmental regulations and legislation, that not only affect the actual manufacturing process and pollutant emissions but also waste generation and disposal, and, critically to the energy-intensive UK brick industry, the supply of energy and the production of greenhouse gases.

What is a recycled material?

It is first important to understand actually what is meant by recycled materials, and therefore at least have a firm foundation upon which manufacturers can make claims. BS EN ISO 14021: 2001 *Environmental labels and declarations. Self-declared environmental claims (Type II environmental labelling)* defines recycled materials as:

> *Material that has been reprocessed from recovered material by means of a manufacturing process and made into a final product or into a component for incorporation into a product.*
>
> *Such material includes pre and post consumer waste, but excludes scrap materials that are generated and re-utilised in the same production process.*

It is clear that, based on this widely used definition, recycled materials specifically exclude any materials created at source and reused in the process. On this basis, production, waste, offcuts and crushed fired waste brick (grog) cannot be claimed to be recycled materials, although clearly it can be claimed that this is good housekeeping. Similarly, the recycling of scrubber residues, from brick kiln stacks, back into the manufacturing process, would not be viewed as recycling on the basis of this definition.

Therefore *recycled materials* have been limited to those materials derived from outside the manufacturing process, and *recovered materials* being those materials that are recovered and reused from the actual manufacturing process.

Why use recycled materials in brick and block manufacturing?

In simple terms, the brick/block-making process is a very straightforward manufacturing process which relies on the properties of the raw materials to achieve the physical properties of the product following a process of drying and firing, for clay bricks, and curing for concrete blocks, and the autoclave process for autoclaved aerated concrete (AAC).

Historically, winning clay, typically adjacent to the site of the brickworks or aggregates from next to the quarry, has been the easiest and cheapest way of keeping the bulk raw material costs to a minimum. Generally clay and aggregate deposits associated with a works were very well understood by the works manager, and the products that could be produced were limited to those that could be achieved with the clay or aggregates available on site.

Specifically within the brick sector, the last 30 years have seen a fundamental step change in this approach to brickmaking. It is not uncommon now for a works to have a portfolio of up to 30 brick types that it may be requested to produce. Pressures from customer expectations, or at least those perceived by the companies sales force, current fads, optimisation of production capacity to meet projected orders and minimisation of transport costs, have resulted in works being very adaptive and flexible.

While many aesthetic appearances of bricks can be altered by surface decoration, sand and stains, engobes, roller patterns, the properties of the body tend to remain the same. Changing the body composition is not easy, and often results in a number of compromises, and often increased raw materials and manufacturing costs. A number of UK brickworks, who's own clays produce red bodies, currently produce buff products and vice versa. No longer does the indigenous raw material dictate the body composition, typically clays and other mineral additives such a chalk and limestone are added into the blend to create a new body composition.

Likewise for the concrete block sector, AAC and aggregate concrete blocks are typically manufactured on a regional basis to avoid the high cost of transportation and distribution, so there is a need to match physical performance characteristics based on materials from different sources, aggregate and cement. Therefore it is not uncommon to have the same 'named' product having a very different appearance to the same product manufactured in a different region. As aesthetics are less of an issue with blocks, as they very rarely are visible in the final building, this is less of an issue than it is with bricks.

Recycled materials can therefore play a major role as body additives in brick and block making. If the materials are fully characterised, and the application targeted carefully, there is no reason why recycled materials should be viewed any differently from virgin raw materials. Success is therefore intrinsically linked to the identification of the targeted application or applications.

The key to successful use of recycled materials in a manufacturing process is therefore reliant upon the identification of the reason or reasons why the material is being used over virgin raw materials. These can be broken down into five main categories (Smith, 2001):

1. financial
2. environmental
3. strategic
4. technical
5. marketing/procurement.

Financial

Financially, one possible reason for using a recycled material as a secondary raw material could be that the material producer would be willing to pay for the user to use the material. If the cost of delivery and royalty to a consumer is less than the disposal, to either landfill or other outlets, then both the producer and the user benefit financially.

Environmental

In terms of an environmental benefit, one of the key driving forces behind the development of new clay or concrete/aircrete body design is related to emissions to atmosphere. Inert or active fillers, some of which can be sourced through recycling routes, can help reduce the emissions to air. Inert fillers act by dilution of the primary raw materials, often the main source for the emissions during the manufacturing process. By substituting primary raw materials for inert recycled materials, the emissions can be reduced. Active fillers, or more accurately body modifiers, act by reacting with the primary raw materials and forming new phases that contain elements and compounds that would normally be emitted to air.

Hand in hand with impacts upon the environment are those based on the emissions to air from the combustion process, clay brick and cement manufacturing. Both have a high energy input through combustion processes, therefore substitution of raw materials that reduce the energy consumption are beneficial, not only in terms of cost (fuel cost being high) and also in terms of emissions to air from the process.

Fluxing additives in the brick manufacturing process can help reduce the top firing temperature, thus saving energy per tonne of product. Within the cement-based block industries, the use of waste to fuel the cement manufacturing process (Cemfuels), along with pozzolanic non-cement binders as replacements, pfa (pulverised fuel ash) and ggbfs (ground granulated blastfurnace slag), in the production of the blocks themselves, results in a reduction of emissions SO_x, NO_x and CO_2 and associated costs, while using a recycled or recovered material from another process waste stream.

Strategic

The use of a waste material for strategic reasons would primarily be to conserve raw materials. Again, the process would essentially be the use of inert fillers rather than primary clay or aggregates for the production of bricks and blocks. Depending on the unit cost of extraction of the primary clay and aggregate, this may also offer cheaper raw materials.

As with most ‘external’ factors that affect the manufacture of construction products, products that require the extraction of materials from out of the ground are, naturally, seen as having a high impact. The ‘planning’ process in the UK is, at least in design, there to protect the interests of both industry and local residents, as well as strategically the needs of the ‘nation’. Planning, by its nature, is not a rapid process, and it is not uncommon for the planning process for the development of a new quarry, for either aggregate or clay, to take in excess of 7–10 years. The potential benefit of extending the life of any operational extractive site, therefore can be significant, not only in a secure source of raw materials of the desired quality but also in ‘buying time’ in securing the next source. Increasing the life of a quarry by just one year, can have a significant impact upon its value to a manufacturing process. The benefits are of course material-specific. In general, sand and gravel quarries have a significantly shorter lifespan than hard-rock quarries or clay pits.

In addition, within the ‘strategic’ aspects of the evaluation process, the longevity of supply of a recycled material must be taken into account. Often such products are the ‘wastes’ from other processes and as such are dependent upon those processes not changing fundamentally. If they do, the material may be no longer available in the volume or at the cost/revenue conditions at the current time. There is therefore a significant risk that, once common use for a recycled product becomes established, the producer of this product may see it as a legitimate revenue-generating stream, therefore no longer a problem waste but a valued co-product. It is at this stage that the ‘waste’ becomes a valuable commodity and, if a manufacturing process become dependent upon it, monopoly supply issues may develop.

Technical

Clay body modification, by use of recycled materials, for technical reasons is aimed at producing either a higher specification product, or a different aesthetic quality, colour or texture. To produce a technical benefit the addition of a recycled material typically results in a change in the level of vitrification, strength, structure pore distribution, colour, and/or a reduction in soluble salts, and kiln scumming.

Some recycled materials are able to address more than one of these key functions, and therefore are potentially more valuable than others. For example, recycled glass in powder form could be added to a brick body for a number of reasons:

- to reduce firing energy and/or faster push rates; this would be a direct financial saving;
- could also result in reduced pollutant (HF) and greenhouse gas (CO_2) emissions, therefore of environmental benefit;
- the addition could save virgin raw material, therefore extend the life of a quarry and be strategic;
- improve the physical properties of the body, typically freeze–thaw durability water absorption or strength.

It should be stressed, though, that the reactivity, and thus the overall benefit for glass and other recycled materials, is both clay type and manufacturing process specific, and not a 'one fits all' solution.

Recycled material use in block manufacturing also benefits technically from the use of some recycled materials. Some of the insulation blocks use recycled pfa, which not only acts as a cement replacement in the binder but the hollow spheres also aid in the formation of the voids, which enhance the thermal insulation properties of the blocks. As an example, 'Thermalite'®, manufactured by Hanson, uses up to 80% pfa in the manufacture of the blocks, thus significantly reducing the need for virgin raw materials and also producing a lighter weight, and technically more beneficial product.

Marketing/procurement

A factor that has come to the fore in recent years is the 'pull' from the end-users of the construction products being manufactured. The concept of 'sustainable construction' now requires 'sustainable construction materials', therefore manufacturers are often caught between the business drivers requiring manufacturing process efficiency, technical characteristics and cost, versus the need to meet expectations of the customers buying the products, or the 'stakeholders' and shareholders of the company.

In pure marketing terms, the use of alternative raw materials derived from recycled and waste streams can be a new way of marketing a familiar product. Brick and blocks are not the most advanced construction product available in the marketplace; they have a long history of use and most end-users are aware of the technical performance characteristics. In short '*they have used them before and know what to expect*'. By changing the manufacturing process and incorporating alternative raw materials into the product, this may have benefits in the four other 'reasons for use'. Technical benefits can be sold along with the environmental, effectively making the products 'greener' than the original.

The 'climate of change'

The internal and external factors that ultimately have an effect upon the brick industry can be viewed simply as 'push' and 'pull' factors. 'Push' factors are those that traditionally have affected the manufacturing process, and include the legislative framework under which manufacturing is permitted, while the 'pull' factors are those that are imposed upon the industry by the end-user, either customers or architects/designers/specifiers.

Push factors

In very simple terms, the push factors are those that are linked to the actual manufacturing process, and therefore include such issues as raw materials, product manufacturing process, product specification and quality, and more recently the legislative framework under which manufacturers are bound.

Both recycled and recovered materials have traditionally been used to address material issues, typically problems with raw materials, the manufacturing process or product appearance and/or quality. In the case of clay bricks, these materials are often used to modify the clay body, and have either been specifically targeted, in order to modify a particular physical property, e.g. drying stability, plasticity, water absorption, durability, or have been added for general aesthetic characteristics, as with Town Ash, used to give the 'London Stocks' appearance.

Within block production, costs and technical performance have driven the use of recycled materials. Alternative, cheaper cement alternatives, used as binders, such as pfa and ggbfs as pozzolanic cement replacements, as well as recovered aggregates either from other mineral processes, such as china clay waste (sands) in the south-west of England, or from construction and demolition waste (C&D) waste.

More recently, recycled materials have been used to address specific legislative issues, in particular environmental issues. The addition of body fuel in clay brick manufacturing, while extensively used as a colourant, also provides the benefit of embodied energy, and thus reduced direct energy costs. Likewise the addition of pfa, slag and incinerated wastes has been used as both diluters and modifiers, resulting in the reduction of the primary pollutant source (the raw materials) and also the potential to reduce the firing temperature or speed of firing to reduce the firing energy.

While by their nature, an evolving situation, it is worth identifying the more recent legislation that has or will affect the brick and block manufacturing industry, and the use of recycled and recovered materials in the manufacturing process.

General environmental legislation:

IPPC	Integrated Pollution Prevention and Control
PPC	Pollution Prevention and Control
CCA	Climate Change Agreement
EUETS	European Union Emissions Trading Scheme

Waste and recycled materials legislation:

Waste Management Directive
Landfill Directive
Mining Waste Directive
Hazardous Waste Directive (HWD)
Hazardous Substances Directive (RoHS)
Waste Electrical and Electronic Equipment Directive (WEEE)
REACH

Construction specific regulations and directives:

Construction Product Directive
Construction Products Regulation (in development)
Regulated Dangerous Substances (aka Dangerous Substances)

It is unfortunate that no one directive or regulation covers all the aspects of the use of recycled (and waste) materials in manufacturing processes, therefore it is important to bear in mind that compliance with one does not equate to compliance with all.

With the implementation of the directives and regulations specific to the handling and disposal of 'wastes' during 2004/5, and specifically the classification of wastes for the purpose of suitable landfill sites, greater burden has been placed on the identification and general nature of the waste, especially hazardous wastes, irrespective of whether or not it can be recycled.

The Environment Agency (EA), who polices the waste and environmental legislations in the UK, has taken a very 'literal' approach to the new regulations. This has resulted in a significant amount of confusion and at times conflict in advice from the Department for Environment, Fisheries and Rural Affairs (DEFRA) and the Department of Trade and Industry (DTI). While one government department is promoting recovery and recycling of materials from the waste stream, another is effectively stopping this by increasing the burden of administration for any one wanting to use the recycled materials.

By far the most contentious debate currently active is 'when is a waste not a waste?' Members of the DTI's Environmental Innovation Advisory Group (EIAG) have recently highlighted this point, and the following statement published in the minutes of the April 2005 meeting:

> *EIAG Members stressed the difficulty of persuading purchasers to change from a 'virgin material' to a recycled material if the definition of waste was ambiguous or unclear or the recycled material carried added regulatory burdens. Important to be clear about the point at which recycled material ceased to be waste. The priority should be to prevent materials from being defined as waste in the first place. The risk to business (of regulatory implications) was 'absolutely deadening'.*
>
> (DTI, 2005)

What has been clear, up to July 2005, is that companies wishing to use recycled materials, derived from waste streams, were finding that in order to use such materials the transportation, storage and use of the materials was governed under the Waste Management Regulations, and in the majority of cases resulted in the need for both waste transportation documentation and waste management licences for the factories. In some cases very loose (probably not legally robust) understandings with local EA officials were in place; since this date the new Hazardous Waste Directive, in conjunction with the Landfill Directive that came into force in 2001, has changed the method and ultimately the diversity of materials that are now classed as hazardous.

Since the publication of the European Waste Catalogue (EWC), a number of materials that were not previously classified as hazardous, now are, e.g. CRT, fluorescent tubes and gypsiferous waste. It is clear that the disposal of waste is driving the legislation and, even though the waste is being recovered and recycled into another process, in the eyes of the EA it remains a waste until it is converted into something else by a process. On this basis, a waste added to a brick body remains a waste until the brick is fired and comes out of the kiln. While being delivered to the factory, and through the forming process, the material is still technically a waste, and thus Waste Management Licences (WML) for the factories are required.

It is widely agreed that a more 'common sense' approach is required and fundamentally, as stated by the EIAG, the key is not designating a material as a waste in the first place.

Pull factors

Within the last 10 years, pressure from the end-users of bricks and blocks, for manufacturers to declare a recycled material content, has been increasing. Bodies promoting recycling, such as WRAP (Waste and Resources Action Programme) and Building Research Establishment (BRE), within construction products, have resulted in a number of initiatives to establish a baseline against which all construction products can be measured and compared. The *Green Guide to Specification*, *Code for Sustainable Homes* and the *Green Guide to Housing Specification* have all been published by BRE as part of the BRE Environmental Assessment Method (BREEAM) programme. WRAP has recently initiated a targeted programme on procurement of manufactured materials, including largely construction and fittings for domestic and commercial buildings. The emphasis is on getting builders, architects, designers and specifiers to write into the procurement requirements a need for specific recycled materials content, and be able to demonstrate 'responsible sourcing'.

It is these initiatives that have been identified as 'pull factors'; in simple terms, making it a requirement of supply that bricks and blocks for certain 'green' projects must have a recycled materials content. If successful, it is not intangible that this may develop into a general construction materials requirement, especially for construction in the public sector, and could be a key requirement for financing large infrastructure construction projects through the 'ethical and environmentally aware' financial institutions on the stock exchange.

The commercial implications of not having a quoted recycled materials content could in the future be a significant factor. The risk, though, of declaring recycled material content, and more specifically identifying what the material is (or was), may place manufacturers 'heads above the parapet' regarding the status of works, waste management licensing, and the potential view that 'dangerous or hazardous wastes' are being stored and used in the brickworks, generating suspicion and concern with near neighbours.

Current recycled materials usage in the UK brick manufacturing sector

Recycled material content, or by way of an example specifically within the UK brick manufacturing sector, 'MARSS' (Materials from Alternative, Recycled and Secondary Sources) materials has rapidly become a metric that is declared as part of the sector's yearly key performance indicators (KPIs). At a manufacturer level, and even in some cases down to a product level, there is a growing tendency to declare the recycled content of a brick or block. However, there are a number of different 'protocols' as to what and how this value can be declared. WRAP, the UK government's 'waste watchdog', has published a guide to the declaration of recycled content for construction products in general. The *Rules of Thumb Guide* (WRAP, 2008) became a recognised 'system' to use in the construction product manufacturing sector; however, it only addressed parts of the 'sustainability' issues. Other 'systems' such as MARSS, generated for the brick manufacturing sector in the UK, cover a wider, and more 'holistic' approach to recycled material content.

The way that recycled content is declared is also a variable. Historically the percentage recycled material content was calculated on a weight-for-weight basis; however, a number of recycled material comparators now use 'value'. This is as a result of many recycled materials being used in high-volume, but low-value applications, e.g. crushed concrete as sub-base, rather than it being used as a high-value alternative material as a direct aggregate replacement. The trend therefore has started to change, and the value of a recycled material becomes important, especially when procurement factors are taken into account.

Within brick manufacturing, while a number of materials have been trialled, used or currently in use, those that have stood the test of time and have made it through the trials process have, on the whole, been used for physical property enhancement or modification, and are therefore governed by the 'push factors'. The same is also generally true for the block manufacturing sector.

Recent emphasis on the cost of energy and greenhouse and pollutant emissions, has meant that, since 2002, trials have concentrated on materials that will reduce manufacturing impacts, raw material costs in the block sector and both firing temperature and or emissions in the brick.

Factors that appear to be critical in whether or not a recycled material is used are as follows:

- technical performance
- cost
- availability (volume, transportation).

In the future it is possible that additional factors will become applicable, namely:

- energy price
- permitted emission levels
- end-user requirement – customer expectations and supply chain requirements
- waste management/hazardous waste legislation, regulation and administration.

Conclusions

The use of recovered and recycled materials in brick and block manufacturing appears to be common sense; they can enhance physical properties, reduce virgin raw material use, reduce curing, firing time and temperature, and reduce emissions. Currently, use is based on the need to address the 'push factors', those of manufacturing requirements, and with ever-increasing energy prices and stricter emission limits, such materials can play a major role in achieving a cost-effective solution.

On a technical level, the use of alternative materials needs to fundamentally meet the 'fitness for purpose' rule. Ultimately this requires comprehensive development and testing of the products to national and international standards. Many developments fall at one of the many hurdles placed in the path of product development. Products can meet all the strength and durability requirements but fail the 'reaction to fire', purely on the basis that the alternative raw material is combustible. Further treatment to counteract this may be prohibitive in cost terms, therefore the product never reaches the marketplace.

Changes in procurement and specification requirements, especially in the public and areas of the private sector of construction, are moving towards a recycled content requirement, and as such products without this may not be eligible for certain projects.

There are still a number of critical issues to be resolved if widespread use of recycled materials is to become commonplace in the brick and block manufacturing sector, not least the waste management and hazardous waste legislation. Ideally, recycled materials should never be identified as 'wastes' and therefore the legislation targeted at waste will not be applicable but, until such a time is reached, manufacturers will continue to find confusion and barriers to use on the basis of current waste management practice.

Sustainability and life-cycle analysis

Before continuing, it is worthwhile considering the sustainability of masonry in terms of the two recognised time periods: cradle to gate (i.e. the manufacturing period) and cradle to grave (i.e. the whole life of the material). There is still discussion about which time period is most important and how much the cradle-to-gate period influences the assessment of the performance of the masonry from cradle to grave.

Why should masonry have to be so defensive in terms of its sustainability credentials? If we look at masonry in terms of cradle to grave, we are looking at a construction material that, by and large, and when given the chance, will last for over 100 years with very little maintenance and in doing so will outperform the majority of the current construction materials used today. We say, 'when given the chance' because it is reasonable to expect that, within that 100 plus years, the masonry structure may be required to be demolished. Should this occur, it introduces a second major benefit of masonry which supports its sustainability credentials when considered from cradle to grave. Masonry can now be recovered reasonably easily, and recycled either back into other materials or potentially back into the manufacture of further masonry units, hence establishing a truly closed-loop system. Third, there is ongoing discussion on whether the best approach is to demolish existing structures and re-build in order to accommodate a required change in use of the building or to extend the life of the existing building by modifying to suit the new requirements.

There are plenty of examples of buildings constructed from masonry, in fact much more than any other construction material, where a change of use has been possible without demolishing (i.e. conversion of old industrial buildings/warehouses to residential/office accommodation) or where refurbishment has been possible because the existing building has been constructed from masonry rather than another material. In the case for the sustainability of masonry over its whole life, these are significant factors. However, do they balance the negative factors evident during the manufacture of some masonry types? And is it acceptable to try to assess the sustainability of masonry by comparing its performance during the cradle-to-grave and cradle-to-gate periods? The fact is that when considered from cradle to gate, there are many ways in which its manufacture could and should be improved. So far we have provided a flavour of how the industry is achieving/attempting to do this. The following introduces another example of how the industry is attempting to improve the sustainability of masonry.

Alternative binders

As discussed so far, the majority of the improvements to the sustainability of masonry units have been achieved through the replacement of traditionally extracted natural aggregates with recycled aggregates or the partial replacement of the cement binder content. However, the fact is that many of the units under discussion here still use either cement or clay as the binder to stabilise the aggregate matrix.

Clay is an extracted natural material which is a finite resource. Couple this with the implications to the environment of the actual extraction and transport of the material and it is easy to see why government targets in response to EU directives actually demote this type of practice. Cement, regardless of the admirable attempts that the industry makes to improve the sustainability of the material, is still a major contributor to global CO_2 emissions (Hammond and Jones, 2008) and is hence perceived by the majority of the public, whether rightly or wrongly, as a major factor responsible for global warming.

Research is currently being performed at Leeds University into developing a construction unit which replaces the need for cementitious or clay-based binders and instead uses a waste material as the binder. An additional and significant benefit is that these new waste binders allow the incorporation of 100% recycled aggregates. This makes valuable use of several resources, which might otherwise find themselves being put to landfill – an option that may not be available before the end of the next decade.

Two waste binders have been researched. Initially, waste bitumen was considered (Forth *et al.*, 2006) (**Figure 1a**). This material was found to be successful in encapsulating several recycled aggregates incorporated either individually or in combination with up to two of the other materials from the following typical list: incinerated bottom ash (IBA), incinerated sewage sludge ash (ISSA), pulverised fuel ash (PFA), furnace bottom ash (FBA), glass, steel slag, rice husk ash and soil. The technology was developed sufficiently, such that the product (block- or brick-sized unit) could easily be developed for the market. The properties of the unit are at least equivalent to current aggregate concrete units and many clay bricks (<40 MPa). However, this was seen as a future development for the UK (in other developing or transitional parts of the world, its success encapsulating soil may enhance this technology's introduction). As the research developed on this bitumen-bound unit, priority shifted as the potential of another waste binder, waste cooking oil (WCO), was identified (Forth and Zoorob, 2006). Waste cooking oil has proven to be at least as successful as the bitumen-bound unit, however it is considered more sustainable.

The manufacturing of a waste vegetable oil-bound unit can be briefly described as mixing well-graded aggregates with WCO at room temperatures. The oil-coated aggregate is then transferred to a mould and compacted. The compacted sample is then demoulded and cured in an oven for no longer than 24 h. During curing, a complex process of oxidation and cross-linking irreversibly polymerises the oil, removing the viscous nature of the oil and producing an aggregate matrix with elastic-brittle properties. The process works by encapsulation with only minimal chemical reactions present and is quite different to the 'interactive' process present between the cement- or clay-bound binders in current traditional units. Hence, this WCO-bound unit can incorporate 100% waste materials, which is a level of incorporation not always possible in the traditionally bound materials.


(a)

(b)

Figure 1 Long-term stability of masonry: (a) waste bitumen bound units; (b) WCO-bound units

Results from samples tested within the laboratory show that compressive strengths of up to 36 MPa can be achieved. The bulk density for the cured samples ranges between 1.56 and 1.965 g/cm^3 (although theses values can be reduced depending on the aggregates incorporated and the proportions of the aggregates in the mix). Typical 24-hour water absorption test results were between 4% and 11.5% and the initial rate of suction was well below 1.5 kg/m^2/min. The long-term stability of the units was also monitored. The specific creep of the unit was less than 200 microstrain, however the unit showed no evidence of shrinkage or expansion (although there was an initial expansion, similar to the expansion of kiln fresh clay units, but this finished within 14 days of being manufactured). **Figure 1b** illustrates laboratory samples of WCO-bound units constructed as masonry and monitored for long-term stability.

Indicative tests of masonry constructed from WCO-bound units are also being performed. **Figure 2** shows a wall that has been constructed at the Poundbury Ecotown, Dorset. Indicative fire tests have also been successfully performed (**Figure 3**) (Forth and Heaton, 2009). Freeze–thaw tests performed at CERAM Building Technology have suggested that the unit has the potential to be classified as F1, suitable for ‘moderate exposure’.

A WCO-bound construction unit is expected to be on the market within the next 12 to 18 months, by the middle of 2010 (**Figure 4** illustrates a WCO-bound block unit). It is


Figure 2 Test panel at Poundbury, Dorset

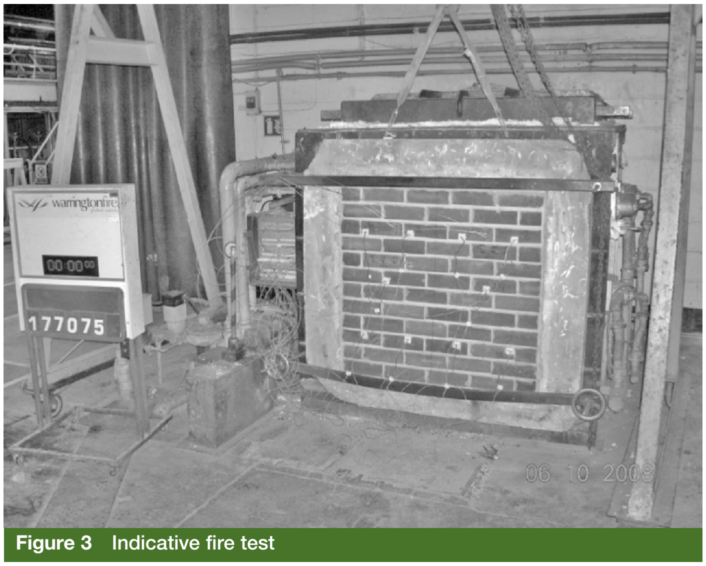

Figure 3 Indicative fire test

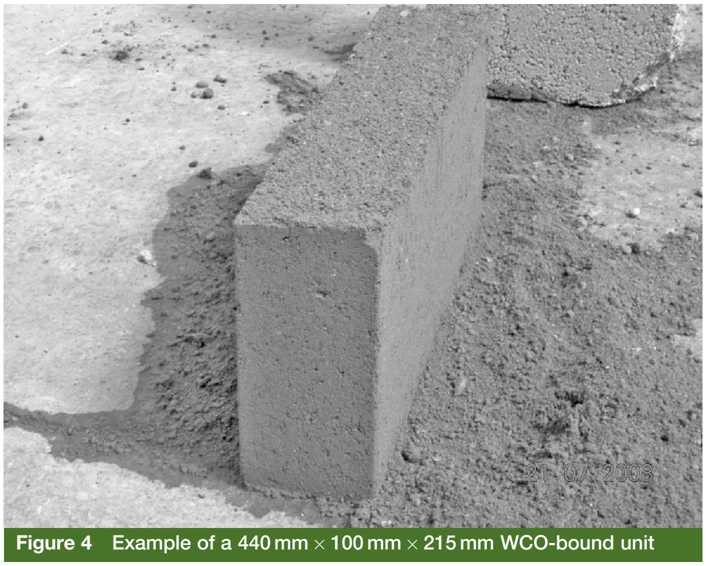

Figure 4 Example of a 440 mm × 100 mm × 215 mm WCO-bound unit

not intended to replace existing units, as clearly the quantity of WCO available for recovery would not permit this. It is expected to satisfy a niche market, i.e. one that considers sustainability as a major influence of choice. Commercial strategies dictate that manufacture and distribution are done regionally, perhaps by county to minimise collection and delivery radii. Co-location of manufacturing plant with existing or planned incinerators is also a requirement – the additional benefit here is that heat from the incinerators can easily be diverted to cure the units. These strategies have also factored in the current various demands for WCO and therefore only assume access to 40 to 50% of the oil recovered at the moment (2009).

Basic scientific research into other alternative binders is still continuing and, encouragingly, another waste material binder has been identified which is showing signs of being just as successful. This waste material is also available in much greater quantities. It does therefore seem possible that the sustainability of masonry as a whole when assessed from cradle to gate can be further improved.

Glass bricks

One final example of a product that has been developed as a result of the need to utilise waste streams and convert them into sources of ‘valued’ raw materials is that of the ‘glass brick’.

At first glance these bricks and clay paver units look almost identical to clay-based products, however these are products that are manufactured by a patented process and use 97% recycled container or CRT (cathode ray tube) glass, mixed with a binder. This project was developed from the need to utilise in a better way the ‘glass mountain’ being generated in the UK, as a result of a surplus of mixed- and green-coloured container glass being recycled, bottle banks and commercial collection services but with no reuse option available. At times during the 2000s the UK had in excess of 1.4 million tonnes of this type of glass available, much of it ending up in landfill.

WRAP, as part of the development drive for material recycling funded work at Staffordshire University, to develop a construction product manufactured from almost entirely recycled glass. The project successfully developed a product that not only looked identical to an equivalent clay product but also possessed similar if not better technical properties, and the fusion temperature and thus energy input two-thirds of that for conventional clay bricks. Commercialisation of this process by Geofusion Ltd, who has acquired the rights to this product, is pending as of March 2009, but has the potential to utilise a vast proportion of recycled container and CRT glass in the UK.

Recovery and reuse

The final stage of sustainability, after a given building has been earmarked for demolition, is the recovery and reuse of the units. As this is a goal, careful consideration of the value and cost of recovery and reuse need to be factored into the demolition process.

Taking clay bricks as an example, recovery of old bricks and the reuse of them as bricks requires the demolition process to allow the breakdown of the masonry in a controlled and defined manner. The value of the recovered bricks can in some cases be 2 to 4 times that of a ‘new’ factory fresh brick. It is not uncommon for ‘salvaged’ bricks to fetch £1000 per 1000 bricks, against £200 per 1000 for new bricks.

Recovery is very much dependent upon the ability to ‘easily’ recover the bricks from the masonry, straightforward if the mortar used is lime based, much more difficult if cement based.

For many, the incurred costs far outweigh the potential value and therefore the use of the ‘wrecking ball’ generates a general construction and demolition waste that can be used as a general low-grade aggregate.

The same is true for blockwork: rarely is there any incentive to recover concrete blocks or AAC from a demolition site. However, careful demolition and the avoidance of cross-mixing certain materials with others can maintain the value of the ‘rubble’, and increase the options available for recycling.

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Chapter 37

Movement in masonry

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CONTENTS

Dimensional changes in unreinforced masonry structures result from external influences such as settlement and from internal physical processes, which relate to the inherent properties and behaviour of the unit and mortar components of the masonry. This chapter concentrates on these inherent properties and discusses the four main causes of movement: changes in moisture content; reaction to load application; temperature changes; and chemical actions in relation to clay, concrete, calcium silicate, natural stone and reconstructed stone units. Where pertinent, reference is made to Eurocode 6 – Design of Masonry Structures. While each of these movements can occur in isolation, the chapter emphasises the fact that two or more of these movement types will at some stage in the life of the structure act cumulatively. It is when cumulative behaviour occurs that the performance of the structure is more likely to be affected and that movement failure, i.e. cracking, may occur. The effect of cracking on the durability of masonry structures is considered. However, it is also suggested that the onset of cracking may also be useful as it can indicate the potential presence of structural problems from external factors.

Overview

Masonry is affected by a number of physical processes which cause small dimensional changes in the finished construction. The extent of these movements depends on the materials, the environment and the construction methods. Any dimensional change will affect the performance of a structure and it is therefore necessary to estimate these movements at the design stage and where possible restrict them to acceptable levels. There is a wealth of data available from the manufacturers, representative trade organisations such as the Brick Development Organisation or the Concrete Block Association, and from various research institutes, which will allow the designer to estimate movements due to the properties of the materials; some of this data also considers the effect of the environment. However, the ability to assess the influence of the construction method and more importantly the dimensions and orientation of the required structure can only really be developed by the designer with experience gained from previous masonry design. With the decline in masonry design, beyond conventional housing construction, this is a major problem as fewer designers are able to gain this experience and those with the knowledge are not being given the opportunity to pass their experiences on. As the lack of understanding of (a) movement in practice and (b) the limits to the extent to which movements can be controlled increases, it has led in many cases to the overuse of movement joints. Movement joints are successful in accommodating movement but with their recent excessive and often blind incorporation within masonry construction it has led to masonry being used less structurally. In many cases, the number of joints proposed is uneconomical and can lead to problems with overall stability. The other extreme is one where, due to the designer's lack of experience (and also the desire to satisfy the architect's requirements), movement joints are spaced/located incorrectly or are almost entirely omitted, resulting in excessive short-term movement failure (i.e. cracking). This chapter discusses the causes of movement, each of which is relative to both the vertical and horizontal direction, in unreinforced masonry. Where pertinent, it introduces the recommendations of the current design codes, BS EN 1996-1-1: 2005, BS EN 1996-2: 2006 and references the extensive and very detailed section on this topic in BS 5628-3: 2005. Beyond recognising that movement joints are a valid approach to dealing with movement, the joints are not discussed any further in this chapter. Designing and detailing for movement is discussed in Chapter 33. Maintaining experienced designers within the industry is therefore important for the effective design of masonry buildings and to ensure the potential for masonry to be used innovatively.

Causes of masonry movement

In no particular order, there are four main causes to dimensional changes in masonry structures:

1. changes in moisture content
2. application of loads
3. temperature changes
4. chemical actions.

All of these factors relate to the inherent properties and behaviour of the masonry, which it is important to recognise is actually a composite composed of at least two elements, namely the unit and mortar components. These causes are considered distinct from external influences such as differential settlement or ground movement from tree root growth for instance, the deflection of any supporting/supported structure (i.e. roof) or seismic/dynamic movements, which are not considered in this chapter.

It will be seen from the following sections that the four causes of dimensional change listed above are in many cases a combination of reversible and irreversible movements and that the causes are also very often cumulative. It will also become apparent that it is not the movement that causes problems for the designer but the restraint that exists within the structure to these four types of movement.

Changes in moisture content

As mentioned above, masonry is a composite composed of at least two elements; the unit and the mortar, and it is quite possible that while the unit may exhibit moisture expansion, the mortar may contract. The overall movement of the masonry is therefore the result of a combination of the individual movements of the elements.

The majority of construction materials can absorb or lose moisture; as such their moisture content will change with time, depending on the environmental conditions. When moisture is absorbed or lost at ambient temperatures, the dimensional change is termed a reversible movement. This type of movement is considered small (generally less than 0.02% or 200 microstrain) and with this type of movement both the unit and the mortar element will generally either expand or contract simultaneously depending on whether the masonry is subjected to wetting or drying, respectively. (The degree of movement may be different for each element depending on the pore structure of the two elements, the stiffnesses of each element and the restraint that exists due to their relative stiffness.) The fact that this type of movement is termed reversible movement indicates that it is not a permanent movement. However, masonry can exhibit a permanent or irreversible movement.

Irreversible moisture movement is a permanent dimensional change that occurs with time. Of the more common masonry unit types mentioned in this chapter, only clay units exhibit a long-term irreversible moisture expansion, the level of which depends on the type of clay and the degree of firing. This long-term expansion is a result of the adsorption of water by the brick unit. Adsorption is the chemical bonding of moisture to the clay material; this chemically bound water can only be removed by heating the brick to temperatures in excess of 600°C, a process which by removing this chemically bound water will also remove the irreversible expansion. (Absorption, as mentioned previously as the cause of reversible expansion, refers to water which enters the pores of the material causing expansion much like the process observed in a sponge. The water absorbed is normally termed ‘free’ water.) The irreversible expansion of clay bricks can be separated into two time phases: up to two weeks old and beyond the age of two weeks. Within the first two weeks of the bricks being extracted from the kiln the bricks are known as ‘kiln fresh’. In this condition, they are known to exhibit a significant proportion of their overall irreversible expansion, very quickly. It is not therefore recommended to construct masonry using ‘kiln fresh’ clay bricks.

The second element of the clay masonry is mortar. Mortar is cementitious by nature and will therefore exhibit long-term shrinkage, which is a contraction. Assuming a drying environment, the shrinkage of the mortar is termed a drying shrinkage (which also includes a proportion of shrinkage which results from carbonation, however this is much smaller than drying shrinkage). Autogenous shrinkage is also present but is only considered significant when mortar is not exposed to drying. The shrinkage of mortar develops hyperbolically. As such, the rate of shrinkage is greater during the initial stages of drying and reduces with age of exposure to drying. Typically, 80% of the drying shrinkage of mortar will have occurred after one year’s exposure. However, shrinkage is significantly affected by the drying environment (particularly the humidity), the geometry of the masonry section (which determines the length of the drying path) and curing (the initial rate of shrinkage and the long-term shrinkage is reduced by prolonged curing). Measurements taken on a mortar prism, subsequent to 14 days’ curing, gave 2300 microstrain of shrinkage after one year (Forth and Brooks, 1995a). Within clay brick or calcium silicate masonry, mortar only accounts for approximately 13% of the vertical height of the composite so its contribution to the long-term movement of the masonry is much reduced. However, in practice it is very rare that masonry is cured for 14 days and normally one or two days is more common. Thus, the influence of the shrinkage of mortar on the overall moisture movement of masonry may therefore become more significant in practice.

It is important to note that as masonry is a composite it is actually not appropriate to consider the behaviour of the two elements in isolation; there is also the interaction of the two elements on each other to be considered. For instance, when bricks are laid with fresh mortar, the brick will absorb water from the mortar (this is all part of the bonding mechanism). However, if the masonry is exposed to drying after 1 day, some of this water will not be able to return to the mortar and will therefore be lost from the system by way of the brick (the moisture is being removed or lost from the mortar while it is in its plastic state and so does not contribute to its drying shrinkage which occurs once the mortar has set). This process can significantly reduce the shrinkage capacity of the mortar (Forth and Brooks, 1997).

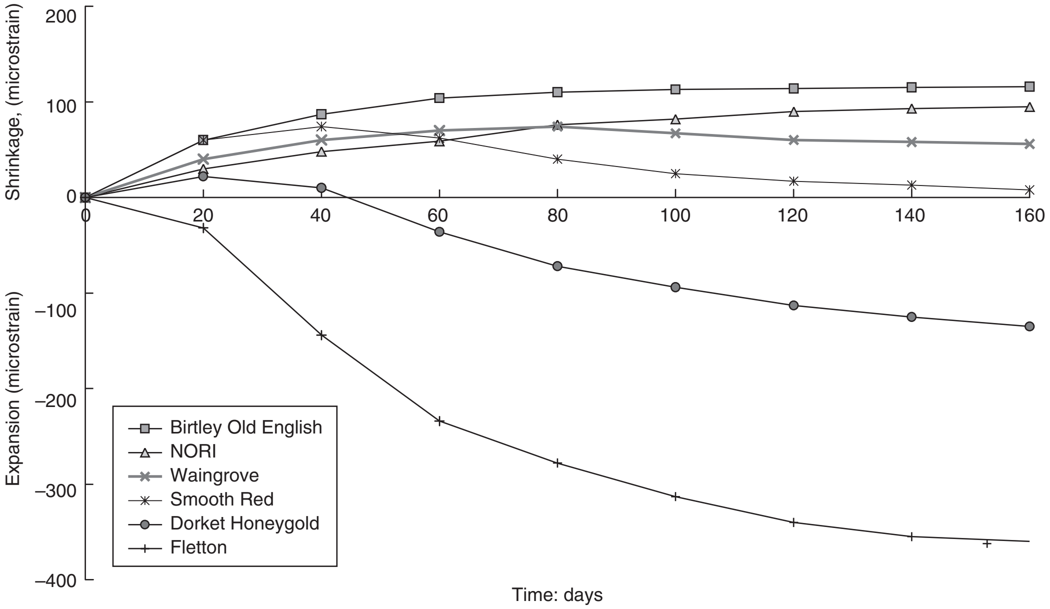


Figure 1 Moisture movement of various types of clay masonry

From these descriptions of the moisture movement of the individual elements of the clay masonry composite it is possible to see how the moisture movement of clay masonry may not always be an expansion (Forth and Brooks, 1995a). In fact, the long-term movement of clay masonry can exhibit itself as a long-term shrinkage, a short-term shrinkage followed by a long-term expansion, or a long-term expansion (see **Figure 1**).

Several attempts have been made to relate the moisture expansion of individual clay bricks with that of the corresponding clay brick masonry using a single value. The ratio of brickwork to brick movement has been suggested as 0.6 (Smith, 1992) (for horizontal movements, although it has also been quoted for vertical movements even though clay bricks are anisotropic (Forth and Brooks, 1997)). Researchers have also developed a method to predict the long-term movement of individual clay units (exposed to the external atmosphere but sheltered from the direct action of the environment) from a short-term test (Lomax and Ford, 1988). The test involves measuring the expansion of bricks which have been subjected to 4 hours of atmospheric pressure steaming; the results have been very successful. The idea of a single movement ratio in conjunction with this short-term test to predict long-term unit expansion would be very useful as a design tool for predicting the long-term moisture movement of the clay masonry. Unfortunately, the vertical movement of clay masonry cannot be represented by a single figure (as discussed above, owing to the possible trends of long-term moisture movement, the ratio could range from a negative expansion (a contraction) to a value greater than 0.6 but less than 1.0 or unity) as there are too many influencing factors (Forth and Brooks, 1995b). As is often the case with clay masonry, the variability is too great. However, the approach is encouraging and could be applied perhaps as a range which represents clay type, age, mortar type and environmental conditions.

Theoretically, the expansion of the masonry cannot be greater than the expansion of the individual brick, hence the maximum value of 1.0 for the brickwork/brick ratio mentioned above. However, cases have been reported, typically when high absorption bricks with medium to high soluble salt content are used, where the free vertical expansion of the masonry is indeed greater than that recorded from the individual unbonded bricks. Where this occurs, the expansion of the masonry has been deemed an ‘enlarged expansion’ (Forth and Brooks, 2000). This enlarged expansion occurs at the interface of the brick bed face with the mortar bed joint and is the result of soluble salts having been deposited in this region during the construction of the masonry. It is caused by a process called cryptoflorescence, and is a further example of where the interaction of the brick and the mortar elements has to be considered if the behaviour of the masonry composite is to be predicted. A simple guide as to whether this enlarged expansion has occurred is therefore to use the brickwork-to-brick ratio of 1.0 as a threshold value. The process of cryptoflorescence is discussed later in the Section ‘Chemical actions’.

As with mortar, concrete aggregate units or autoclaved aerated concrete units are cementitious by nature and will undergo long-term contraction due to shrinkage. The shrinkage of these units is similar in trend to that of mortar, although the long-term values of aggregate blocks is less due to the aggregate matrix that they possess (coarser; potentially gap graded) which is known to offer greater restraint to shrinkage. For these types of masonry, the movements of the individual elements are therefore additive and long-term shrinkage will result. Again, the interaction of the two elements, particularly considering the absorption properties of the units, may affect the overall masonry moisture movement.

Calcium silicate units are also known to exhibit shrinkage. However, due to lime being used as the binder, carbonation shrinkage forms the major portion of the drying shrinkage of these units. During manufacture, calcium hydroxide is produced during the steam curing of the lime. This calcium hydroxide is then converted to calcium carbonate (which is smaller by volume than the calcium hydroxide) by the carbon dioxide in the atmosphere. Due to this mechanism, shrinkage occurs more slowly than say for the concrete blocks or mortar. As with concrete block masonry, the movements of the individual elements (units and mortar) are therefore additive and long-term shrinkage will result in calcium silicate masonry. Note that, whereas with the other unit types, wetting and drying cycles may produce reversible moisture movements, research has shown that wetting calcium silicate units can enhance the long-term shrinkage of the unit, and hence the masonry, as it promotes the further generation of calcium hydroxide which can then be carbonated (Edgell, 1995).

The moisture movement of natural stone masonry units, where it occurs, is reversible and depends on the moisture content at the time of construction and the type of stone. For instance, limestone (a sedimentary rock) and granite (an igneous rock) exhibit insignificant amounts of movement due to changes in moisture content. However, sandstone (another sedimentary rock) can exhibit considerable dimensional changes with changes in moisture content and care must be taken when incorporating sandstone lintels in clay brick masonry. Natural stone masonry using natural limestone masonry units laid dry and protected from wetting would therefore be expected to exhibit only a small amount of moisture movement resulting from the shrinkage of the mortar alone. Whereas, masonry constructed using saturated natural sandstone masonry units which are allowed to dry immediately after construction could potentially exhibit a significant amount of shrinkage with or without the additive effect of the mortar shrinkage. It is clear that a range of dimensional changes is possible due to moisture movements in natural stone masonry and to recognise this, BS EN 1996-1-1: 2005 recommends a range of long-term moisture movements (in mm/m) of −0.4 (shrinkage; 400 microstrain) to +0.7 (expansion; 700 microstrain) for all magmatic (igneous), sedimentary and metamorphic rock types. These comments relate to natural stone units constructed with mortar using normal coursing patterns (including 'ashlar' masonry). These values do not specifically relate to dry-stone construction. However, it is possible that the moisture movements of the individual stone units may enhance, for instance, the causes of bulging often seen in this type of walling.

Reconstructed stone masonry units or manufactured stone masonry units, as they are now referred to in the more recent European Standards, are predominantly manufactured using aggregates with a cementitious binder and are intended to resemble and serve similar purposes as that of natural stone. (Typical aggregates and binders include natural, lightweight, foamed or expanded blastfurnace slag or pulverised-fuel ash (PFA) aggregates and various forms of Portland cement, ground granulated blastfurnace-slag, PFA or lime binders, respectively. They also incorporate certain types of admixtures.) As such, these types of unit tend to be grouped together with dense aggregate concrete units. As the units are cementitious by nature, it is reasonable to expect that they will exhibit long-term shrinkage and as such, to limit this moisture movement, the user needs to aim to keep the units as dry as possible before construction (higher moisture content relates to greater long-term shrinkage). Again, once constructed with mortar, which also shrinks, the reconstructed stone masonry is likely to exhibit long-term shrinkage. Prior to the new Eurocodes, a maximum figure of 0.06% (0.6 mm/m or 600 microstrain) for the long-term drying shrinkage of reconstructed stone units was recommended (BS 6457: 1984). This is reflected in BS EN 1996-1-1: 2005 which, as mentioned above, groups reconstructed stone with dense aggregate concrete units and recommends a range of long-term moisture movement for reconstructed stone masonry of −0.6 mm/m (shrinkage; 600 microstrain) to −0.1 mm/m (shrinkage; 100 microstrain).

Table 1 summarises the ranges of final moisture movement of masonry as recommended by BS EN 1996-1-1: 2005.

Type of masonry unit		Long-term moisture movement: mm/m
Clay		−0.2 to +1.0
Calcium silicate		−0.4 to −0.1
Dense aggregate concrete or manufactured stone		−0.6 to −0.1
Lightweight aggregate concrete		−1.0 to −0.2
Autoclaved aerated concrete		−0.4 to +0.2
Natural stone	Magmatic Sedimentary Metamorphic	−0.4 to +0.7

Table 1 Ranges of moisture movements for masonry

Application of loads

In its simplest form (unreinforced and not prestressed) and when subjected to a compressive stress, masonry is generally considered to be an elastic–plastic material. It is important to understand that, as a composite material, the movement of the masonry due to the application of a load is a combination of the movement of at least two elements. Predominantly, these elements are the unit and mortar elements. Both the unit and the mortar will contract elastically when subjected to compressive forces. However, if this compressive stress is kept constant, then the composite will then behave plastically as the materials creep. Creep is defined as an increase in strain with time, in excess of the elastic strain, due to a constantly applied stress. Although it is not surprising that the mortar will creep as it is cementitious by nature, it has also been found that masonry units can exhibit creep (Forth, 1995). In cementitious materials, the mechanism of creep is in part associated with micro-cracking, drying and the movement of adsorbed and inter-layer water within the microstructure of the material under the action of internal stresses which result from the external loads (Neville *et al.*, 1983). The creep of clay units is more a measure of the degree of microcracking within the material.

Elastic movement

For masonry loaded in compression, BS EN 1996-1-1: 2005 recommends that the short-term 28-day elastic modulus, E_{mas}, of the masonry can be predicted as follows:

$$E_{\text{mas}} = 1000 f_{\text{k}} \tag{1}$$

where f_{k} is the characteristic compressive strength of the masonry.

The elastic modulus, E_{mas}, presented here is actually the secant modulus derived at a value of one-third of the ultimate failure stress (the stress–strain relationship for masonry is non-linear (Ilston, 1994), hence the use of the secant modulus). The relationship is derived from a review, in 1989, of a wide range of published clay brick masonry results. This review actually plotted secant modulus against ultimate brickwork strength. The scatter in the data was very high, however it was found to be not too unreasonable to represent this data taking $E_{\text{mas}} = 1000 f_{\text{ult}}$, where f_{ult} is the stress at failure, with an upper-bound and lower-bound limit of $1200 f_{\text{ult}}$ and $300 f_{\text{ult}}$, respectively. The proposal following this review was to use the upper-bound figure for structural analysis and the lower-bound figure for serviceability considerations. Recently, EN BS 1996-1-1: 2005 reflected this approach by using $1000 f_{\text{k}}$ as the modulus for ultimate cases and $600 f_{\text{k}}$ for serviceability cases. By using f_{k} and estimating the characteristic compressive strength based on the type of unit, the unit strength and the type of mortar, the code also attempts to allow for much of the scatter illustrated in the 1989 review.

Clay brick unit type	Predicted modulus: GPa BS EN 1996-1-1: 2005	Measured modulus: GPa Masonry geometry Phase				
		1				2b
		SLW	CW	HP	SP*	SP
Fletton	8.0	1.5	1.6	1.4	1.1	3.7
Birtley Old English	8.6	5.7	–	–	–	6.7
Dorket Honeygold	12.9	4.8	–	–	4.0	6.4
Smooth Red	17.4	19.2	–	–	–	19.0
Nori	–	27.8	–	27.8	10.4	26.3
Waingrove	–	24.2	–	–	–	–

Notes:

* Solid pier subjected to low humidity curing conditions (40 to 50%)

SLW: Single-leaf wall; CW: Cavity wall; HP: Hollow pier; SP: Solid pier

Table 2 Measured and predicted modulus of elasticity (Forth and Brooks, 1994, 1995b)

Tables 2 and **3** illustrate the measured moduli of several different types of masonry and compare these measured moduli with those predicted by the code.

The measured moduli presented in **Table 2** were recorded at 28 days. The masonry had been carefully cured under polythene for 14 days and then exposed to drying in a controlled environment of 21 ± 1°C and a relative humidity of 65 ± 5%. A class II mortar had been used throughout. (No predictions are available for the Nori and Waingrove masonry as the code only caters for bricks with compressive strengths not greater than 100 MPa.) The code appears to significantly overestimate the values measured on the Fletton and Dorket Honeygold masonry. Although it overestimates the Birtley Old English masonry and

Mortar type	Measured modulus: GPa Single-leaf wall masonry: v/s = 44 mm	BS EN 1996-1-1: 2005
1 : 3	25.0	24.0
$1:\frac{1}{4}:3$	25.0	24.0
$1:\frac{1}{2}:4\frac{1}{2}$	23.8	18.2
$1:3\frac{1}{2}$: MC	23.4	18.2
1 : 4 : AEA	20.8	18.2
1 : 1 : 6	20.3	15.6
1 : 5 GGBS	19.5	15.6

Notes:

MC: Masonry cement; AEA: Air-entraining agent; GGBS: Ground granulated blastfurnace slag cement replacement; v/s: volume/surface area

Table 3 Measured and predicted modulus of elasticity of Armitage class B engineering single-leaf walls (Forth and Brooks, 1995b) underestimates the Smooth Red masonry, these predictions are more accurate. Both the Fletton and Dorket Honeygold units have high water absorption properties and it has been shown that the bond between the brick and the mortar can be influenced by the unit water absorption property, promoting microcracking (this is synonymous with the micro-cracks occurring at the aggregate–paste interface in concrete). When a load is applied, the strain recorded will also represent the closing of these microcracks. This enhanced strain will therefore result in a lower modulus for the masonry. The presence of microcracking helps to explain the extreme non-linearity in the stress–strain relationship sometimes recorded by other investigators at low stresses (Hendry, 1998). It also brings into question the relevance of using a relationship based on a secant modulus obtained at one-third the ultimate stress of the masonry. The presence of microcracking can also be shown to be dependent on the curing regime/drying environment. This is clearly shown by comparing the Solid Pier results of Phase 1 and 2b (**Table 2**) which shows how important curing/the environment are on the elastic properties of masonry.

Table 3 illustrates the measured moduli of Armitage masonry constructed using various mortar types. The masonry was cured for 14 days under polythene and loaded at the end of this curing phase. Although the code is conservative and underestimates the actual measured moduli, the predictions are all within 25% of the measured values which is reasonably accurate.

So far we have only referred to clay brick masonry, as results using this type of masonry have formed the basis for the code guidance. This code guidance is also supposed to be applicable to aerated and dense aggregate concrete blocks and reconstructed masonry units. In fact, there is some evidence to indicate that although reasonably appropriate, the code formula does actually underestimate the elastic modulus of these types of masonry (Edgell, 1995).

The design guidance for calcium silicate masonry is again based on a review of the available data and again there is a large scatter of results. For design purposes it is recommended that the elastic modulus of the masonry is equal to 650 multiplied by the compressive strength of the masonry.

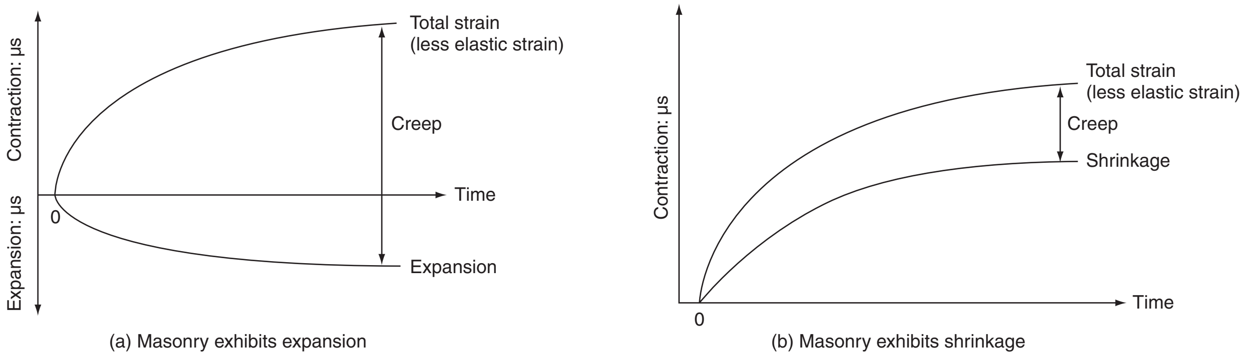
This section is intended to provide a note of caution to all designers who are expecting to predict the elastic behaviour of masonry. It is clear how influential curing, environment, age at loading and consistency of loading are on the behaviour of masonry. What is also clear is that as a designer it is difficult to predict these variables or to ensure certain controls at the design stage, a stage which is anything up to year from when construction commences (this also applies to the estimation of creep discussed below).

Creep of masonry

Neville (1983) defines creep of concrete as the increase in strain measured under constant stress after the elastic strain at loading has been deducted. If drying takes place under load, account also has to be made of shrinkage of concrete. In this case, creep is defined as the difference between the total time-dependent deformation of a loaded specimen and the shrinkage of the similar unloaded specimen, measured from the time of application of the load and stored under the same environmental conditions throughout the same period. In masonry, the same definition of creep has been adopted. For example, in a laboratory, the creep of the masonry (C) is given by the total strain ($\varepsilon_{\mathrm{wy}}$) measured on a wall subjected to a constant stress (σ_{y}), minus the elastic strain (e_{wy}) resulting from applying a load and minus the moisture movement strain (S_{wy}) measured from a control wall or:

$$C = \varepsilon_{\mathrm{wy}} - e_{\mathrm{wy}} - S_{\mathrm{wy}} \tag{2}$$

In Equation (2), the moisture movement strain is assumed to be positive for shrinkage and negative for expansion. Hence, for clay masonry, where long-term moisture expansion is possible, the moisture movement strain is therefore added (i.e. a double negative). **Figure 2**



(a) Masonry exhibits expansion

(b) Masonry exhibits shrinkage

Figure 2 Creep of clay masonry

Type of masonry		Value of ϕ_∞	
		Range	UK NAD (2005)
Clay		0.5 to 1.5	1.5
Calcium silicate		1.0 to 2.0	1.5
Dense aggregate concrete and manufactured stone		1.0 to 2.0	1.5
Lightweight aggregate concrete		1.0 to 3.0	1.5
Autoclaved aerated concrete		0.5 to 1.5	1.5
Natural stone	Magmatic Sedimentary Metamorphic	'Very low'	'Very low'

Table 4 Ultimate creep coefficients, ϕ_∞

illustrates the creep of masonry subjective to a compressive stress and exhibiting either moisture expansion or shrinkage.

BS EN 1996-1-1: 2005 provides guidance on the creep of masonry in terms of a range of ultimate creep coefficients, ϕ_∞. (A creep coefficient is the ratio of creep strain to elastic strain, both resulting from the same constant stress.) **Table 4** illustrates the final or ultimate creep coefficients recommended in this code and the UK National Annex Document to Eurocode 6 – Part 1-1: 2005.

The ultimate or final creep coefficient is required to calculate the long-term elastic modulus in the code (BS EN 1996-1-1: 2005). The long-term modulus is based on the short-term secant modulus reduced to allow for creep effects, such that:

$$E_{\text{long term}} = E_{\text{mas}}/(1 + \phi_\infty) \tag{3}$$

For the application of a unit stress (1 MPa), this simplifies to:

$$E_{\text{long term}} = 1/(\varepsilon_0 + \varepsilon_c) \tag{4}$$

where

ε_0 = the elastic strain due to a unit stress
ε_c = the ultimate creep due to a unit stress (ultimate specific creep).

Knowledge of the ultimate creep coefficient of masonry is also required by the code in order to calculate the effects of any accidental eccentricities. In the UK, the view in the past was that the creep of clay, calcium silicate and concrete blockwork was not likely to be significant, particularly in unreinforced masonry. (In fact, prior to the introduction of EC6, the UK codes used to allow for creep in the partial factors of safety. These factors ensured that the stresses were kept low enough such that any additional effects due to creep were limited and covered by the factors of safety/capacity reduction factor (Hendry, 1989; Haseltine, 1998).) However this view was not accepted and there are therefore now allowances for creep eccentricities in the new code as:

$$e_k = 0.002\phi_\infty(h_{\text{ef}}/t_{\text{ef}})\sqrt{(te_m)} \tag{5}$$

where:

ϕ_∞ = the final creep coefficient (**Table 4**)
h_{ef} = the effective height
t_{ef} = the effective thickness
t = the thickness of the wall
e_m = the eccentricity due to the loads in the middle fifth of the wall due to accidental eccentricity.

However, e_k may be taken as zero for all walls built with clay units and for any wall having a slenderness ratio of $\leqslant 15$ (which is greater than the normal floor-to-ceiling height utilised in standard housing construction).

It is important that a designer is aware of how the creep coefficient develops with time. Owing to the way creep of masonry is defined, the development of the creep coefficient is non-linear and the rate of development of the coefficient may be different with time, i.e. if the moisture movement of the masonry is one of short-term shrinkage followed by long-term expansion, creep will be much lower at early ages and increase at a greater rate with age. In design, short-term relaxation of induced stresses may be more important than any long-term effects and because the development of creep coefficient is not linear it is not possible to assume, for instance, the amount of creep after 6 months based on the ultimate values (something which is much more possible in concrete design).

Before the advent of the cavity wall in the early 1940s, masonry structures tended to be composed of elements with large cross-sectional areas which were therefore subjected to low stresses. Also, the mortar joints, where most of the creep occurred, were thinner. Hence, older designs tended to minimise creep. Since the introduction of the cavity wall system, masonry elements are now thinner and therefore more highly stressed. They are also constructed using thicker mortar joints. Creep may now therefore need to be considered in design. For instance, in a cavity wall where the inner leaf is concrete blockwork and the outer leaf is clay brickwork, differential stresses could develop as the concrete block will potentially creep more than the clay brickwork. Another example might be where a clay masonry panel is restrained by a stiffer structural member such as a foundation. To control the build-up of stresses, the movement must either be accommodated by providing movement joints or by allowing for it in the initial design.

In unreinforced masonry, the current design approach is to ensure that the stresses are still kept low. Clay units exhibit little creep; therefore, in clay masonry the creep is concentrated in the mortar joints. Although the creep of

Material	Coefficient of linear thermal expansion: $10^{-6}\,\mathrm{K}^{-1}$
Clay masonry units[a]	4 to 8
Concrete masonry units[b]	7 to 14
Calcium silicate masonry units	11 to 15
Mortars	11 to 13
Natural limestone masonry units[c]	3 to 10
Natural sandstone masonry units[c]	5 to 12
Natural granite masonry units[c]	5 to 10

[a] Thermal movement of clay masonry units depends on the type of clay

[b] Thermal movement of concrete masonry units depends on the type of material and the mix proportions

[c] Thermal movement of natural stone masonry units depends on the type of stone

Note: Concrete masonry units includes reconstructed units

Table 5 Coefficient of linear thermal expansion of masonry units and mortar. From BS 5628-3: 2005

Type of masonry unit		Coefficient of thermal expansion: α_t 10^{-6}/K
Clay		4 to 8
Calcium silicate		7 to 11
Dense aggregate concrete or manufactured stone		6 to 12
Lightweight aggregate concrete		6 to 12
Autoclaved aerated concrete		7 to 9
Natural stone	Magmatic	5 to 9
	Sedimentary	2 to 7
	Metamorphic	1 to 18

Table 6 Ranges of thermal properties of masonry

mortar can be quite high, the mortar forms only approximately 13% of the area of the clay masonry wall in the vertical direction so the overall creep of the masonry is low. Concrete units do creep. However, there is an even lower proportion of mortar area in the vertical height of a concrete block masonry wall (approx 6%). With the majority of concrete units having creep values potentially only 10% that of the creep of mortar, the overall creep of concrete masonry may be similar or slightly greater than that of clay masonry. This is an extreme generalisation; however, it is to a certain degree reflected in the code (see **Table 4**).

The values presented in **Table 5** are based on work originally performed by Lenczner (1980) on masonry piers between the 1960s and 1980s. Recent research has shown that these values are reasonable. However, they will tend to underestimate the creep of single-leaf walls and overestimate the creep in pier elements due to the influence of the geometry of the section. (A single-leaf wall has a smaller volume-to-surface area ratio than a solid pier; hence the drying path is shorter and so the wall will lose moisture more easily and faster – its creep will therefore be higher.) The values in the code also fail to recognise specifically the effect of mortar type, curing regime, age at application of load and environmental conditions, all of which have a major effect on creep as they do on elastic modulus. Creep coefficients (determined at 160 days) for some clay masonry loaded at an age of 3 days have been recorded as high as 2.7. Also, 160-day creep coefficients of 1.9 have been measured for aggregate concrete block masonry loaded at 3 days (Forth *et al.*, 1998). Although BS EN 1996-1-1: 2005 does provide a range of final creep values for each type of masonry, there is little guidance on how to choose a value within each range. It is therefore likely that a designer (if he or she decides to use the data available in Eurocode 6 – Part 1-1: 2005 instead of the single values available in the NAD (2005)) will understandably choose the larger value in order to be conservative, i.e. if the design situation requires the estimation of the greatest movement. However, although creep can be considered an additional movement which needs to be accommodated in design, the phenomenon is not always detrimental. Any stress build-up due to restraint of movement can be relieved by the masonry's ability to creep. For this case, with only limited guidance, the designer is likely to choose the smallest value recommended in the range, again, in order to be conservative and minimise the benefit of creep.

The creep of stone masonry is considered to be very low. Therefore, it is unlikely in modern stone masonry structures that the level of stress will be high enough to cause anything but primary creep and as such, apart from the potential for creep to enhance eccentrically applied loads discussed above, it is unlikely that creep will lead to structural failures. There have been recent failures in more ancient structures, i.e. the Civic Tower of Pavia in 1989 (Binda *et al.*, 1992). However, there were many well-documented reasons for this failure, of which creep may or may not have been a leading factor but more a secondary effect.

Finally, there are many relationships presented in the literature for the prediction of elastic and long-term movements of masonry; however, only one, developed by Brooks (1990) considers the full range of influences that affect these movements and has shown reasonable and consistent success.

Temperature changes

Masonry unit manufacturers will have data on the thermal movement of their units. Typical values of linear thermal expansion (or contraction; thermal movement is considered reversible for normal temperatures in the UK) of masonry units and mortar can be found in Table B1 of BS 5628-3: 2005.

Thermal movement data are also available on the corresponding masonry. This can be found in clause 3.7.4

of BS EN 1996-1-1: 2005, extracts of which are provided below.

The thermal movement of masonry is expressed in terms of the thermal coefficient of expansion, α_t. This coefficient describes the increase (or decrease) in length per degree change in temperature (microstrain/°C (or K)). These coefficients offer a guide to the longitudinal (or horizontal) thermal movement of masonry in an unrestrained state. In general, the horizontal movement of the masonry is considered to result from the movement of the individual masonry units, although some adjustment may be necessary for brick walls due to the greater quantity of mortar present (than for instance, a concrete block wall).

Therefore, the unrestrained linear horizontal thermal strain ε_t is:

$$\varepsilon_t = \alpha_t T \tag{6}$$

where T = temperature change.

The thermal stress, σ_t, induced will therefore be equal to the thermal strain divided by the elastic modulus, E_{mas}:

$$\alpha_t = \varepsilon_t / E_{mas} \tag{7}$$

However, in the vertical direction, as the movement of the mortar in the masonry is not restrained by the masonry units, the vertical movement may be determined by multiplying the dimensions of the units and the mortar by the respective coefficients.

As mentioned earlier, it is very rare that masonry is completely unrestrained or that it is entirely restrained. Any restraint to thermal movement, whether internal within the masonry itself (i.e. due to temperature gradients within the cross-section of the masonry element or differences in thermal coefficients of the individual elements of the masonry element) or external (from a foundation base) will lead to differential movements. This will then lead to the development of tensile stresses; these stresses may be axial or flexural depending on the detail (i.e. restraint along a base or restraint at a short wall return); however, they may or may not lead to cracking. How to incorporate the degree of restraint into an estimation of the linear thermal movement is complicated. Thermal movement in its simplest form as represented by Equation (4) could be rewritten to reflect the degree of restraint by incorporating a factor R:

$$\varepsilon_t = R\alpha_t T \tag{8}$$

where R = a factor for the degree of restraint.

Unfortunately, restraint factors have not been developed in masonry design and there is very little guidance on this phenomenon (in many ways, it relies on the experience of the individual designer). Previously, Copeland (1957) incorporated a factor for the degree of restraint in the development of his stress ratios for different wall aspect ratios to assist with the detailing of movement joints. His investigation was not concerned with determining a value for R for various restraint conditions but simply assumed a restraint value. Within his approach he also considered moisture movements in combination with thermal movements. A discussion on the cumulative effects of all the movement types is presented later. Copeland's approach requires further development; however, it is robust and, clearly, quantifying the internal and external restraint imposed on masonry in terms of a restraint factor would enhance his approach. A better understanding of restraint would assist designers with the detailing of movement joints in masonry structures today.

A few basic factors to consider when attempting an assessment of thermal movements:

- reflectivity value of the exposed masonry surface
- geographical orientation (south-facing walls are subjected to a greater temperature range than any other elevation)
- thermal mass of the masonry
- temperature differences between external and internal surfaces
- the use of different masonry materials within the structure (for instance for the inner and outer leaf of a cavity wall)
- the temperature at the time of construction
- variation in the external restraint of a masonry element (for instance, the restraint imposed on the internal leaf of a cavity wall infilling a framed structure will be very different to the outer leaf which may be continuous past the frame).

In many cases, due to the complexity of the situation and the limited guidance available to designers, an accurate assessment of the thermal movement of masonry will not be possible. However, on a more positive note, providing sufficient consideration of the situation is achieved, with subsequent appropriate structural detailing, it is likely that any thermal expansion or contraction will more likely lead to movement rather than cracking.

Chemical actions

Any dimensional change related to chemical action is mainly due to sulfate expansion. As such, this type of movement is predominantly related to clay brick masonry as it is only this type of unit which has a supply of soluble sulfate salts (from sulfate-bearing clays) although sulfates may also be derived from several locations related to the placement of the masonry structure, i.e. ground waters, the ground or flue gases in chimneys, etc. and so other types of masonry can be affected. Firstly, not all clay bricks have sufficient soluble salt content (mentioned in Chapter 31). Secondly, for this type of expansion to occur, the masonry must either remain wet or be subjected to wetting and drying cycles. Finally, it is important to appreciate the location of this expansion. The chemical reaction is one that exists between the soluble sulfates and certain components of the cement mortar. It is therefore the mortar joint where this expansion occurs. This action may also lead to a deterioration of the mortar.

As mentioned in Chapter 2, clay brick masonry can, if the soluble salt content is high enough, exhibit efflorescence (the deposit of a white salt substance on the surface of the bricks possibly up to one year or more after the bricks have been laid). This deposition is often cyclic and is related to the wetting and drying cycles that occur in masonry exposed to the environment. In many ways, although aesthetically undesirable, structurally it is the best result. When the bricks are wet from the rain, the salts are put into solution and a humidity gradient is established within the brick (highest at the surface). When the rain stops, the bricks begin to dry out and the humidity reduces at the surface and becomes greater within the brick. This reversal of the humidity gradient draws the moisture, which carries the salts now in solution, out from within the brick. Efflorescence, as was explained earlier, is when the moisture is carried to the surface and when moisture is removed the salts are deposited on the surface. However, what if the salts are deposited before the moisture gets to the surface? If this is the case, it can actually be quite serious as several cycles of deposition of salts beneath the surface can lead to the surface layer of the brick being pushed off.

Similarly, when bricks are laid, moisture is absorbed by the brick from the mortar to depths of around 20 mm, depending on the absorption properties of the brick. If soluble salts are present within the brick, these will then pass into solution. As the pore structure of the mortar establishes itself during setting and beyond, this moisture is pulled back towards the mortar and out of the brick. As drying commences, salts are deposited at the interface of the brick and the mortar in the region of the bed joint (the third phase referred to earlier). The deposition of these salts ultimately leads to an expansion in the masonry which is in excess of the individual brick expansion and has therefore been termed an enlarged expansion (see ‘Changes in moisture content’ above). This enlarged expansion is caused by the process of cryptoflorescence which is the deposition of salts beneath the surface of the unit (Forth and Brooks, 1995b).

Both expansions occur in unrestrained masonry such as free-standing walls or parapets or where restraint is very low as it has been shown that this expansive force can be restrained by a compressive stress as little as 0.15 MPa (Forth and Brooks, 2008).

Chemical expansion can also occur in the form of corrosion of cavity wall ties. This expansion is more localised; the normal frequency of cavity wall ties is up to 900 mm apart horizontally and every sixth course vertically (specific guidance is available in BS EN 1996-1-1: 2005). Corrosion of wall ties occurs due to the ingress of chlorides from, for instance, de-icing salts. When the ties corrode, the oxide or rust produced exceeds the volume of that of the original metal (the rust volume is approximately five times the volume of the original metal). This localised expansion may be sufficient to cause cracking. Since the 1980s, stainless steel ties have been used in the UK. However, corrosion of ties may be a problem in any house built since the 1940s when cavity wall construction became dominant. Between the 1940s and the 1980s mild steel ties were used although towards the end of this period they may have been galvanised, offering some protection from corrosion. The extent of corrosion of the ties may be limited to the outer leaf of the cavity or to both inner and outer leaf. This type of localised expansion/cracking may be applicable to all types of masonry. In summary, due to the localised nature of the expansion, it is the potential cracking which is of concern in this type of chemical attack. The failure of the wall ties due to corrosion can potentially cause the loss of structural integrity and lead to structural failure. Any localised cracking due to the expansion of the ties if identified early enough may be a precursor to failure and an indication that remediation is required. Remedial work of this nature, although potentially intrusive, is available as standard within the construction industry.

Movement effects are cumulative

It is important to understand that if movements due to any of the four causes discussed above are identified, they do not occur in isolation. For instance, as well as thermal movement, the other effects such as reversible/irreversible moisture movement, chemical- and load-induced movement may also still be acting i.e. these effects are cumulative.

For typical temperatures within the UK, the coefficient of thermal movement for masonry is considered to be the same for both heating and cooling. Considered on a daily basis, movements resulting from thermal effects are therefore taken as reversible. However, there are two points to be aware of here.

1 Consider masonry subjected to a daily temperature cycle and exhibiting irreversible expansion. (Over one day, it is likely that the thermal dimensional change will be the dominant movement.) Any restraint of expansion (from irreversible moisture expansion and temperature increase) will put the masonry into compression and generate compressive stresses which will, to some extent, quite rapidly be relieved by compressive creep. Any restraint of contraction (due to cooling; remember this contraction will be less than the expansion due to the effect of the irreversible expansion) will put the masonry into tension producing tensile stresses which will again to some extent be relieved by tensile creep. Significantly, there is evidence to indicate that at low levels of stress, tensile creep may be greater than compressive creep (up to five times greater). If this is

the case in masonry, daily cycles of temperature may result in a build-up of compressive stresses, even after the effects of long-term creep have been considered. Of course, this is dependent on the age of the masonry and is also affected by the time of the day and the time of the year at which the masonry is constructed.

2 For masonry constructed during the summer and exposed to an external environment, the overall movement for the first 6 months is likely to be contraction due to the ambient temperature cooling as winter approaches. If the irreversible moisture movement during this period is one of long-term shrinkage, the contraction will be enhanced. If it is one of long-term expansion then the contraction will be reduced. It may also be pertinent to consider the reversible movement of the masonry. It could be argued that the masonry is likely to be wetter at the end of the 6-month period (we are now in the depths of winter) and therefore it is likely to exhibit reversible expansion which will act to reduce the contraction. With restraint present, tension stresses will be induced which may lead to cracking (some stresses will be relieved by creep). Subsequent expansion during the following summer may also help to relieve these tension stresses and cause any cracks that have formed to close (note; there is a likelihood that that cracks may not close due to the degradation of the crack surfaces).

For masonry constructed during the winter, the overall movement over the first 6 months is likely to be expansion as summer is approached (this may again be enhanced or reduced by the irreversible movement trend; it is likely to be reduced by the reversible movement) and hence will result in compression within the masonry. This compression is likely to have a minimal effect however; it could be argued that these compressive stresses may enhance any eccentricity effects but the relief by creep is likely to make these stresses insignificant. Subsequent contraction during the following winter may also reduce these compressive stresses. Any tension stresses induced from restraint to contractions during the following winter period will also be reduced as effectively the masonry is starting from a compressive stress state. It may therefore be better to build in the winter!

In the above scenarios, restraint is considered in the form of end-restraint. The examples are purposely simplistic, however they demonstrate the potential cumulative effects of these movements and the complexity of these effects when considered cumulatively. (Currently, no research exists to indicate whether the daily or yearly effect is more critical.) The effect of this cumulative movement may actually only be considered a problem if the result of these movements is the generation of cracks within the masonry. It may therefore help to consider that cracks are not caused by the temperature or moisture changes but by the fact that the tensile stress generated from the restraint of this movement has exceeded the tensile strength capacity of the masonry.

Is cracking a problem?

The design philosophy of certain structural materials, for instance reinforced concrete, is such that elements designed using reinforced concrete are designed to crack. Design methods are utilised which limit/control the cracking. It must be made clear that masonry structures are not designed to crack. Towards the start of this chapter, four major causes of movement which result from the inherent properties and behaviour of masonry were given. (Externally induced movements, i.e. from settlement, were excluded.) Therefore, if the designer has adequately accommodated the potential movements with appropriate design detailing and, where necessary the incorporation of movement joints, then the masonry should not exhibit cracking.

If the masonry is going to crack due to the cumulative effect of these four types of movement, it is likely that cracking will occur reasonably soon in the life of a structure, owing to the rates and duration of these movements (although it is accepted that corrosion is a long-term problem). On the whole, these cracks, once formed, remain fairly stable and with careful treatment can be repaired. It is important that these cracks are repaired as movement cracks in masonry structures, if left unrepaired, can lead to durability problems, as, for instance, it permits the ingress of water and to structural problems in terms of the potential reduction in the overall stability of the structure. During the early life of a masonry structure there is the potential for both movement and structural cracking to occur. These cracks may be co-located or independent. A professional inspection should, however, be able to indicate the cause of the cracking. It is, though, unusual to find cracks formed in older masonry structures which result from the dimensional changes discussed above (again accepting the potential for cracking from tie corrosion and perhaps where there has been a change of use to the building and this has been incorrectly designed). Cracking in older masonry structures is therefore useful as it can act as an indication of potential structural problems.

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