Structural Grouts

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Preface

Over the past two decades, cementitious grouts have been used for an increasing number of exciting structural purposes, with ever more demands being placed on the grout. Many of these developments have been made possible by the use of admixtures and cement replacement materials, such as pulverised fuel ash, which are now widely accepted as giving economic and, more critically, technical advantages in all cementitious based materials.

This book gathers together a substantial amount of information on grout properties, utilisation and practice which has hitherto been available in only fragmented or restricted forms. The book is aimed principally at engineers who may be considering the effectiveness or usefulness of grouting for a given application, writing specifications, designing grout mixes or planning grouting operations. It will also be of value to advanced course students and researchers.

The emphasis is on achieving efficient and effective use of grouts from an understanding of the nature of the behaviour of cements and cementitious materials; the first part of the book, 'Properties', is directed to this end. The first chapter discusses the structure of fresh and hydrated cementitious composites, the second chapter deals with fresh and early age behaviour, and the third chapter hardened properties. All three chapters deal mainly with grout containing Portland cements, and all discuss the influence of cement replacement materials and admixtures. The fourth chapter considers a number of more specialist grout mixes.

The second part of the book, 'Applications', discusses some interesting and innovative structural grouting situations, including grouting in steel and concrete offshore structures, grouted fabric formwork, prestressing duct grouting, grout for structural repairs and grouting of tunnel linings. Each chapter in this section has been written by an experienced practitioner. In each case, the characteristics and nature of the grout used and the grouting operations, plant and quality control are described. Case studies are included. The reader will be able to relate much of the information presented to the first part of the book.

A problem that we encountered in the early stages of determining the scope of the book was the definition of a grout. Some would argue that, strictly, a grout should be a mixture of cement (or binder of cement plus supplementary cementing material), water and admixtures only. However, it becomes apparent that most practitioners would include a mixture containing an inert filler (i.e. a fine aggregate) in the overall definition, particularly when the same or similar methods can be used to place the filled or unfilled mixture, and the choice is based primarily on the properties required. We have therefore used this wider definition, and the earlier chapters include reference to the effect of a filler on the relevant properties.

Although the book is not confined to offshore grouting, a considerable amount of the information presented relates to this. This has perhaps seen the most significant development in recent years, although the 'spin-off' to onshore practice has been considerable. Both of the editors, and a number of the authors, participated in the UK based Managed Programme on 'Grouts and Grouting for Construction and Repair of Offshore Structures' which ran from 1977 to 1987 and was promoted by the Marine Technology Directorate and sponsored jointly by the Science and Engineering Research Council, the Department of Energy and the offshore industry.

P.L.J.D.
S.A.J.

Note on standards

There is a large number of standards and specifications from many countries that are relevant to the grouting materials, formulations, uses, construction methods and practices described in this book. We have not attempted to provide a listing of these but details of those directly referred to in the text are included in the reference lists at the end of the relevant chapter.

As far as possible the authors have, where appropriate, used terminology corresponding to that in current standards and specifications, with one important exception—Portland cement. Until 1991, British Standard BS 12 defined Ordinary Portland Cement, and all cement users became familiar with this product and the 'typical' or 'average' properties to be expected from its use. In anticipation of the European Standard for cement (ENV 197) BS 12 was revised in 1991, with classification based on the compressive strength of a standard mortar made with the cement. The classes are 32.5, 42.5, 52.5 and 62.5 (corresponding to 28 day strengths in N/mm^2), with each class sub-divided into L, N or R for low, normal or high early strength. The terms 'Ordinary' and 'Rapid Hardening' Portland Cement are no longer used in the standard, the nearest equivalent being class 42.5 and 52.5 respectively. The standard for Sulphate Resisting Portland Cement (BS 4027) has been retained, but revised to include similar strength classes.

However, most of the data presented in the book were obtained in the time of 'Ordinary Portland Cement'; furthermore, it will be many years before all those concerned with Portland cement will stop referring to 'opc'. For these reasons we have continued with this usage throughout this book, and not changed to the less familar and more clumsy 'class 42.5N Portland cement'.

In the United States, the American Society of Testing and Materials standard ASTM C150 recognises five different types of Portland cement. ASTM Type I is a general purpose cement, roughly equivalent to the previous British Ordinary Portland Cement, Type II is a low heat cement offering moderate resistance to sulphate attack, with no direct British equivalent, Type III is a rapid-hardening cement, Type IV a slow-hardening low heat cement and Type V a cement with improved sulphate resistance, roughly equivalent to the British Sulphate Resisting Cement.

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Part One
Properties

1 Chemical and physical structure of cement grouts

P.L.PRATT

1.1 Introduction

The last two or three decades have seen a substantial increase in both the range and quality of the materials used in cementitious grouts, and the understanding of their behaviour, although the latter is by no means complete. It is the aim of this chapter to provide a description of:

- the chemical and physical nature of the most important of these materials, which include Portland and aluminate cements, cement replacement materials (also known as mineral admixtures) and chemical admixtures;
- the processes involved when these react with water and each other; and
- the structure of the products so formed.

This will necessarily be brief, but will hopefully provide a sufficient basis for the understanding of much of what follows later in the book. The reference list includes further texts that can be consulted for more extensive and detailed information should this be required.

1.2 Portland cements

Portland cement was patented by Joseph Aspdin early in the nineteenth century. The cement was made by burning powdered chalk and clay in a kiln followed by grinding the resultant clinker to a fine powder. Today, late in the twentieth century, similar raw materials are still used with a well-controlled burning process at significantly higher temperatures than in the early kiln.

1.2.1 Manufacture

Chalk or limestone are used as the source of calcium oxide, and clays or shales are used as the source of silica in the manufacture of Portland cement. These widely available minerals are crushed, ground and blended together before burning in a long, rotating, inclined kiln to form calcium silicates, which are the principal cementing phases. Besides silica, clays and shales contain iron and aluminium oxides. These give rise to calcium aluminates and calcium ferroaluminates, which act as fluxes in the production process, lowering the melting temperature of the raw mix so that clinkering of the cement occurs more easily at lower kiln temperatures. In a modern 'dry-process' plant, the raw mix is heated to 800–1000°C using the kiln exhaust gases in a pre-calciner, to remove carbon dioxide and water, before passing into the kiln in which it is heated slowly to about 1500ºC. Dicalcium silicate ($2CaO.SiO_2$, or C_2S in shorthand form*) forms first, followed by melting of the calcium aluminosilicate phases at about 1200ºC leading to rapid shrinkage and the formation of nodules by sintering. At temperatures of 1200–1500ºC, C_2S and the remaining free lime dissolve in the melt leading to the crystallisation of tricalcium silicate, C_3S. The final product consists of clinker nodules containing C_3S and C_2S in a matrix of the molten phase which crystallises on cooling to form C_3A and C_4AF. This latter is sometimes called the interstitial phase, because it fills the interstices between the silicate phase crystals. Minor phases include free lime, CaO, and magnesium oxide, MgO, as well as alkali metal sulphates.

C_3S, C_2S, C_3A and C_4AF are idealised forms of the phases actually found in cement. Impure C_3S, known as alite, and impure C_2S, or belite, both contain impurity ions in solid solution such as Mg^{2+}, Al^{3+}, Fe^{3+} and K^+ and Na^+. These impurity ions tend to stabilise the high-temperature crystal structures (or polymorphs) of the silicate phases and the same is true of alkali metal ions in C_3A, the aluminate phase. C_4AF, the ferrite phase, is part of the solid solution between C_2A–C_2F, with a ratio of A/F =1. The actual ratio A/F in a particular cement determines the composition of the ferrite phase.

After cooling, the clinker nodules are ground in a ball-mill to a fine powder with a small amount of gypsum added to control the early hydration of C_3A, thus avoiding flash-setting.

1.2.2 Oxide analysis and phase composition

Compositions of Portland cements are given in terms of the oxides of the elements present, with typical ranges given in the first column of figures in Table 1.1.

The potential compound (or phase) composition of a cement is often calculated from its oxide composition using the method pioneered by Bogue

*Cement chemists use a shorthand notation to describe the phases present in cement, with single letters standing for the oxides. Thus CaO=C, SiO_2=S, Al_2O_3=A, Fe_2O_3=F, MgO=M, H_2O=H and $SO_3=\bar{S}$. In this way dicalcium silicate, $2CaO.SiO_2$, becomes C_2S, tricalcium silicate, $3CaO.SiO_2$, becomes C_3S, tricalcium aluminate, C_3A and tetracalcium alumino ferrite, C_4AF. Similarly, gypsum, $CaSO_4$, becomes $C\bar{s}$, and calcium hydroxide, $Ca(OH)_2$, becomes CH.

Table 1.1 Typical compositions of Portland cements

		Cement				
		1	2	3	4	5
Oxide analysis (%)						
SiO_2	18–24	20.0	19.9	20.2	20.7	21.6
Al_2O_3	4–8	5.3	5.2	4.9	3.6	3.9
Fe_2O_3	1–5	3.4	3.3	3.1	5.9	5.2
CaO	62–67	64.5	65.0	64.5	64.5	64.2
MgO	0.5–4	1.2	1.2	1.2	1.1	0.8
SO_3	2–3	3.0	2.6	2.9	2.4	2.3
K_2O		0.8	0.6	0.7	0.5	0.4
Na_2O		0.1	0.2	0.1	0.2	0.1
Free lime	0.5–2.5	2.2	1.0	1.0	0.8	1.6
Loss on ignition		0.9	1.6			1.2
Bogue composition (%)						
C_3S		52.6	62.3	61.4	65.8	50.1
C_2S		17.7	10.0	11.6	9.7	24.2
C_3A		8.3	8.2	7.7	0.0	1.5
C_4AF		10.3	10.0	9.4	17.9	15.9
Specific surface area (m^2/kg)		421		376		335

(1929), which assumes that the phases have their simple chemical formulae. Examples of the results of this calculation for four Portland cements are shown in Table 1.1, from which it is immediately apparent that very small changes in the amount of oxides can lead to large changes in the amounts of the phases. The Bogue calculation in fact gives incorrect results because the phases do not have the assumed simple chemical compositions, and are not likely to be in equilibrium in view of the rapid cooling of the clinker as it emerges from the kiln. Taylor (1990) has considered this problem carefully, concluding that the values given by the Bogue calculation are low by as much as 8% for alite, low for aluminate, high for belite and perhaps satisfactory for ferrite. He gave details of a modified calculation based on using the best available estimates of the actual composition of the four phases instead of those of the pure compounds, and he then found reasonable agreement with experimental results from microscopy and quantitative X-ray diffraction (QXRD).

Comments on the resulting properties of the cements in Table 1.1 are included in section 1.2.5.

1.2.3 Particle size and shape

Cement is ground to a fine powder so that the process of hydration can occur both quickly and completely to make the most economic use of the material. The particle size of ordinary Portland cement (opc) ranges from a few microns to perhaps 50 μm, with an average of 10–20 μm. The smaller particles hydrate

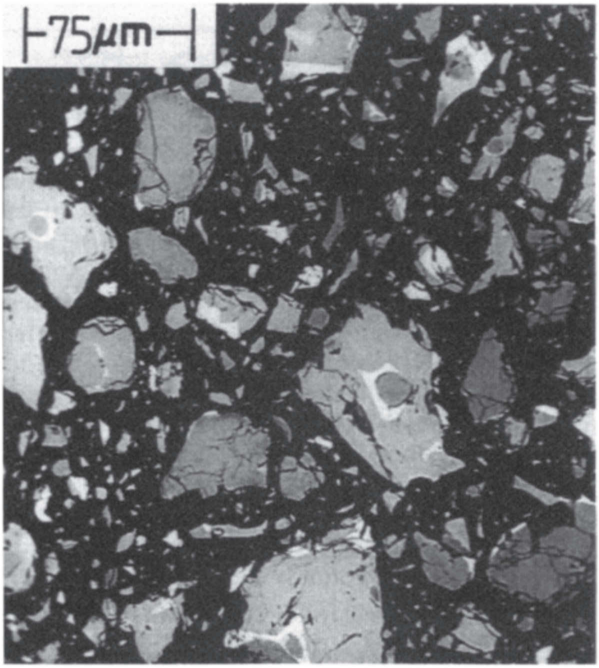

Figure 1.1 Back-scattered electron micrograph of unhydrated Portland cement (Lota *et al*., 1990).

rapidly in the first 24 h while the largest particles are unlikely ever to hydrate completely. A good idea of the range of sizes can be seen in Figure 1.1, which shows cement particles distributed in resin on the stub of a scanning electron microscope. The specimen has been polished with diamond paste so that the particles are seen in cross-section. By using back-scattered electron-imaging regions of different phase compositions are seen as different shades of grey; in this way the larger particles are seen to be polymineralic, containing C_3S, C_2S and the interstitial C_3A and C_4AF, while the smaller particles are mostly single-phase C_3S. This comes about because C_3S is the most brittle phase which often cracks during cooling of the clinker and is the easiest to grind.

Cement particles tend to be sharp and angular; their shape is determined by the way in which the clinker cracks during the grinding process in the ball mill. Pre-existing cracks in the C_3S tend to join up and new cracks form most easily in the interstitial material and between individual grains of alite and belite. Often the proportion of C_3A and C_4AF exposed on the surface of ground cement is significantly higher than their overall content in the cement and this has implications for the subsequent hydration.

The distribution of particle sizes is readily obtained using a laser-based instrument which measures the amount of light scattered over a range of angles by the cement particles, suspended in a liquid. Figure 1.2 shows

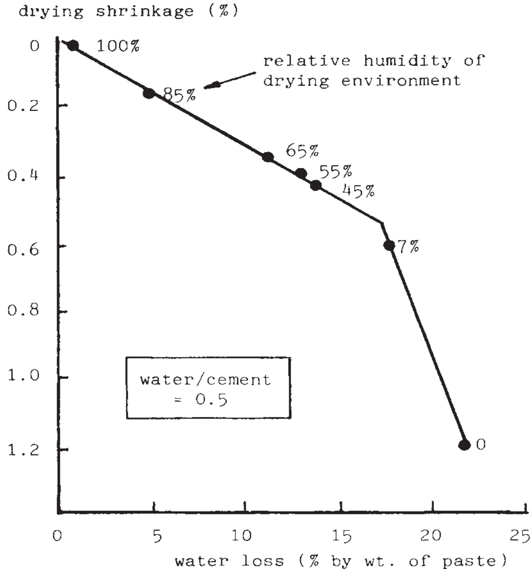

Figure 1.2 Typical particle size distribution of Portland cement and pulverised fuel ash (pfa) (Halse et al., 1984).

results for a range of cements and a typical UK pfa (see section 1.3.1), plotted as a percentage by weight below a particular particle size.

1.2.4 Specific surface area and specific gravity

The fineness of a cement controls both its setting time and its rate of strength gain. It is normally described by the specific surface area, i.e. the total surface area of all the particles per unit weight of the material, in units of m^2/kg. The smallest particles contribute the most to the total, and therefore the measurement of these is the most important. Unfortunately this measurement is difficult, and the result depends very much on the method used. A variety of techniques have been used including nitrogen adsorption, air permeability and light scattering, each giving substantially different results for the same cement sample. Typically, specific surface areas of 1000, 300–350 and 150–200 m^2/kg respectively have been obtained. The reasons for the differences include:

- Nitrogen adsorption occurs on all exposed surfaces, including any surface pores and cracks to which the nitrogen atoms can gain access.
- Permeability methods measure the resistance to flow of air through a packed bed of cement of known dimensions and porosity. Lea and Nurse (1939) introduced the constant flow-rate method which forms the basis of BS 4550 (BSI, 1978), while Blaine (1943) developed the simpler constant volume method which is widely used in the USA, the UK and in many

other countries. It is normally calibrated against the Lea and Nurse method. A significant amount of the surface area of pores and cracks do not contribute to the flow resistance, and so a lower result than that from the nitrogen adsorption test is obtained. The results are analysed using the Carman-Kozeny equation for viscous flow through a bed, which involves knowledge of the density of the cement. This is measured by the displacement method in kerosine and typical figures for cements are 3100–3250 kg/m^3.

- The light-scattering turbidimeter, developed by Wagner (1933) in the USA, measures the amount of light transmitted through a column of cement dispersed in kerosene as a function of time. As the cement settles to the bottom, so the top of the column transmits more light and the bottom becomes opaque. This method assumes an artificial lower limit of 3.8 μm for all particles smaller than 7.5 μm and so underestimates the specific surface area compared with the other techniques.

1.2.5 Comparison of sulphate-resisting, oilwell and rapid-hardening Portland cements

Table 1.1, already briefly discussed in section 1.2.2 above, shows the relationship between oxide and phase composition for a number of Portland cements. Despite significant variations in the phase composition, cements 1 to 3 would until recently have been known as Ordinary Portland cements (see note on standards on page vii), the most noticeable difference being the higher free lime content of cement 1 resulting in relatively low C_3S and high C_2S contents. Cements 4 and 5 have reduced A/F ratios, resulting in a zero or near zero C_3A content and a higher C_4AF content. Converting C_3A to C_4AF in the cement in this way offers improved resistance to sulphate attack (see chapter 3). Cement 4 is, in fact, typical of sulphate-resisting Portland cement. Cement 5 is an Oilwell G cement, which also has low C_3S/high C_2S and is coarse ground to help slow down the hydration process at the high temperatures deep in an oil well. Table 1.2 gives typical compound compositions and properties of Portland cements classified according to the United States ASTM standards. Type I is roughly equivalent to cement 1 in Table 1.1, with the exception of being slightly less finely ground. Type II, with a lower C_3A content has moderate sulphate-resisting properties, and the rapid-hardening properties of type III have been achieved by a higher C_3S content (primarily at the expense of C_2S) and a finer grinding. The improvement in 1 day strength and the consequent 'penalty' of higher heat of hydration is apparent. The low heat property of type IV cement results from the reduced C_3S content and the coarser grinding, and the strength gain is correspondingly slower. The lower C_3A content of type V gives this cement sulphate-resisting properties, as in cements 4 and 5 of Table 1.1.

Table 1.2 Typical composition (%) of Portland cements to ASTM C150 (Mindess and Young, 1981)

	Cement type				
	I	II	III	IV	V
C_3S	50	45	60	25	40
C_2S	25	30	15	50	40
C_3A	12	7	10	5	4
C_4AF	8	12	8	12	10
Specific surface area (m^2/kg)	350	350	450	300	350
Compressive strength (1 day, N/mm^2)	7	6	14	3	6
Heat of hydration (7 days, J/g)	330	250	500	210	250

The strength and hydration properties of all cements can be readily altered by the use of chemical and mineral admixtures and by altering the temperature at which hydration occurs.

1.3 Cement replacement materials

A number of industrial waste products can be added to ordinary Portland cement to form blended cements. As well as cement replacement materials, these materials are often called **mineral admixtures** or the more descriptive **supplementary cementing materials**. They all contain significant amounts of silica in a finely divided active form, which is capable of dissolving in the high pH pore solution of hydrating OPC to form further calcium silicate hydrate. Provided the waste products are less expensive than OPC, and are readily available in a suitable form, it is common practice to use them as cement replacement materials. In many cases these mineral admixtures react more slowly than OPC, giving increased strength and improved impermeability to the hardened cement paste at later ages.

1.3.1 Pulverised fuel ash (pfa) or fly ash

Coal-fired power stations burn finely powdered, or pulverised, fuel in their boilers to raise steam to drive the turbines. The ash from this finely powdered fuel is in the form of small spheres which are prevented from escaping via the chimney into the atmosphere by electrostatic precipitators. As Table 1.3 shows, United Kingdom ashes contain about 50% SiO_2, 30% Al_2O_3 and 8% Fe_2O_3. These low-lime ashes, equivalent to class F fly ashes in the United States, contain 2–3% CaO and about 4% K_2O. The average particle size is about 10 µm, and the particle size distribution is typically a little coarser than Portland cement (Figure 1.2)

Table 1.3 Typical composition of pulverised fuel ash (pfa), ground-granulated blast furnace slag (ggbs) and condensed silica fume (csf)

	pfa	ggbs	csf
Oxide analysis (%)			
SiO_2	50.0	36.5	96.6
Al_2O_3	31.1	11.3	1.7
Fe_2O_3	8.1	0.9	0.1
CaO	1.9	39.6	–
MgO	1.6	8.2	0.1
SO_3	0.8	0.3	–
K_2O	4.3	0.6	–
Loss on ignition	3.8	2.1	1.5

1.3.2 Ground-granulated blast furnace slag (ggbs)

Blast furnace slag is a high-lime waste product with about 35% SiO_2, 11% Al_2O_3 and 40% CaO (Table 1.3). Granules or pellets are produced by rapidly cooling the molten slag with air and water to give particles a few millimetres in diameter which are largely glassy in structure. These have to be ground, or interground with the cement, to a fine powder before use. Since slags contain both CaO and SiO_2 in near-equal proportions, they are capable of being activated by a variety of chemicals, including Portland cement, to form calcium silicate hydrate.

1.3.3 Condensed silica fume (csf) or microsilica

Condensed silica fume is a by-product from the extraction of silicon or the manufacture of ferrosilicon. It has to be condensed and precipitated to prevent escape into the environment. The result is submicron sized particles of nearly pure SiO_2 (Table 1.3). The particles are thus very much smaller than those of either Portland cement, pfa or ggbs.

The relationship between the compositions of the these three mineral admixtures can be seen in the ternary diagram of the $CaO–SiO_2–Al_2O_3$ system shown in Figure 1.3. Also shown are the composition of Portland cement and high alumina (or calcium aluminate) cement.

1.4 Chemical admixtures

A wide range of chemicals can be added to cement grouts during mixing to control the processes of setting and hardening and, less directly, the strength of the hardened grout. Like the mineral admixtures used to replace cement, a

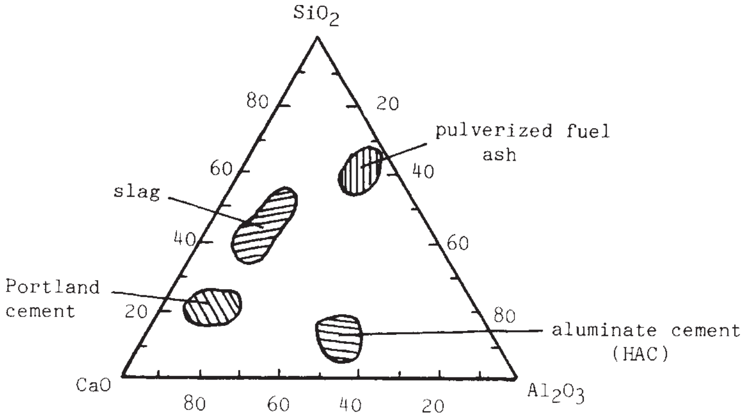

Figure 1.3 Ternary diagram of $CaO–SiO_2–Al_2O_3$ system (Regourd, 1985).

number of the chemical admixtures are made from the waste products of other industries, and their interaction with the chemistry of cement hydration is by no means clear. It is important to carry out a trial mix with a particular branded product, and the specific cement to be used for a particular grouting application, to ensure that the change(s) in properties that are required are of the correct type and magnitude.

Admixtures are used to accelerate or retard setting and hardening, to control the workability, or, more generally, the rheology of the fresh mix, and to improve the durability.

1.4.1 Accelerating and retarding

Acceleration and retardation of setting of a particular cement is achieved by changing the kinetics of the chemical reactions of the silicate phases in the cement in the first few hours of hydration. Accelerators, such as calcium chloride, calcium formate and sodium chloride (as in sea water) all increase the rate of hydration of C_3S when added typically at 0.5 molar concentration in the mixing water. Other inorganic salts also produce an accelerating effect increasing with increasing charge and decreasing size of the ion. For example, sodium aluminate acts as an accelerator, with its main effect being to change the hydration of the aluminate phases, producing more hexagonal aluminate hydrates. This can lead to very rapid setting.

Retarders appear to act by adsorption onto the surfaces of hydration products both from C_3S and from C_3A. Ramachandran, Feldman and Beaudoin (1981) showed that retarders are most effective with cements low in aluminate and low in alkali and that they should be added a few minutes after mixing when the aluminate has already started to react. Typical retarders contain organic molecules such as sugars, hydroxycarboxylic acids or their salts, such as citrates, and lignosulphonates. The lignosulphonates are waste products from the wood pulp and paper industry and often contain sugars as a significant impurity.

The amount of admixture used can be very critical and the manufacturer's instructions must be followed carefully with a trial mix before use. Some admixtures, e.g. glucose, accelerate the hydration of the aluminate phases and retard that of the silicate phases. Others act as accelerators at low concentrations with a particular cement and as retarders at high concentrations, with these effects varying with temperature.

1.4.2 Controlling workability and durability

The simplest way of increasing the workability of a fresh grout mix is to add more water, with potentially disastrous consequences for strength, impermeability and thus durability. Water-reducing chemical admixtures have been developed which allow the same workability to be achieved at a lower water/cement ratio with improved strength and impermeability. Normal water-reducing admixtures, or plasticizers, reduce the water/ cement ratio required for a given workability by 5–10%, whereas highrange water reducers, or superplasticizers, reduce the water demand by 15–30% for the same workability.

Water reducers are chemicals which are adsorbed onto the surface of the cement grains and onto the early products of hydration, in the same way as retarders. The effect of the adsorption is to cause all the surfaces to acquire the same surface charge, resulting in a more uniform distribution of the particles in the mixing water due to their mutual repulsion. Not surprisingly, the same chemicals which act as retarders also act as water reducers; those in common use are based on lignosulphonates and hydroxy-carboxylic acids and their salts. Similarly, they are most effective with cements low in alkali and aluminate and when added a few minutes after starting mixing.

Superplasticizers behave in much the same way as normal water reducers with the difference that they can be used in higher concentrations (up to 1–2% by weight of cement) without excessive retardation. They include sulphonated melamine-formaldehyde (SMF) condensates, sulphonated naphthalene-formaldehyde (SNF) condensates and special lignosulphonate polymers. They are all linear polymers with SO_3 anionic groups, at regular intervals along the backbone, which are responsible for the adsorption and the development of a negative surface change.

1.4.3 Other admixtures

Some reduced water demand of the fresh grout can be obtained with the mineral admixtures pfa and ggbs described above. Admixtures such as bentonite absorb water and can help to control bleeding and prevent segregation, but increase the water demand. Other fluid loss controllers include polymers like hydroxyethyl cellulose, which deposits a filter cake of very low permeability over moving water channels.

To eliminate bleeding completely the drying shrinkage of the cement paste must be compensated by an expansive admixture such as finely divided aluminium powder. This is attacked by the alkaline pore solution in the setting cement paste, releasing hydrogen gas which causes expansion. A potential risk is the hydrogen embrittlement of embedded highly stressed steel prestressing tendons.

Other ways of overcoming the effects of drying shrinkage involve the use of an expansive cement which can form large quantities of ettringite (see section 1.5.1) during the first few days of hydration while the paste is still relatively soft. Sources of the required calcium aluminate include high-alumina cement or excess C_3A which is added to the cement together with extra amounts of gypsum. Klein cement contains calcium sulphoaluminate, C_4A_3S which reacts with gypsum and calcium hydroxide to form ettringite and double its volume in a controlled manner.

Grouts exposed to the atmosphere are liable to damage by freezing and thawing of the pore water during cold weather. Damage can be much reduced or eliminated by an air-entraining agent which produces a foam of tiny discrete air bubbles in the cement paste during mixing which remains stable during setting and hardening. Provided the bubbles are less than 0.2 mm apart they provide space into which the freezing water can expand and thus prevent cracking. Air-entraining agents contain longchain molecules with a hydrophilic polar group at one end (normally a carboxylic or sulphonic acid group) and a hydrophobic group at the other (normally an aliphatic or aromatic group). The molecules align themselves around air bubbles with one end in the water and the other in air, thereby lowering the surface energy and stabilising the bubble. As with accelerators and retarders, the response with a given cement is sensitive to many factors and a trial mix under field conditions is needed to establish this. The entrained air will reduce the hardened strength of the grout, but the workability of the fresh grout is normally increased, allowing some reduction in water content which can, at least partially, offset the strength reduction.

1.5 Hydration processes in Portland cement

1.5.1 Chemical reactions and hydration products

The hydration mechanisms of Portland cement are complicated because five distinct primary solid phases are involved—C_3S, C_2S, C_3A, C_4AF and gypsum—and all of these contain impurities in solid solution or precipitate form. In addition, the minor phases, especially the alkali metal ions K^+ and

Na^+, play an important role. As a consequence the chemistry of cement hydration is still not completely understood, despite the enormous worldwide research over many years; for example, a major International Congress on Cement Chemistry is held every six years, the most recent being in Delhi (NCB, 1992). A standard text on the subject was published in 1990 (Taylor, 1990), and other recent publications provide a comprehensive state-of-the-art (ACS, 1989, 1991 and 1992). There is thus only space for a brief review here.

It is important to realise that in Portland cement all of the reactions may be occurring at the same time and the products of one can have important implications for the others.

C_3S is the major cementing phase and its reaction with water can be simplified to

$$C_3S+H \rightarrow C\text{-}S\text{-}H+CH \tag{1.1}$$

The main product, calcium silicate hydrate, is an amorphous gel of variable composition and so normally is written as C–S–H rather than, say, $C_3S_2H_3$; CH, calcium hydroxide or portlandite, is crystalline of fixed composition and occurs in the form of hexagonal plates where space allows.

C_2S hydrates much more slowly to form the same products but with proportionately less CH.

C_3A by itself hydrates extremely rapidly in water, which is why gypsum, $C\bar{S}$, is interground with Portland cement to avoid flash setting. In the presence of gypsum, C_3A hydrates to form calcium aluminate trisulphate:

$$C_3A+3C\bar{S}H_2+26H \rightarrow C_3A.3C\bar{S}.2H \tag{1.2}$$

If there is insufficient CS to react with the amount of C_3A in the cement, this trisulphate becomes unstable, transforming to monosulphate:

$$C_3A.3C\bar{S}.32H+2C_3A+4H \rightarrow 3C_3A.C\bar{S}.12H \tag{1.3}$$

The remaining C_3A forms calcium aluminate hydrates:

$$2C_3A+21H \rightarrow C_2AH_{8+}C_4AH_{13} \tag{1.4}$$

C_4AF behaves like C_3A but hydrates much more slowly, leading to differences in the early stages between OPC and SRPC or oilwell cements. Ettringite formed from C_4AF may contain both Al_2O_3 and Fe_2O_3; this compound is known as AFt, where the 't' stands for trisulphate. Similarly the monosulphate becomes AFm.

1.5.2 Heat of hydration and setting

The chemical reactions involved in hydration are exothermic and are conveniently studied for the first 24–48 hours using an isothermal conduc tion calorimeter. Figure 1.4 shows how the rate of heat output changes with

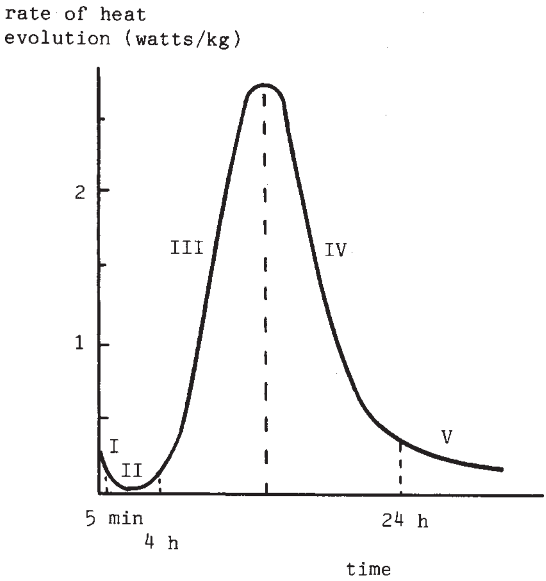

Figure 1.4 Classification of hydration stages of C_3S on the basis of heat evolution (Kondo and Ueda, 1968).

time for pure C_3S in water. An initial fast reaction, stage I, is followed by an induction period, stage II, during which only small quantities of heat are evolved. A major heat peak appears at the end of the induction period, stages III and IV, after which the rate of heat output decreases gradually.

According to Gartner and Gaidis (1989), stage I involves dissolution of the surface layer of the C_3S particles to form a metastable coating of C– S–H(m). This forms rapidly and is strongly bonded to the C_3S surface so as to inhibit further dissolution. Retarding admixtures can extend the induction period, but even then a slow reaction continues. The acceleratory period, stage III, involves the autocatalytic growth of stable C–S– H(s) in the original water-filled spaces, away from the C_3S surface which stabilises the C–S–H(m). Details of how stage II or even stage III are terminated are not yet clear. Dissolution, diffusion and deposition of hydration products are all involved and it seems most likely that diffusion of the slowly moving silicate ion out through the thickening layer of hydration product is responsible for the deceleration in stage IV. It seems likely that stage V is also diffusion-controlled but why and how this differs from stage IV is also not yet clear. Taylor (1985) suggested that a topochemical reaction may occur at the interface with C_3S or C_2S in which Ca^{2+} and Si^{4+} move outwards and H^+ moves inwards. By a series of atomic shuffles the silicate phase can be converted in-situ into C–S–H.

For pure C_3A and gypsum in water the change in the rate of heat evolution with time resembles that for C_3S shown in Figure 1.4. Stage I involves rapid dissolution of C_3A followed by the formation of a layer of gel and the precipitation of ettringite in the form of a diffusion barrier round the particles which results in stage II. Depending upon the fineness of the C_3A, equation (1.2) may go to near completion fairly quickly, leading to exhaustion of the gypsum. At this point the ettringite becomes unstable and the barrier reacts rapidly to form monosulphate (equation (1.3)) in stage III. C_4AF behaves generally like C_3A although the AFt phase appears to be very low in iron content. This suggests the formation of amorphous ferric hydroxide which would help to overcome the shortage of lime when C_4AF reacts to form AFt.

In Portland cements, both C_3S and C_3A should contribute to stage I although the major effect should come from the interstitial phase. Gaidis and Gartner (1991) were able to model the early heat evolution in terms of the interstitial phase for a range of cements for periods of up to 30 min, finding reasonable agreement with experimental measurements of actual temperature profiles in small specimens.

Most commercial isothermal conduction calorimeters require the specimens to be mixed outside the calorimeter and so the first heat peak in stage I is often missed. Results for a typical United Kingdom OPC appear in Figure 1.5, where the tail of stage I can be seen. This tail runs straight into stage III with no extended stage II induction period, unlike many other Portland cements. The second peak, labelled 2, has a distinct shoulder, labelled 3, which is followed by a lower broad peak, labelled 4, between 25 and 45 hours' hydration. Peak 2 is largely due to the hydration of C_3S forming C–S–H and CH and peak 3 is associated with the formation of AFt largely from C_3A (Dalgleish *et al.,* 1981).

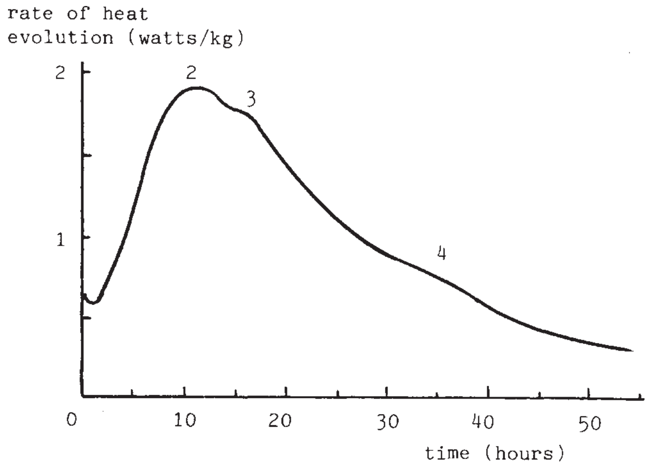

Figure 1.5 The rate of evolution of heat during hydration at 20°C of a typical UK ordinary Portland cement (Pratt and Ghose, 1983).

Table 1.4 Hydration data for a Danish Portland cement (4% gypsum, w/c=0.44, 20°C (Jons and Osbaeck, 1982)

Phase	Cement composition (%)	Degree of hydration (%)		
		1 day	3 days	28 days
C_3S	60	41	66	84
C_2S	22	–	–	~25
C_3A	8	19	46	68
C_4AF	10	–	9	26

Gaidis and Gartner (1991) suggested that peak 3 is due to the transformation of AFt to AFm when the supply of soluble calcium sulphate is exhausted, but this is not supported by X-ray evidence which suggests that the transformation occurs after 1–3 days of hydration, corresponding to peak 4 in the heat evolution curve. The degree of hydration of the individual phases in a Danish cement, measured by X-ray, are given in Table 1.4. There is a marked increase for C_3A between 1 day and 3 days corresponding to peak 4; C_2S and C_4AF both hydrate slowly, reaching only 25% by 28 days.

Setting of the cement represents the end of its workability, and this is often measured by a standard test of penetration resistance, i.e the pressure needed to push a flat-ended plunger into the paste. A penetration resistance of 0.5 N/mm² defines the initial set, and Figure 1.6 shows that this typically occurs

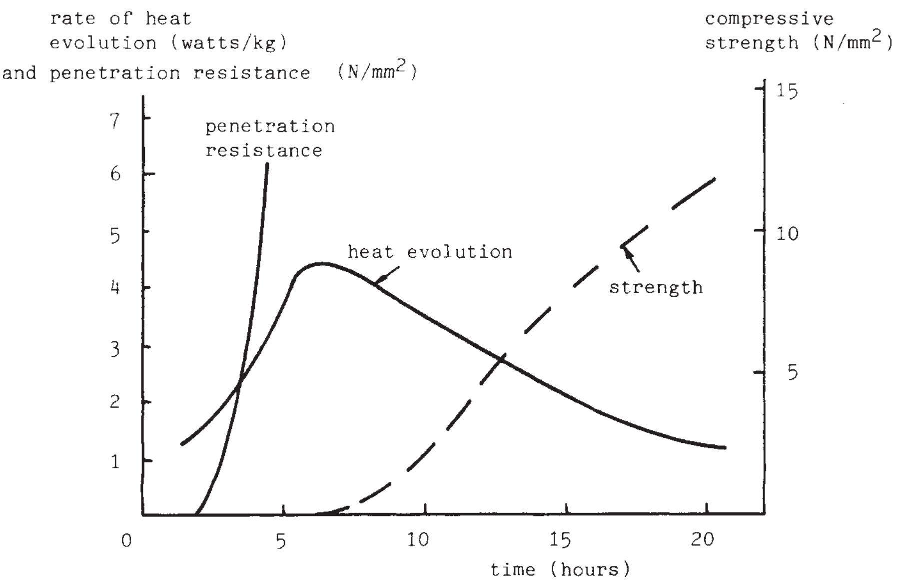

Figure 1.6 Measurements during the early stages of hydration of Portland cement grout (water/cement=0.4) (Domone and Thurairatnam, 1990).

shortly after the minimum in the heat evolution curve. The final set, equivalent to a penetration resistance of 3.5 N/mm², is partway up peak 2. The onset of strength measurable by a simple cube test occurs at or shortly after peak 2.

1.5.3 The effects of water/cement ratio and temperature

The effect of water/cement ratio on the hydration of cement is limited in the first 24 hours but is more significant thereafter. A modest acceleration of the onset of peak 2 is found with a decrease of water/cement from 0.5 to 0.385, due possibly to earlier saturation of the pore solution, and a slightly reduced peak height. Figure 1.7, a plot of combined water as a function of curing time, shows early acceleration, together with later marked changes in the kinetics of hydration which depend on the water/ cement ratio. The reductions are associated with the initial filling of the available water space with a hydration product rich in water.

The effect of increasing the temperature of hydration and curing is to accelerate the chemical reactions involved. Peak 2 in the heat evolution occurs earlier and the peak height can be increased very considerably. For large masses of grout the heat of hydration of the cement is sufficient to raise the temperature in an autocatalytic way. In particular, when using rapid-hardening Portland cement, ASTM Type III, which is both high in C_3S and finely ground, care must be taken to avoid temperatures much above 65°C in order to prevent thermal cracking and the possibility of delayed ettringite formation, causing expansion.

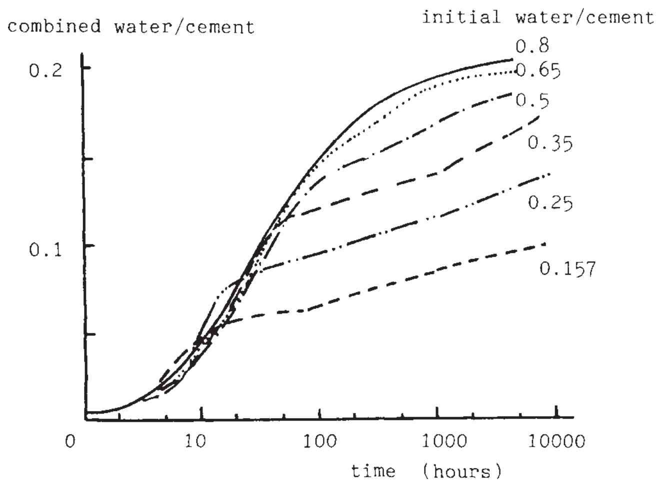

Figure 1.7 The effect of initial water/cement ratio on the increase in the ratio of combined water to cement in Portland cement paste (Taplin, 1959).

1.5.4 The effects of chemical admixtures

Calcium chloride, an extremely effective accelerator, increases the height of peak 2 and decreases the time at which it occurs. For example, a 1% addition by weight to Oilwell G cement at 20ºC resulted in a peak height of 10.5 W/kg about 3 h after mixing, compared to about 3 W/kg 10 h after mixing for the non-accelerated mix (Lota *et al.,* 1990). Lower concentrations of calcium chloride had a more modest effect, and greater concentrations are proportionally less effective. At 5ºC the effects were less marked, but also a more varied response was achieved.

For grout in contact with steel, the use of a chloride-based admixture can lead to an increased risk of steel corrosion, and therefore non-chloridebased accelerators are preferred. In general, effects of similar magnitude to those with calcium chloride can be obtained.

The use of sea water as mixing water at 20ºC produces a noticeable acceleration of peak 2 and an increase in peak height, as shown in Figure 1.8. Again, at lower temperatures the effect is less marked.

Superplasticizers have a retarding effect, which can be significant at high dosage rates. Figure 1.9 shows the effect of a sulphonated polymeric superplasticizer on a low-alkali sulphate-resistant cement mix with a water/cement ratio of 0.25. Mixing without the superplasticizer at this water/cement ratio was difficult but the addition of 3% by weight retarded peak 2

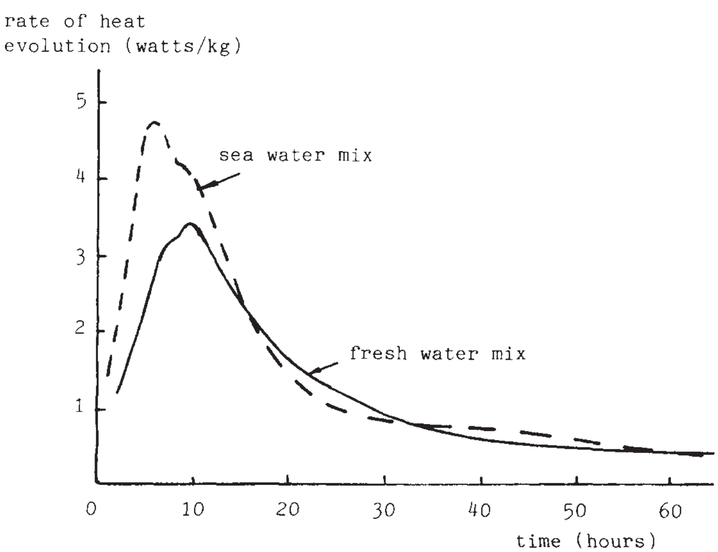

Figure 1.8 The accelerating effect of sea water when used for mixing Portland cement grouts (water/cement 0.45, 20ºC) (Pratt and Jensen, 1988).

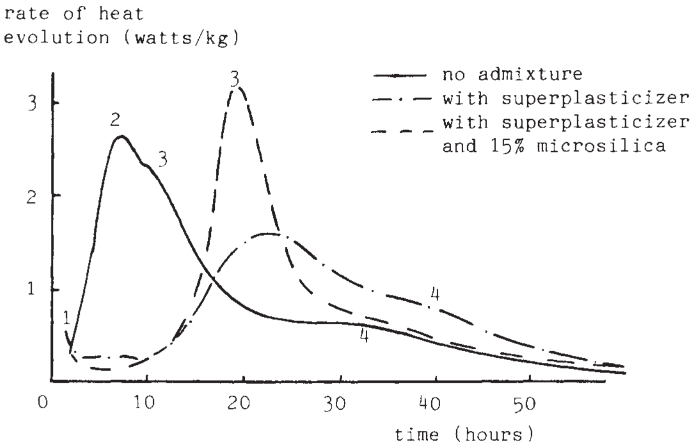

Figure 1.9 The rate of heat output during hydration of a low-alkali sulphate-resisting cement with a superplasticizer and microsilica (water/cement=0.25, 20°C) (Halse *et al.,* 1984).

by 15 h and nearly halved its height. Additions are normally limited to about 1–2% by weight. Replacing 15% of the cement in the superplasticized mix by microsilica enhanced and sharpened peak 2 while reducing peak 4.

1.6 The structure of hardened Portland cement pastes

Hardened cement pastes contain

- the remaining anhydrous material in the middle of the large cement grains
- solid hydration products including C-S-H, CH, AFt, AFm and other minor products
- porosity of varying shapes, sizes and degree of connectivity, and
- water, or pore solution, to an extent depending on the relative humidity.

The relative proportions of all of these change with time of curing and degree of hydration, but all of them must be included in any description of the hardened structure. At the simplest level of description, the water/cement ratio determines the spacing between the cement particles and thus the amount of space available for hydration products to fill. A water/cement ratio of 0.36 should leave just enough space for all the cement to hydrate and all the space to be filled; however, a water/cement ratio of 0.42 is necessary if there is to be enough water for all of the cement to hydrate. **Capillary porosity,** or remaining water-filled space, will be present at water/cement ratios higher than 0.36 after complete hydration, and the

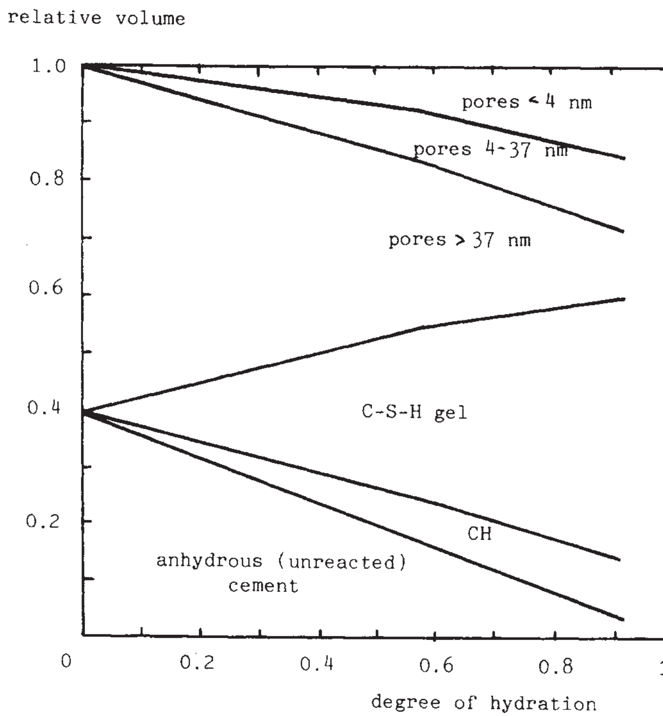

Figure 1.10 Changes in the volumetric composition of Portland cement paste with hydration (water/cement=0.47) (Patel *et al.,* 1989).

strength of the hardened paste falls and the permeability rises with increasing water/cement ratio above this figure. **Gel porosity,** the porosity within the C–S–H itself where the water is bound, does not depend on the water/cement ratio.

A convenient way of demonstrating the changing proportions of all these phases, including porosity, as hydration proceeds is shown in Figure 1.10. Here the change from anhydrous cement to solid hydration products is readily seen, as is the decrease in the coarse capillary porosity and increase in the fine porosity less than 37 nm in size. Such diagrams can be calculated making various assumptions about the kinetics of hydration or can be measured experimentally. They are especially useful to show the effects of water/cement ratio and chemical and mineral admixtures on the hydration process.

It is important to realise that the way in which the phases are distributed in space with respect to each other is probably as significant as the water/cement ratio in determining the useful properties of the hardened paste. A schematic picture of the setting and hardening process is shown in Figure 1.11. The early hydration products form on or near the surface of the

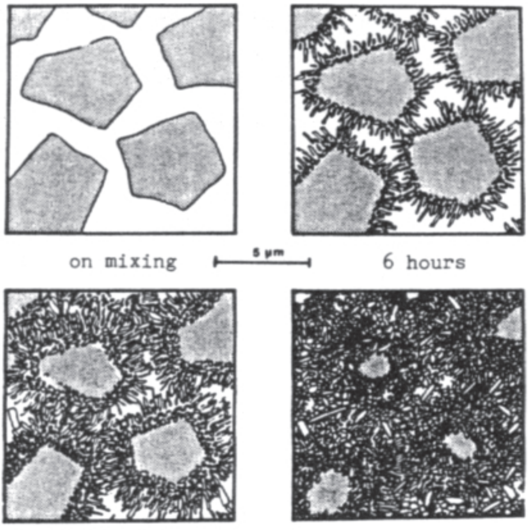

Figure 1.11 Schematic diagram of the setting and hardening process of Portland cement (Taylor, 1985).

cement particles, growing out into the water-filled space in stages I and II (Figure 1.12). Setting occurs after a few hours as the products formed around individual particles join up and grow together. After 1 day (Figure 1.13) the bond between the particles is strong enough to break them open, revealing a gap between the shells of hydration product and the cracked anhydrous alite core inside. Subsequent hardening and the development of strength involves the formation of more product both inside and outside the

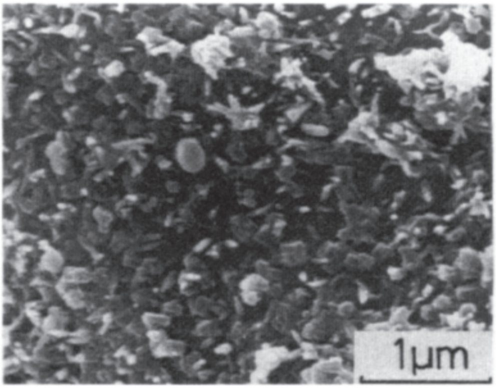

Figure 1.12 Scanning electron micrograph of Portland cement after 5 hours' hydration (Pratt and Jensen, 1992).

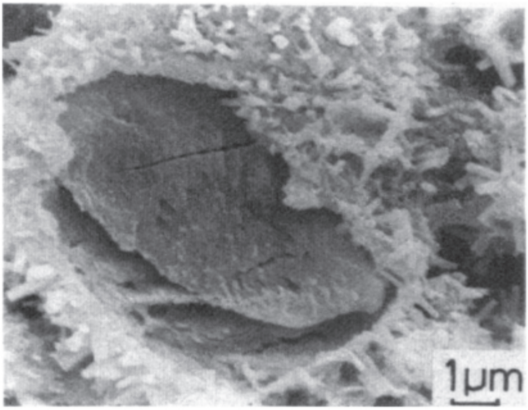

Figure 1.13 Scanning electron micrograph of Portland cement after 1 day's hydration (Pratt and Jensen, 1992).

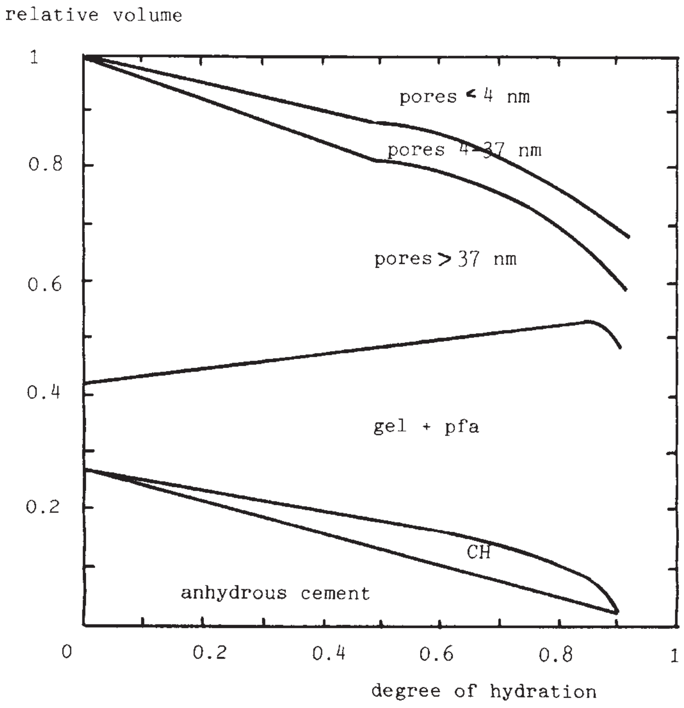

Figure 1.14 Changes in the volumetric composition of a Portland cement-pulverised fuel ash (pfa) blend with hydration (water/solids=0.47) (Patel *et al.,* 1989).

shells, so as to fill the available space. A more detailed description of the development of the microstructure is given by Scrivener (1989).

The effects of mineral admixtures (such as pfa) on the structure of hardened pastes is shown in Figure 1.14. With a low-lime (class F) pfa, the formation of additional C–S–H by the pozzolanic reaction is accompanied by a reduction in the amount of calcium hydroxide. Comparison of Figures 1.14 and 1.10 shows that a valuable by-product of this reaction is the reduction in the amount of coarse capillary porosity, which is replaced by gel porosity in the C–S–H.

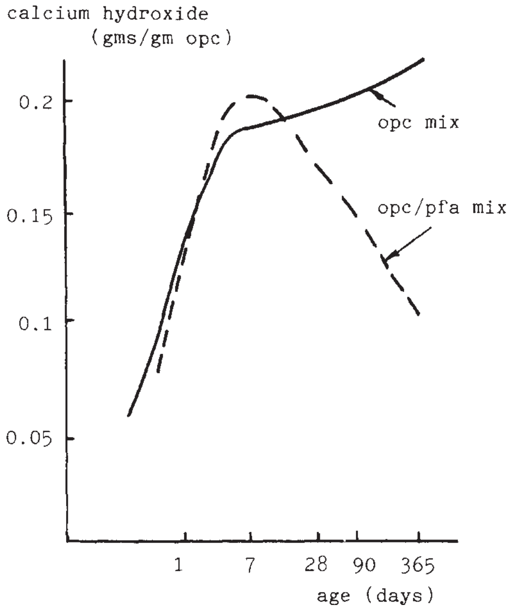

Figure 1.15 The effect of pfa on the amount of calcium hydroxide (from thermogravimetric analysis) in a Portland cement paste mixed with sea water (Jensen and Pratt, 1989).

With a sulphate-resisting Portland cement mixed with 30% pfa and seawater (Figure 1.15), the hydration is retarded initially by the aluminate ions from the pfa and then accelerated after 1 day so that more calcium hydroxide is formed than with the neat cement. However, after 4–5 days the amount of CH falls rapidly as it is replaced by C–S–H. A further benefit of pfa is that the aluminate-rich pozzolanic product is capable of binding chloride ions in the form of monochloroaluminate hydrates similar in structure to AFm.

1.7 The structure of Portland cement mortars

Mortars consist of cement pastes, fine aggregate and the interface region between them. This region typically extends some 30–50 μm into the paste and differs in composition and strength from the normal or bulk paste. Microhardness measurements show decreases of up to 50% across the region, and microstructural studies on a concrete show a sharp increase in porosity and decrease in the amount of unreacted or unhydrated cement on approaching the aggregate boundary (Figure 1.16). In the fresh mortar, large particles of anhydrous cement have difficulty in packing close to the boundary and the smaller ones hydrate out into the water space, leaving empty hollow shells behind them, especially at early ages. Figure 1.16 shows, after 70 days, more than 30% porosity greater than 500 nm near the

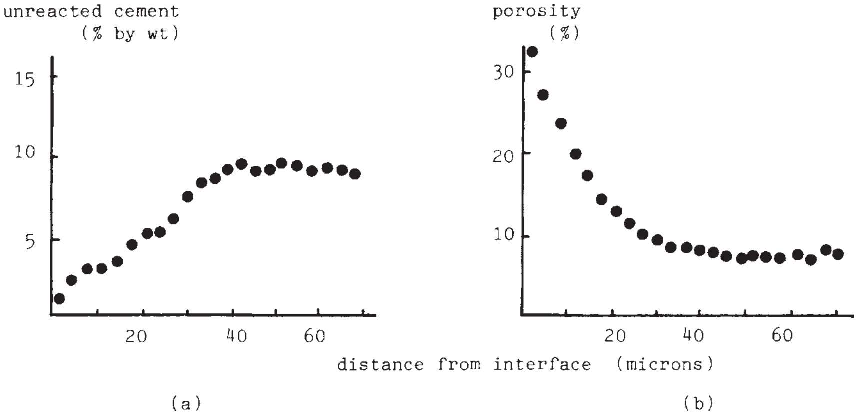

Figure 1.16 Microstructural gradient in the cement paste/aggregate interfacial region of a 10-week-old concrete specimen: (a) anydrous material; (b) porosity greater than 500 nm (Scrivener and Gartner, 1988)

boundary after 70 days compared with 8% in the bulk of the paste for the same specimens. Comparable figures after 180 days are 15–20% near the boundary and 5% in the paste.

The effect of adding 15% silica fume to this mix was to reduce the porosity gradient markedly to 8% porosity near the boundary and 5% in the paste at 180 days. The improved quality of bond resulting from this addition led to an increase in the 28-day strength of the concrete from 80 to 108 N/mm^2.

1.8 High-alumina cement (HAC, or calcium aluminate cement)

Where higher early strength than can be obtained with rapid-hardening Portland cement is required, high-alumina cement (also know as Ciment Fondu) may be used. This was developed by the Lafarge Company in France early this century as a sulphate-resistant cement. Its principal cementing phase is calcium aluminate (CA), hence the alternative name of calcium aluminate cement.

The oxide composition is typically 38–40% CaO, 38–40% Al_2O_3, 15–20% FeO and Fe_2O_3 and a small amount of SiO_2. It is made by melting together a raw mix of limestone and bauxite ores, which are low in SiO_2, at temperatures up to 1700ºC. The resulting phase composition includes CA, C_4AF, $C_{12}A_7$, ß-C_2S, C_2S, C_2AS or gehlenite, pleochroite and an aluminate glass. The cement is ground to a Blaine fineness of 300–400 m^2/ kg, and most of the grains are polymineralic. There are more coarse particles, up to 100 μm in size, than would be found in a Portland cement of the same fineness.

HAC sets at about the same rate as Portland cement but hardens much more quickly. Most of the heat of hydration is emitted within 24 h and the 1-day strength can be as high as that found after 28 days in Portland cement mixes (Figure 1.17). However, HAC is much more expensive than opc because

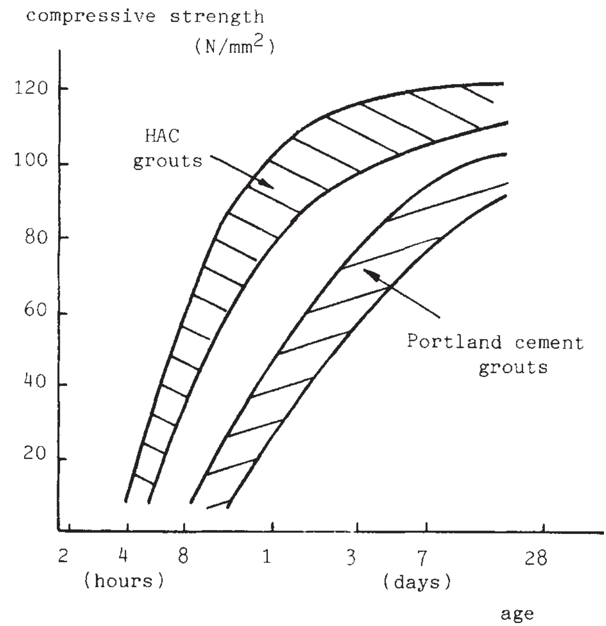

Figure 1.17 The range of maximum early strengths obtainable with high-alumina and Portland cement grouts (Domone and Thurairatnam, 1988).

bauxite ore is expensive, the kiln temperatures are higher and the product is very difficult to grind.

The products of the hydration of monocalcium aluminate (CA) depend on the temperature at which they are formed. At room temperature and below:

$$CA+10H \rightarrow CAH_{10} \quad (1.5)$$

Once setting begins, this is a very rapid reaction and most of the CA hydrates within 24 h.

At temperatures between 25 and 30ºC, the setting time increases and the reaction products become alumina gel, AH_3, and C_2AH_8. At even higher temperatures the cubic phase, C_3AH_6, replaces the hexagonal C_2AH_8 and CAH_{10}. Furthermore, the hexagonal phase initially formed at lower temperatures will convert to the cubic phase in the presence of water, the conversion rate being rapid at high temperatures and slower, but significant, at ambient temperatures. The conversion is accompanied by an increase in coarse porosity as the volume of the solid decreases by about 40%, together with a large decrease in strength. Fortunately, however, the conversion reaction releases a significant quantity of water which helps to hydrate the other phases in the cement, thus reducing the loss in porosity.

In the near adiabatic conditions in the centre of a large grout mass of typical water/cement ratio 0.4, the heat of hydration is sufficient to raise the temperature to over 100ºC within a few hours of casting. At higher water/cement ratios the loss of strength on conversion can become dangerously large and structural failures in HAC concretes have occurred (Neville, 1975). The design strength must be based on the long-term minimum strength after conversion and not on the 24 h strength.

Accelerators suitable for use with calcium aluminate cements include lithium carbonate and lithium citrate and the mineral admixture condensed silica fume. Sea water should never be used for mixing because the setting and hardening processes are adversely affected as a result of reactions with the chloride ions present in the sea water.

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2 Fresh properties of Portland cement grouts

S.A.JEFFERIS

2.1 Introduction

In most applications the fundamental requirements of a grout are that it can be poured or pumped into voids in a structure or the interstices of another material and that it will set as a coherent mass which fills the entire space. In service it is the hardened properties of grouts that are of importance and, in particular, the strength, durability and ability to inhibit the corrosion of steel. The fresh properties of grouts, just as the workability of fresh concrete, cease to be of importance once the material has set. However, the fresh properties of grouts are possibly even more important than the workability of concrete as the grouted region will usually be inaccessible to visual inspection. Thus, in grouting specifications much emphasis tends to be placed on flow behaviour, bleeding prior to set and the strength of the hardened grout. However, it should not be forgotten that the nature of the void to be grouted must also be considered and, in particular, the drainage conditions and the venting arrangements.

2.2 Cement types

Chapter 1 gives an outline of the principal types of Portland cement and cement replacement materials. Clearly the major application for most Portland cements will be the production of concrete and thus most specifications for cement focus on properties that are of concern to concrete. Few specifications for cements include reference to the behaviour of grouts made with the cement. Thus grouts made with cements from different works but to the same specification may show quite different properties, such as rheology and bleed.

2.3 Mix proportions

Before discussing the properties of fresh grouts it is appropriate to examine typical mix proportions for grouts and concretes. Table 2.1 sets out some basic mix ratios for two grouts and a typical concrete.

In concrete the cement will be the minor component by volume and the aggregate will be the major component both by weight and by volume. Hence in concrete the fresh properties such as workability tend to be dominated by the aggregate and water contents and changes in the chemistry or fineness, etc., of the cement have only a rather modest effect on the workability. However, in grouts the volumetric proportion of cement is much higher and changes in the cement will have a much more marked effect.

The first stages in the production of a grout will be the batching and mixing and therefore it is appropriate to consider these stages before considering the properties of fresh grouts.

2.4 Batching

The fresh properties of the grout, such as its rheology or the tendency of the solids to settle, will be strongly influenced by its water/cement ratio. The hardened strength will also be critically dependent on the water/cement ratio of the grout. It is therefore very important that grout mixes are accurately batched. Cement should be batched by weight and the water by weight or volume. If bagged cement is used, batch quantities should be designed to use a whole number of bags and thus avoid dispensing part bags. For critical mixes it may be necessary to weigh the bags as there is some tolerance in bag weight. For larger jobs the cement may be weigh batched or added to a fixed volume of water in the mixer until a predetermined grout density is achieved. Regardless of how the grout is batched, the grout density should be checked at the end of mixing as a quality control procedure.

The density of a grout ρ_g is related to the water/cement ratio as follows:

$$\rho_g=(1+w)/(w/\rho_w+1/\rho_c) \qquad (2.1)$$

where w is the water/cement ratio, ρ_c is the density of the cement grains (typically about 3150 kg/m^3) and ρ_w is the density of water.

For most applications the useful range of water/cement ratio for structural grouts (as opposed to those used in geotechnical engineering or for bulk void filling) will be about 0.3 to 0.45. For a cement with a density of 3150 kg/m^3 this will give a range of grout densities from 2105 to 1889 kg/m^3, as shown in Table 2.1. This is a relatively narrow range and it follows that if density is to be used as a check on batching accuracy then the density measurement device must be capable of good precision. The pressurised mud balance developed for testing oilwell cementing grouts (Rogers, 1963) is often used for density measurements on structural grouts. This instrument has a resolution in the order of ±10 kg/m^3, which corresponds to a resolution of about ±2% on the water/cement ratio—a resolution which may be marginal for grout control on sensitive projects. (It is necessary to use a pressurised mud balance as cement grouts will contain some trapped gas, as detailed in section 2.7.)

Table 2.1 Typical mix ratios for Portland cement grouts and concrete

	Grout		Concrete
Water/cement ratio	0.3	0.45	0.5
Aggregate/cement ratio	0	0	6
Weight per cubic metre of mix:			
Cement	1620	1303	329
Water	485	586	164
Aggregate	0	0	1974
Density	2105	1889	2467
Mix proportions by volume:			
Cement	0.51	0.41	0.10
Water	0.49	0.59	0.17
Aggregate	0	0	0.73

The density of the set grout may be slightly higher than that of the fluid grout as some settlement of cement grains may occur prior to set (with the expulsion of bleed water). Also, the hydration reaction of cement leads to an overall reduction in volume and, once the grout has begun to set, the bleed water may be re-absorbed with, possibly, some further water if this is available from the surroundings.

2.5 Grout mixing

Grout mixers are often classed as high shear or low shear mixers though there is no clear dividing line between the two types. Indeed it may be easier to judge the type of mixer by the grout it produces. Table 2.2 gives an indication of the comparative properties to be expected of high and low shear mixed grouts. This table should be taken as merely a guideline and not a definitive statement of behaviour as some mixers may accentuate particular mix properties.

Issues that appear to be important in differentiating high and low shear mixers include: mixer shaft rotational speed, mix head peripheral speed and

Table 2.2 Comparative properties of high and low shear mixed grouts

Property	Mixer	
	Low shear	High shear
Bleed	High	Low
Yield stress	Low	High
Plastic viscosity	High	Low
Internal cohesion	Low	High

the clearance between mix head and adjacent fixed surfaces and the energy input during mixing (mixer power per unit volume of mix multiplied by mixing time). In the laboratory, mixers with rotational speeds of 5000–20 000 rev/min are not unusual and a 1 kW motor might be used to power a mixer with a 1 litre batch volume and the mixing time could be of order 5 min. In the field the available motor power/unit volume may be much lower, perhaps 0.15 kW/litre. Field scale mixers with very high shaft speeds are available but tend to be very expensive and the mix heads are rather easily damaged. As a result they are seldom used. Many field mixers have a shaft speed of around 1400–2000 rev/min and consist of a mix unit which also acts as a pump and recirculates the grout through a reservoir tank mounted above the mix head. Field mix times tend to be kept as short as possible, commensurate with the time necessary to add the cement and to produce a homogeneous mix. Typically mix times, including cement addition, may be of order 3 min; indeed, the Fédération Internationale de la Précontrainte (1990) recommends maximum mixing times of 4 min for low speed mixers (about 1000 rev/min) and 2 min for high speed mixers (about 1500 rev/min). As a result of the lower power input per unit volume and the shorter mix time, the energy input to the grout in the field may be much lower than in the laboratory.

Low-shear mixers may be little more than agitators with a rotational speed of a few hundred rev/min. It is important that the blades agitate the full volume of the tank in order to ensure that there are no dead regions in which stiffening can occur. For some grout formulations high-shear grouts can be prepared by continuing low shear mixing for perhaps 20–60 min. Such extended mix times would be unsuitable in the field but can be of interest in research work.

Mechanical shear—that is, mechanical disaggregation of cement grains—may also be an important issue in high shear mixing. Jefferis (1985) gives an extreme example of the difference in set time of a special cement-Dead Sea water-attapulgite clay grout used to form a cut-off wall in the Dead Sea. This grout, when mixed in a high shear mixer with an enclosed mix head giving high mechanical shear, showed a set time in the order of 24 h. The same grout when mixed in a Venturi eductor, where there was hydrodynamic shear and at relatively high level but no mechanical disaggregation of cement grains, gave a set time in the order of two years. Clearly this was an extreme example with a sensitive grout formulation. However, it must be recognised that mixer type may influence set time and other grout properties in a manner which can be difficult to predict. This may be particularly important when scaling up laboratory trials to the full-scale situation.

Thus, whenever possible grout trials should be carried out in the full-scale mixers using the full-scale batch volumes and mix times. This can make trials relatively expensive and there may be a temptation to reduce batch volumes for economy. If this is essential then it may be appropriate to reduce the mix time so as to keep the power input per unit volume constant. However, reduction in mix time may be difficult to achieve as this may be dominated by the time required to add the mix solids.

In general, high shear mixers are to be preferred to low shear mixers as bleed is reduced. Also, low shear mixers may leave some lumps in the mix. These may be investigated by gently washing a sample of the grout through a sieve of opening perhaps 0.5–5 mm (the choice of sieve will depend on the application). A well mixed grout should show no retained solids.

2.6 Heat of hydration

The heat release on the hydration of cements is discussed in detail in chapter 1. However, it is appropriate to emphasise that hydration exotherms in setting grouts can be very significant. From Table 2.1 it can be seen that the weight of cement per cubic metre of grout is very much higher than the corresponding weight for concrete.

This high cement content can produce some unexpected results. For example, in concrete technology it is generally expected that replacement of some of the cement in a mix with ground-granulated blast furnace slag will normally produce a reduction in the hydration exotherm. However, if the temperature of the mix exceeds perhaps 35ºC, the exotherm can be more severe. Thus for grouts which may have a high initial temperature as a result of the energy input during mixing and a high total cementitious material content, slag replacement can lead to enhanced hydration exotherms.

The heat of hydration of a Portland cement may be in the order of 300 kJ/kg at 7 days when cured at 20ºC. For a grout of 0.40 water/cement ratio hydrated under adiabatic conditions such a heat release could lead to a temperature rise of 120ºC. While conditions will never be truly adiabatic, temperature rises in excess of 100ºC have been regularly recorded in large grout masses.

Littlejohn and Hughes (1988) recorded a peak temperature of about 110ºC at 40 mm from the external surface of a 2.1 m^3 fabric formwork grouted pipeline support. The grout mass was cured under water which was held at 5–7ºC. This peak temperature was achieved at about 5 h and did not begin to decay for a further 3 h. To achieve the recorded temperature rise would have required a heat release in the order of 250 kJ/kg under fully adiabatic conditions. The recorded temperature peak is thus an indication of both the substantial acceleration of the hydration reactions that occurs when conditions allow the temperature to rise and the potent insulating properties of even a small thickness of grout.

Such exotherms may lead to cracking on cooling and must be carefully controlled. In principle the exotherm could be reduced by chilling the mix ingredients and, especially, the mix water as this will have a higher thermal capacity than the cement powder. However, the energy input during mixing and the heat released from the cement during the mixing period is generally such that the cooling of mix ingredients has a trivial effect on the temperature of the grout discharged from the mixer. Thus, it is more effective to chill the grout after mixing. However, chilling non-Newtonian fluids is much more difficult than Newtonian liquids as the yield stress inhibits convection within the fluid mass. Thus, any cooler will need to be of markedly larger area than that which would be necessary for a comparable but Newtonian fluid. Ideally scraped-surface chillers should be used where the grout is continuously scraped from the cooler surface and replaced by fresh material. However, simple water jackets can be effective and the author has used such jackets in the laboratory though, as noted, the cooling rate was limited.

High shear mixing will lead to a higher discharge temperature for the mixed grout than low shear mixing. However, the grout may have a slightly lower subsequent heat release as some hydration occurs in the mixer. Page (1991) investigated the effect of prolonged high shear mixing on the heat release from a 0.4 water/cement ratio ordinary Portland cement grout. He found that increasing the mixing time from 2 to 15 min reduced the time to the first hydration peak from about 5.5 to 4.5 h but reduced the total heat released at 72 h from 315 to 285 kJ/kg. This is an interesting result and shows that some significant hydration must have occurred in the mixer.

2.7 The three-phase system

A freshly prepared cement grout may appear to be a homogeneous single-phase material. However, the grout is actually a complex three-phase system consisting of solid, liquid and gas phases. While the grout is fluid there will be a tendency for phases to segregate because of their different densities, though this segregation will be inhibited by the viscosity and gel strength of the grout. Admixtures which affect the rheology of a grout may markedly influence the tendency to segregate. For example, a superplasticizer which tends to reduce both viscosity and gel will promote the release of trapped air (though it should be noted that some superplasticizers can promote air entrainment during mixing and thus the overall effect of a superplasticizer on air entrainment is difficult to predict). However, the reduction in gel and viscosity may also promote the settlement of cement solids, i.e. bleeding.

The gas phase may be derived from air trapped within the cement powder when it was added to the mixer (if none of this gas escaped then the air content of the mix might be over 25% of the total volume), air entrained during the mixing process, gas adsorbed on the solid surfaces of the cement, which is released on wetting, and gas deliberately introduced via gas-generating admixtures. If no gas expansive admixture is used the gas phase usually may be effectively eliminated by pressurising the grout to the order of 100 kPa (a head of about 5 m of grout). Thus, in many applications the effect of gas may pass unnoticed.

2.8 Grout porosity

It is important to be aware that the multiphase system of many grouts is not intrinsically stable. In a saturated bed of sand the water might occupy perhaps 20–35% of the total volume depending on the grading of the sand and how it was placed/compacted. For grouts of water/cement ratio 0.3–0.45, the volumetric fraction of water will be in the order of 49–59%. Thus, if the cement were non-reactive and did not hydrate and set, the system would settle very substantially. For example, if grouts prepared at a water/ cement ratio of 0.3–0.45 settled to a porosity of 30% then the bleed water expelled would amount to 27–41% of the initial grout volume. While such bleed is rarely observed in practice it is important to keep in mind that fresh grouts have the potential to lose substantial quantities of water and that significant losses may actually occur if short drainage paths exist (see equation (2.3)) or if the grout is subject to pressure filtration.

2.9 Volume reduction due to settlement of solids

Bleeding of fresh grouts is apparent as a layer of water which develops on the surface of the grout. If there were no reaction between the cement and the water then the volume of this bleed water would be equal to the reduction in volume of the underlying grout mass. This is approximately the situation in the early stages of cement hydration but at later stages the hydration reactions lead to an overall reduction in the volume of the system with the result that some of the bleed water is re-absorbed into the grout mass. From a structural stand-point it is the final volume of set grout which is important and not the volume of bleed water generated. Thus, when studying bleed, attention should be focused on the loss of solid volume and not the volume of bleed water generated. It is also important to keep in mind that, as shown in section 2.8, grouts have a high porosity and thus a high capacity for bleed but only part of this bleed normally occurs as the process is usually stopped by the development of set (the set strength required to stop bleed will be quite small but will be slightly sensitive to the height of the grout column and thus the effective stress acting on the developing grout skeleton).

While considering the loss of solid volume it is appropriate to note that the term ‘syneresis’ is often used for the reduction in solid volume of chemical grouts such as silicate-ester or silicate-aluminate systems. However, syneresis is a process distinct from bleed. It is a three-dimensional shrinkage of the grout mass which, with certain systems (especially high water content low strength grouts), may continue for years after the grout has set. Syneresis has its origins in the internal chemistry and microstructure of the grout whereas bleed is driven purely by the density difference between cement grains and water. Hence, bleed in cement grouts is stopped by setting but syneresis in a chemical grout is not. Bleed is of considerable significance for the following reasons.

1. The loss of solid volume will leave voids within the grouted space which may reduce the effectiveness of the grout and allow corrosion of exposed tendons, etc.
2. If the upper grout surface is not horizontal (for example, grout in an inclined duct) the grout mass may slump to fill the bleed void. Such slumping creates shear planes within the grout which are not healed by the continuing hydration.
3. If the expulsion of water from the mix is rapid it may lead to channels being forced through the grout mass. Again these channels will not be closed by continuing hydration and will remain as preferential paths for the ingress of aggressive agents, etc.

The traditional approach to the prediction of the settlement of solids from suspensions has been to analyse the problem as one of hindered settlement of the solid grains. The rate of settlement is thus estimated as a fraction of the settling velocity of the grains in free liquid. While procedures of this type have been successfully employed for some systems they have little predictive power when applied to the problems of the settlement of grouts. Jefferis (1988) took an alternative view and regarded the settlement of solids to be a diffusion type problem and applied the theory of self-weight consolidation from soil mechanics. This proved to be a very powerful tool as it not only enabled the shape of settlement curves to be modelled but also provided a framework within which bleed data could be scaled up and the effects of drainage into tendons, duct inclination, etc., could be assessed.

Doran (1985) gives a detailed account of the application of consolidation theory to cement pastes. A particular result from consolidation theory is that the rate of surface settlement during the early stages of settlement of a column of grout of height h is given by:

$$\mathrm{d}h/\mathrm{d}t=k(\rho_g-\rho_w)/\rho_w \tag{2.2}$$

where k is the hydraulic permeability of the settling grout skeleton to its mix water and ρ_g and ρ_w are as defined in section 2.4. This rate of settlement will remain constant until time, t_c given by:

$$t_c=0.15h^2/C_v \tag{2.3}$$

where C_v is the coefficient of consolidation of the grout. This is an intrinsic property of the grout which, in principle, may be determined from the settling behaviour but in practice measurements are complicated by set (Doran, 1985). However, the important result is that if the rate of settlement is constant and that settlement is stopped by set during the period of constant rate of settlement, then the total settlement will be the product of the time (measured after placement and prior to set) and the settling rate. From equation (2.2) it can be seen that the rate of settlement is independent of *h*, the height of the grout column, and thus the total settlement will also be independent of the height of the column. This is a somewhat unexpected result and it means that when scaling up results from laboratory tests, bleed should not be scaled with the height of the column but that it will be equal in magnitude to that which occurred in the laboratory. Thus a grout which showed 2 mm of bleed in a laboratory test would show 2 mm of bleed in a full-scale column. The only requirement is that the laboratory test column is sufficiently high that set occurs before the time t_c is exceeded. Typically, the necessary column height will be in the order of 100–400 mm. Thus bleed tests ought to be carried out at a series of column heights so that it can be confirmed that a sufficient height has been achieved to reach the limiting bleed. However, many procedures for the measurement of bleed require the use of relatively shallow grout containers. The measured bleed in such tests will be a much greater proportion of the column height than will occur with taller columns.

In the above application of diffusion theory it has been assumed that the drainage is only vertically to the upper surface of the grout. Many other drainage situations can be analysed and some of these are discussed in chapter 10. An interesting and special drainage situation can occur in overwet concrete or high water/cement ratio grouts. The rate of escape of bleed water may be such as to wash channels through the grout; that is, channelled bleed may occur. The channels may be initiated by some imperfection in the formwork or the duct or adjacent to reinforcement, etc., or yet again the movement of trapped air bubbles with the mass. The potential for such initiation can be demonstrated by analysing the pore pressure distribution within a fresh grout mass. It can be shown that at any point within the grout the pressure is almost sufficient to hydraulically fracture the grout and thus any small disturbance may initiate a bleed channel or self-induced hydraulic fracture.

Once formed a bleed channel will represent a preferred drainage path and velocities in the channel may be such as to carry fines from the setting material. Channelled bleed is thus often identifiable by the appearance of small 'volcanoes' of deposited fines on the surface of the grout. Figure 2.1 shows the surface of a 0.5 water/cement ratio grout which has suffered channelled bleed. The material forming the volcanoes is often very dusty and creamy white in appearance suggesting that it has a high lime content.

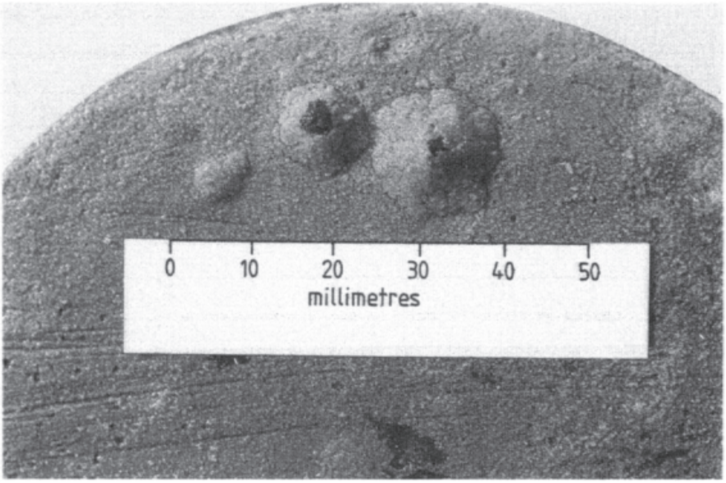

Figure 2.1 Surface of a 0.5 water/cement ratio grout showing channelled bleed.

Consolidation theory shows that at any time the degree of consolidation is a function of the square of the drainage path length and as the formation of bleed channels will reduce the average drainage path length within the grout mass it will also substantially accellerate bleeding. Thus a high water/cement ratio grout will not only have a high potential bleed due to its high porosity, but if channelling occurs a higher proportion of this bleed will be manifest before set occurs.

2.10 Pore pressures within a grout mass

If a pressure transducer were set in the side of the grout column so as to measure the fluid pressure it would initially show a pressure equal to the height of grout above the test point multiplied by the density of the grout. However, as settlement occurred and water was expelled from the grout the pressure would drop. Once the grains had reached a point where no further settlement was possible (either because they had reached a stable state of packing or because of set) the pressure would be just that due to a column of water. This dissipation of pore pressure is observed in grouts. However, the pressure continues to drop after it reaches the hydrostatic state. This is because of volume changes occurring within the grout as a result of the hydration reactions. It should be noted that if the fluid pressure in the grout column is below hydrostatic, then some stress must be carried by the cement grains and thus the grout must have achieved some set (or the grains reached a stable packing density just as if they were a cohesionless sand—but this is most unlikely). The appearance of pressures below hydrostatic therefore could be regarded as a definition of set.

2.11 Volume change due to cement hydration

In addition to the volume change due to bleeding a grout may be involved in a number of other volume change processes:

(a) removal of water by the hydration reactions of the cement
(b) increase in volume of the solid phase as a result of the hydration of the cement
(c) the action of expansive admixtures designed to cause expansion prior to set
(d) the action of post-set expanding admixtures.

For Portland cements:

The volume of reaction products is:
> the volume of anhydrous cement grains
< the volume of dry cement+the volume of reacted water

Thus the net effect of (a) and (b) is an overall reduction in volume. The absolute reduction in volume may amount to over 6 ml per 100 g of dry cement on full hydration. Figure 2.2 (from Holmes, 1993, private comm.) shows the volume of water drawn into a hydrating cement mass per 100 g of dry cement powder. It can be seen that water is still being drawn in at 100 h, long after the grout has set. In contrast, Figure 2.3 shows the settlement of the grout surface and thus the external volume change. It can be seen that this volume change is limited to about 0.9% and is effectively complete at about 6 h and there is then a small expansion (which may be largely thermal). The external volume change will be mainly as a result of bleeding provided that free water (which may be bleed water itself) is always available at the surface to replace that removed by the hydration reaction (which will be quite modest at 6 h).

Thus in general the external volume change due to hydration (as opposed to bleeding) will be very limited as the grout will have gained considerable strength before the hydration reaction is anywhere near complete. However, the situation is actually more complex and the boundary conditions may significantly influence the behaviour of the grout.

As already discussed the reduction in volume that occurs as the cement hydrates will lead to an overall reduction in the volume of the fluid mass. However, very soon some structure will develop so that the grains are no longer

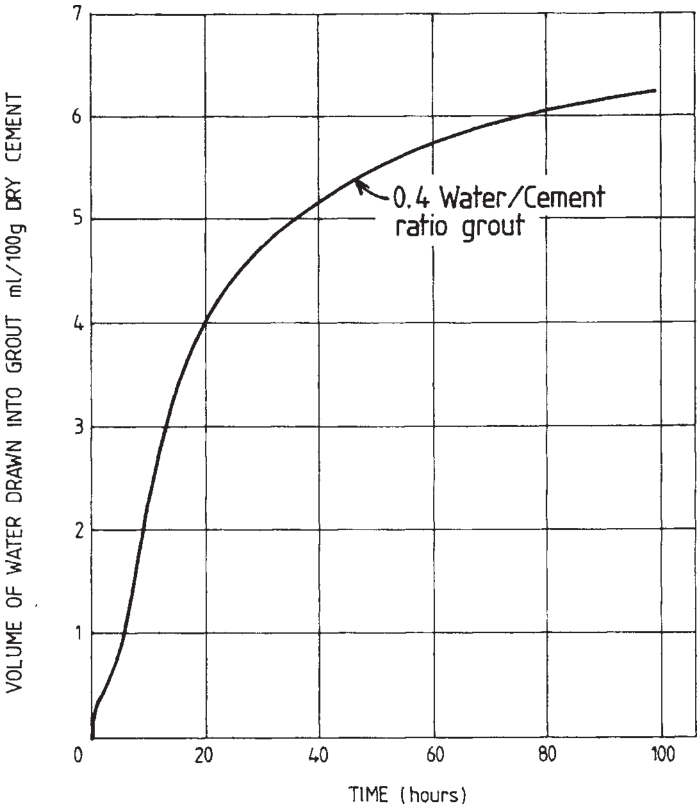

Figure 2.2 Volume change on hydration (water drawn into hydrating grout).

free to move but are linked by interparticle bonds. Further reduction in volume will then lead to the development of negative pressures in the pore fluid between the cement grains. This in turn will tend to draw in water from the surroundings. If such water is available the negative pressures within the grout mass will be controlled by the rate of water removal and the hydraulic resistance encountered by the water moving into the grout mass. This resistance will depend on the depth within the grout and the permeability of the grout. The permeability will be a function of hydration time. Initially it may be relatively high, perhaps of order 10^{-6} m/s, but, in a mature low water/cement ratio grout, may be of order 10^{-13} m/s (Mansoor, 1983). Thus in the early stages of hydration (the first hours) the reduction in volume of the system may be easily compensated by water drawn in from the surface, provided this is available, without any marked reduction in pore pressure. If there is no free water at the grout surface air will tend to be drawn into the grout mass. However, before this can occur, the negative pressures within the

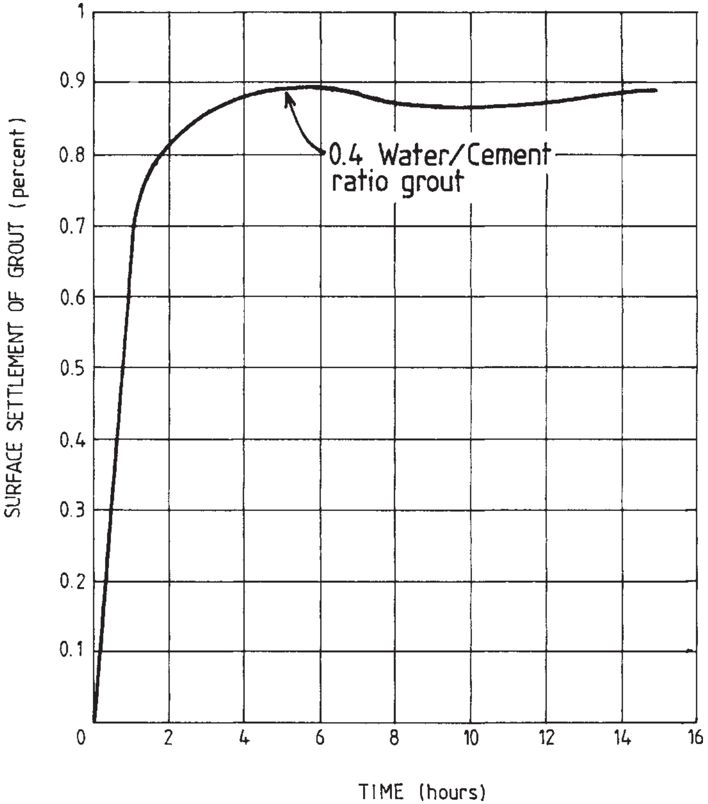

Figure 2.3 External volume change calculated from surface settlement.

grout mass must be sufficient to overcome the capillary pressure, Δp, across the menisci that will form at the surface between the cement grains. This pressure will be controlled by the spacing between ther cement grains thus:

$$\Delta p = 2\sigma \cos \theta / r \tag{2.4}$$

where σ is the surface tension of the pore fluid, θ the angle of contact and r the radius of the meniscus. Thus the minimum pressure before air can be drawn in will be a function of the interparticle spacing and, hence, the fineness of the cement and the water/cement ratio of the grout. The pressure reduction may be such as to reduce the pressure within the grout mass to the vapour pressure of water or lower if cavitation does not occur within the grout. Clearly the negative pressure will induce compressive stresses in the grout and thus tend to produce shrinkage.

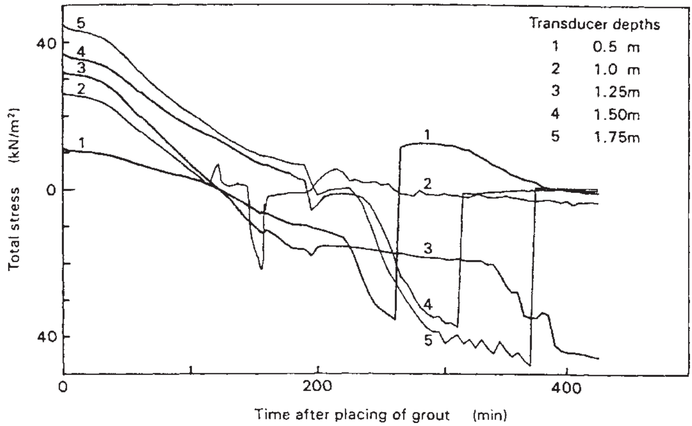

Figure 2.4 Pore pressure distribution as a function of time for a 2 m column of 0.4 water/cement ratio grout.

At depth in large grout masses or tall grout columns the condition at the surface may be of less significance so that pore pressure is controlled by the rate of volume change and the elasticity of the grout skeleton. Thus the development of negative pore pressures within a grout mass will depend on:

(a) the rate of reduction in volume of the grout system
(b) the surface condition—water available/water not available
(c) the hydraulic resistance of the grout mass and thus the depth within the grout mass and its permeability
(d) the rate of strength development in the grout and the elasticity of the grout skeleton.

The interplay of these factors is complex and it is very difficult to predict the time at which negative pressures will begin to develop and the magnitude they will attain. Figures 2.4 and 2.5 show pressure and settlement measurements on a 2 m high column of 0.4 water/cement ratio grout. The following general trends can be identified:

(a) an initial reduction in pore pressure as a result of consolidation, the dissipation taking longer at greater depths
(b) a period of more rapid pore pressure reduction as the pressures drop below hydrostatic (shown as zero pressure)
(c) sharp increases in pressure which for the transducer at 0.5 m depth in the grout (i.e. nearest the surface) is mirrored by a sharp settlement at the surface.

The shape of the settlement time curve suggests that the pressure relief is as

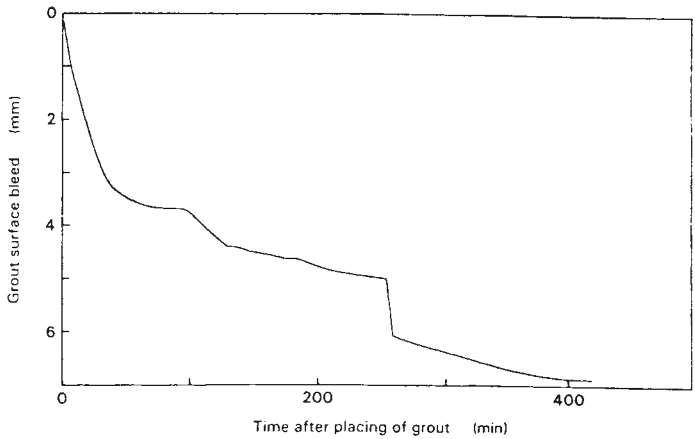

Figure 2.5 Settlement as a function of time for a 2 m high column of 0.4 water/cement ratio grout.

a result of cracking of the grout skeleton and that cracking may have occurred on more than one occasion. If, in a grout mass, cracking is to be limited then it is important that water should be available to the hydrating grout and thus the surface must be water flooded and the grout depth must be limited. Clearly limiting grout depth is seldom practicable and thus some cracking is inevitable in systems where there is a volume change on hydration.

2.12 Filtration

The simplest form of water loss from a grout is bleed as a result of settlement of the cement solids and accumulation of water on the surface of the grout. However, water loss also may occur as a result of drainage into tendons or leaks in the grout containment system. These situations also can be analysed using the theory of consolidation introduced in section 2.9. However, the author is of the opinion that consolidation theory is inappropriate to situations where the stresses induced as a result of drainage or as a result of externally applied pressure (in duct grouting it is normal practice to pressurise the duct at the end of grouting) are large compared with the gel strength of the material. In such situations the material may ‘crush’ and form a filter cake. The result of this is that rather than a gradual change in solids concentration through the grout mass there will be a sharp distinction between the bulk material and a filter cake. In soil mechanics the applied loads are seldom sufficient to cause a filter cake to form. However, in certain circumstances a cake will form. For example if a clay is mixed with a large amount of water and then pressurised a filter cake will form (this occurs with the bentonite clay slurries used in slurry trench excavations). However, if the same clay is mixed with less water and then pressurised it will follow the diffusion (consolidation) model. Both processes give similar trend curves for the rate of settlement or water expulsion etc. and thus when back-analysing data it is not possible to determine whether the internal process was one of diffusion or filter cake accretion. To determine which process dominates it is necessary to deter mine the pore pressure profile or solids content profile during the

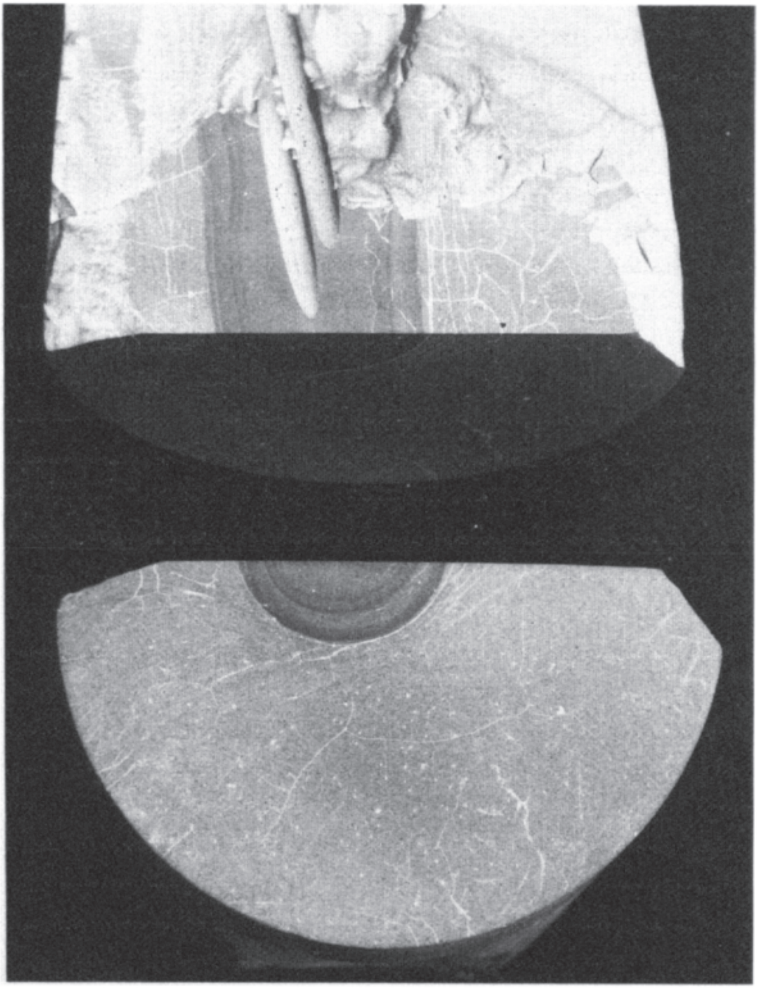

Figure 2.6 Grout from around a prestressing strand showing a dark band of filter cake.

settlement process. When considering external effects such as the rate of surface settlement it may be acceptable to use either model but that which is actually most appropriate will depend on the applied pressure and the strength of the material. Strictly it is incorrect to use either model alone as they represent extremes of behaviour and in practice real materials will show behaviour intermediate between the two models.

Filtration will be important when grouting voids which are not watertight or if there are leaks or if drainage occurs into prestressing strands. There is no standard test for the filtration properties of structural grouts but the American Petroleum Institute pressure filtration rig which was designed for oilwell drilling fluids and oilwell cements can be used to good effect (Rogers, 1963).

An interesting example of filtration is the filtering of water into the prestressing strand. In tall vertical ducts water may spurt from the strands as a result of the differential pressure between grout and water. In horizontal or near horizontal ducts water may be seen dripping from the strands at the anchorages. It should be remembered that any water lost from a duct after the completion of grouting will be leaving a void somewhere in the system.

In hardened grout the effect of filtration can be seen in the reduced water/cement ratio of the grout adjacent to the strand. This water/cement ratio can be quantitatively determined by laboratory testing but a qualitative indication of the extent of the filter cake can be obtained by examining the colour of the grout. A reduction in the water/cement ratio leads to a darker coloured grout. Thus, if filtration occurs into a strand an annulus of darker grout may be formed around the strand. Figure 2.6 shows a piece of grout broken from around a single prestressing strand in a 75 mm diameter vertical duct. The specimen came from a depth of about 1 m in the grout column.

2.13 Grout rheology

There is an enormous literature on cement paste, concrete and grout rheology. The work underpinning this literature has been carried out for a number of reasons including:

(a) rheological measurements may be used as an indicator of microstructural changes occurring in hydrating cement paste
(b) the investigation of the effect of admixtures
(c) the development of rheological test techniques
(d) the development of cementitious systems that are easily pumped or placed.

In this text, the main concern for grout rheology is pumping and placement. Unfortunately this is an area which has been rather less researched than some of the other areas. Tattersall and Banfill (1983) provide a substantial volume of data on the workability of concrete and also useful data on the rheology of cement pastes. In the oil industry there is a substantial literature on oilwell cementing but much of this is particularly concerned with the high temperature and pressure conditions that may occur down-hole. Also the standard rheological test instrument of the oil industry, the consistometer, is rather rarely used in the construction industry. Thus there can be difficulty in relating oil industry data to construction industry practice.

Grouts are complex time and shear history dependent, non-Newtonian fluids. They are also multiphase systems (see section 2.7). Furthermore, certain test or placement procedures may lead to separation of the liquid and solid phases. Thus in pipe flow or in a rotational viscometer a lubricating layer may form at the moving boundary to produce a more fluid surface layer and a stiffer bulk material. This behaviour will be in addition to the normal plug flow behaviour of a non-Newtonian fluid whereby there is no flow in the regions where the local shear stress is less than the yield stress of the grout. Thus, in a concentric cylinders viscometer with a rotating bob and a stationary cup the region near the bob may flow but near the wall of the cup the fluid may be stationary. Fluid then may be drawn from the stationary region into the fluid region so that the two materials become independent rheological bodies and attempts to calculate the position of the solid/fluid interface from the properties of the material in the fluid region will fail. Some authors have used grooved bobs in an attempt to reduce segregation and also slip (differential velocity at the interface between the fluid and the rotating bob of the viscometer). However, the pumping action of the grooves may be such as to actually promote segregation. Sakuta (1981) tested grouts of water/cement ratio 0.3–0.5 using a bob with 0.1 mm grooves and found that the torque on the grooved bob was less than that on a corresponding smooth bob. He also showed that the thickness of the fluid region adjacent to the bob may be extremely small. For this he used a number of sleeves that could be inserted into the cup in order to reduce the thickness of the grout annulus. He found that the sleeves had no effect on the torque on the bob until the grout annulus was less than 2 mm thick.

The effect of water migration and thus lubricating layers is particularly important in pipe flow of grouts and may substantially reduce the pumping pressures. However, a number of cautions are appropriate. The effect of the lubricating layer may be more pronounced in small-diameter pipes than in large, and if there is any leakage of fluid from the pipe the lubricating layer may be lost and the combination of deposition of solids by filtration at the leak and loss of lubrication may lead to rapid blocking.

2.13.1 Rheological testing

There is an immense variety of rheological testing techniques and many industries have developed their own preferred (and often empirical) procedures for particular fluids. Cheng has done an immense amount of work on test procedures (see, for example, Cheng, 1985). He has also advanced the concept that the procedures should be selected on the basis of their ability to discriminate between acceptable and unacceptable behaviour. An important criterion for all rheological testing is thus to ensure that any test or quality control procedure that is adopted is a good indicator of the rheological property or properties that are important at full scale.

For grouts there are many possible test procedures, including: flow funnels (CEN draft standard prEN 445, 1992); Fédération Internationale de la Précontrainte (FIP, 1990), the plunger test (CEN draft standard prEN 445, 1992), the Colcrete flow trough (see section 10.6.5) and the Fann Viscometer (Rogers, 1963). These instruments may have quite different average shear rates and while a funnel test or the plunger test may be appropriate for duct grouting, the Fann viscometer may be more appropriate for testing grouts that will be used at higher shear rates. It should not be assumed that all instruments will rank all grouts equally. This can be highlighted by considering a simple Bingham fluid with an equation of state:

$$t=\eta_p\gamma+\tau_0 \qquad (2.5)$$

where τ is the shear stress, γ the shear rate, η_p the plastic viscosity and t_0 the yield stress. For a Newtonian fluid, τ_0 will be zero and η_p will be the Newtonian viscosity. The Bingham model is often used as a baseline for cementitious grouts, though the actual behaviour is much more complex and τ_0 and η_p will be not only time dependent but also dependent on the shear history of the test sample. Consider now the ranking of a grout prepared in a low shear mixer and a high shear mixer. Low shear mixed grouts can have a relatively low yield stress but high plastic viscosity, whereas a high shear mixed grout might show the inverse behaviour (see Table 2.2). A test which involves relatively high shear rate such as the Fann viscometer could rank the low shear mix as less fluid than the high shear mix, whereas a flow funnel might give the opposite ranking. Thus when testing grouts it is important to try and match the shear rate in the test device and the application. In theory an instrument that offers a wide range of shear rates would be the most useful, and indeed this is the case for research work. However, such instruments will inevitably be high cost devices and they may not be sufficiently robust for use on site. The normal practice on site is therefore to use a device in which the operator has no control of the shear rate—for example, a flow funnel.

While shear rate is an important parameter in grout testing there are a number of other important non-dimensional groups, including the Reynolds number and the Hedstrom number. These and other groups would have to be considered in any attempt to use scale models to predict the flow behaviour of grouts (see section 10.3.6). Consideration of the conditions for dynamic similarity shows that it is not possible satisfactorily to model grout flow at reduced scale. Thus grout trials in scale models will be of limited value and important trials ought to be carried out in ducts of full-scale cross-section. It may not be necessary to model the whole length of a duct but only critical sections.

The conclusion is that there is no universal test for grouts. When selecting test procedures it is important to assess the ability of any procedure to provide useful discrimination between acceptable and unacceptable grouts. On site, properly selected empirical tests may prove cheaper, more robust and easier use and to interpret than absolute tests, such as a rotational viscometer, which allow rheological parameters such as yield stress and plastic viscosity to be independently calculated. The corollary is, of course, that for new applications it may be necessary to develop new tests.

2.14 Admixtures

A wide range of admixtures is available to modify the behaviour of grouts. Some of these are highly specialised, such as those detailed by Annett in chapter 9. It is not appropriate to try to review all the types of admixture that are available; however, it is instructive to consider the effects of a few general types.

2.14.1 Superplasticizers

Superplasticizers can be very effective at increasing the fluidity of grouts, and in particular they can substantially reduce the gel strength of the grout thus greatly increasing its ability to penetrate crevices, interstices, etc. Bleed may need to be checked experimentaly for superplasticized mixes as the situation is somewhat complex. The increased fluidity may encourage bleed but this may be offset by the improved dispersion of the cement grains and thus the bleeding rate may not be greatly affected. However, some Superplasticizers can show a retarding action which increases the set time and, hence, the time during which bleed occurs. No general rules can be given and it will be necessary to test formulations individually.

2.14.2 Antibleed admixtures

At the simplest these may be viscosifying agents. One of the simplest viscosifiers for water is sodium carboxymethyl cellulose. However, this material is precipitated by the calcium ions in cement and thus is unsuitable. Non-ionic hydroxyethyl celluloses can be very effective. The combination of superplasticizer and viscosifying admixture can give most interesting results and, in particular, produce a viscous grout with little gel—that is, a grout which may flow rather slowly but has good penetrating power and will self-level, etc. The viscosity can be particularly useful when placing grout underwater where it may provide cohesion and reduce the tendency for the grout to disperse in water (Jefferis and Sarandilly, 1988).

2.14.3 Gas expansive admixtures

These admixtures are designed to induce some expansion in the grout prior to set. Typically the admixtures may be based on activated carbon or aluminium powder, although gas-loaded zeolites could find application.

Activated carbon can absorb substantial volumes of gas which may be released when the material is wetted. The gas release therefore may be relatively rapid after wetting and because the gas release is a desorption process the pressures which can be generated may be rather limited. However, the expansion process is useful and has found application in grouts for seating bridge bearings, etc.

Aluminium powder will react with the alkalies in cement to produce hydrogen gas. In theory 1 g of aluminium should produce 1.2 litres of gas and thus 0.003% of aluminium by weight of cement could produce a 5% increase in the free volume (the maximum permitted by the FIP, 1990) of a 0.4 water/cement ratio grout. As the gas is produced as a result of a strong chemical reaction, the generation process will not be significantly inhibited by the ambient pressure in the grout though the volume of gas generated will be influenced by this pressure. In principle, the gas release will start as soon as the aluminium comes in contact with strong alkali. However, the reaction may be delayed by surface treatment of the aluminium powder. This can be useful, as otherwise there is a risk that all expansion will be complete before the grout has been placed. The aluminium powder will need to be extremely fine and well dispersed in order to produce many fine bubbles rather than fewer larger bubbles. If large bubbles are formed they may burst and locally tear the grout—for example, in a vertical duct the grout mass may be separated into a number of discrete elements. Bursting will also release some of the gas and thus limit the expansion achieved.

As indicated, for any gas expansive agent the expansion per unit volume of gas released will be influenced by the ambient pressure in the surrounding grout. For example, if a gas-expanding grout partially fills a duct which is then closed off, the gas escape from the grout may lead to the pressure in the void always being comparable to the effective gas pressure in the grout and thus the grout may not expand to fully fill the void. Furthermore, the slow build up of pressure in the void has been observed to cause a compression of the gas within the grout mass, with the result that after an initial expansion the grout shows a slight contraction. If the duct is vented expansion will occur but the continuing release of gas may lead to channels being forced through the grout mass. Also, the movements induced in the grout mass may lead to some damage and loss of structural integrity. The conclusion is therefore that gas expansive admixtures must be used with great care and in small doses (perhaps 0.001% by weight of cement).

Venting arrangements also will need to be carefully addressed and trials may be necessary to optimise procedures. Just as with superplasticizers it may be advantageous to add gas expansive materials in conjunction with other admixtures, such as viscosifiers which develop internal cohesion and thus reduce the tendency of the gas expansion to burst the grout and so improve the amount of gas that is retained within the grout and hence its expansion.

A further benefit of gas expansion is that it will soften the grout system (see sections 2.10 and 2.11) and so may reduce the negative pore pressures that develop as a result of the cement hydration reactions. This may considerably reduce the potential for self-induced cracking of the grout mass. The gas bubbles may also act as crack arresters and thus gasexpanded grouts should be less brittle than pure cement/water grouts.

An objection often raised to hydrogen gas-generating admixtures is the possibility that the hydrogen may lead to embrittlement of post-tensioned prestressing strands. Nascent hydrogen (H^+), such as is produced by the action of acids on metals, certainly can have such an effect. However, it is held that the action of alkali on aluminium produces molecular hydrogen (H_2) which does not cause embrittlement. This subject is still a matter of debate though several national standards for grouting now permit the use of aluminium powder.

Because of the very small quantities to be added it may be expedient to dilute the aluminium powder with an inert filler in order to increase its volume and make it easier to disperse uniformly throughout the grout. Pulverised fuel ash can be used for this and indeed may be found useful as a diluent when adding other solid admixtures. For example, many viscosifying agents tend to lump if not added very slowly to a well mixed system. The problems of lumping can be eliminated by pre-dispersing in an inert powder.

2.15 Acoustic wave velocities

Acoustic wave techniques have been widely used in the non-destructive investigation of grouts and concrete and as a research tool when studying the hydration processes in cements. Unfortunately, a number of misconceptions remain in the literature and thus it is appropriate to devote a significant section of this chapter to such techniques. In concrete technology two general procedures are used for the measurement of acoustic wave velocities:

(a) measurement of time of flight of a sharp ultrasonic pulse using purpose-made transducers—a procedure that may be used for both fresh and hardened grouts
(b) measurement of the resonant frequency of small beams of the hardened material.

In some texts it is not made clear that method (b) is actually a procedure for the measurement of wave velocity and that fundamentally the only difference between methods (a) and (b) lies in their boundary conditions. For a compressive wave in bulk medium the propagation velocity, V_L is given by:

$$V_L = \sqrt{\frac{E(1-\nu)}{\rho_g(1+\nu)\ (1-2\nu)}} \qquad (2.6)$$

where E is the elastic modulus, ρ_g the density of the grout and ν its Poisson's ratio. For the material to behave as a bulk medium the lateral extent of the material around the transducers must be at least one wavelength of the propagated wave. Thus, for a fresh grout where the velocity might be of the order of 1400 m/s with a 30 kHz transducer the minimum dimension would be of the order of 100 mm. If the size of the sample is reduced so that the minimum dimension perpendicular to the direction of travel approaches about one-fifth of a wavelength (ie about 10 mm for the above conditions) then the velocity of propagation will be that for a rod:

$$V = \sqrt{E/\rho_g} \qquad (2.7)$$

In concrete technology, the time of flight method is seldom applied to thin rods but the resonant frequency method is used. For this a variable frequency oscillator is applied to one end of a rod or beam and the frequency adjusted until the beam is in longitudinal resonance. For a rod clamped in the middle the fundamental resonant frequency will be that at which the half-length of the rod is equal to a quarter wavelength of the applied signal (i.e. there is an antinode at each end of the beam). Hence, at resonance:

$$L = \lambda/2 = V/2f \qquad (2.8)$$

where f is the frequency of the applied oscillation, λ the corresponding wavelength in the grout specimen and L the length of the rod or beam. It thus follows that if transmission is to occur as in a rod, the length of the beam should be greater than 2.5 times its diameter—a condition which is seldom explicitly stated in published test procedures.

For dimensions intermediate between those for bulk and rod propagation the behaviour will be more complex and there can be no simple formula for the velocity, etc. Unfortunately, in published papers on acoustic velocity measurements, the sample dimensions are not always stated and thus literature values for compression wave velocities must be treated with some caution. A further caution is appropriate for the resonant frequency method. With some equipment, resonance is determined by seeking the peak amplitude response of a transducer mounted at the opposite end of the beam to the oscillator. It is preferable to follow both the frequency and the amplitude of the received wave. If the beam is not clamped exactly at the centre the two halves may oscillate at slightly different resonant frequencies. This will lead to a broadening of the resonance peak and thus uncertainty in the resonant frequency. If the received signal is monitored, such dual resonances may be easily detected and the clamping of the beam adjusted. It is of course also necessary that both the oscillator and receiver should have a flat frequency response with no resonances around the resonant frequency of the beam.

The resonance method is very versatile and can be used to determine flexural and shear resonances as well as compressive resonances. In shear the wave velocity, V_s, is always given by

$$V_s = \sqrt{G/\rho_g} \tag{2.9}$$

where G is the shear modulus of the grout. In flexure the situation is much more complex and the specimen dimensions may strongly influence the measured wave velocity.

Clearly the resonant frequency method can be applied only to solidified grouts. It is discussed here as it is an acoustic process but this is seldom made clear in texts on concrete where it tends to be treated as a distinct technique.

2.16 Prediction of acoustic wave velocities

If a fresh grout is assumed to be a fluid then the compressive wave velocity, V_g, is given by

$$V_g = \sqrt{K_g/\rho_g} \tag{2.10}$$

where K_g is the bulk modulus of the grout and may be estimated using a rule of mixtures from the bulk moduli of the solid and liquid phases. Thus:

$$1/K_g = x_c/K_c + (1 - x_c) / K_w \tag{2.11}$$

where K_c and K_w are the bulk moduli of the cement and water respectively and x_c is the volume fraction of cement in the mix which can be obtained from the water/cement ratio w and the densities of the cement, ρ_c, and the water, ρ_w, as follows:

$$x_c = (1/\rho_c)/(1/\rho_c + w/\rho_w) \tag{2.12}$$

The density of the grout also can be calculated from a rule of mixtures, as given in equation (2.1). Typically the density of the grout will be of order 2 and as most solids have higher compressibilities than liquids it must be expected that $K_g > K_w$. Thus:

$$V_g > V_w/\sqrt{2} \tag{2.13}$$

where V_w is the compressive wave velocity in water. It can also be shown that the velocity in the grout can only exceed that in water (i.e. $V_g > V_w$) if the water/cement ratio, w, is given by

$$w < 1/((\rho_c/\rho_w)\ (\rho_c/\rho_w - 2)) \tag{2.13}$$

and hence only if $w < 0.276$ for $\rho_c = 3150$ kg/m^3.

Figure 2.7 shows a plot of the ratio of the velocity in the grout to the velocity in water (V_g/V_w) as a function of the compressibility of the cement grains (the grain compressibility is presented as a multiple of that of water) for various water/cement ratios.

Thus, for all practical grouts the compression wave velocity in the grout, if regarded as a fluid, should be less than that in water but greater than about 0.7 times the velocity in water (see equations (2.13 and 2.14)). In fact the literature contains many reports of compression wave velocities in fresh grouts and concrete where the measured velocity is considerably less than that given by equation (2.10). Far too often these velocities have been accepted as a true representation of the behaviour of a cement-water system and on occasion quite elaborate hypotheses concerning the microstructural changes occurring within the hydrating grout have been constructed on the basis of such measured velocities and their development with time. The measured velocities are, of

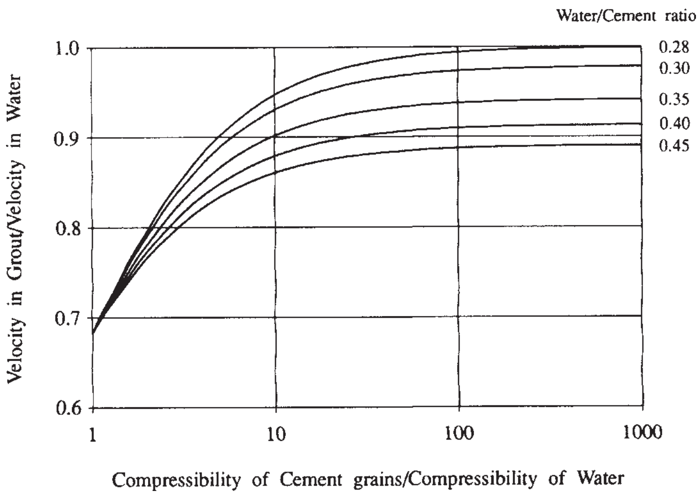

Figure 2.7 Compression wave velocity ratio as a function of cement grain compressibility.

course, not wrong but any assumption that they represent the behaviour of a simple two-phase cement-water system is entirely ill founded. The fact is, they are the velocities of a three-phase cement-water-gas system as discussed in section 2.7. This can be demonstrated by simply pressurising the system to dissolve any trapped gas or, alternatively, vacuum degassing it. After such treatment the velocity will be found to be close to that of pure water, thus implying that K_g, the compressibility of the grout, is about twice that of water as the grout typically will have a density about twice that of water. That the compressibility of the de-gassed grout is higher than that of water is reasonable, as the compressibility of the solid cement grains—just as for any solid—would be expected to be greater than that of the liquid.

The pressure needed to de-gas the grout is typically of the order of 30–70 kPa and thus it is apparent that there is relatively little air in the system. If equation (2.11) is extended to include the compressibility of air it can be shown that the ratio of the compression wave velocity in the three-phase system to that in the de-gassed system as a function of the volume fraction, x_a, of gas in the system is as follows:

Volume fraction of air in grout (%)	10	1	0.1	0.01	0.001	0.0001
Velocity ratio	0.02	0.06	0.2	0.5	0.9	0.99

Thus even a minute fraction of air will seriously reduce the wave velocity. Of course it must be accepted that to treat the grout as pure fluid is overly simplistic but the model gives a useful indication of the substantial effect of air in the system. This phenomenon is well understood by hydraulic engineers concerned with transient pressures in water distribution systems, but has not always been appreciated by cement technologists.

2.17 Shear wave velocity

A compressive wave can travel through solids, liquids and gases. However, shear waves can travel only in solids and will be rapidly and totally attenuated in any pure fluid. Thus in principle for a grout the time at which a shear wave can first be propagated through the material will represent the time at which it first takes on some set and begins to transform from a fluid to a solid. This time is of interest to those studying the hydration of cement and of considerable practical use to those involved in grouting works and also the stabilisation of toxic and radioactive waste with cement. In an extensive programme of research undertaken by the author using purpose-made shear wave transducers it was found that shear waves could be transmitted through grouts of water/cement ratio 0.35 from the time of first mixing. Therefore as regards shear wave transmission such materials are always solids. It would thus appear that there are some interparticle linkages from the time of first mixing of cement and water. Of course wet sand could transmit shear waves as a result of the frictional contacts between sand grains without the need for any interparticle bonding. However, the relatively high porosity of cement grouts (see section 2.8) suggests that there should be little frictional bonding in a fresh grout and that the interparticle linkages must be due to chemical bonding. Further evidence for this was obtained by trying to pass shear waves through a 0.35 water/cement ratio grout which had been killed by the addition of sugar. The wave was so heavily attenuated that no pulse was received over a path length of 250 mm (the same path length as had been used for the pure 0.35 water/cement ratio grout).

The compression wave velocity and shear wave velocity in any material are interlinked, and if one is known the other can be calculated if the linking parameter, the Poisson's ratio, is also known (and viscoelastic effects are ignored). Thus:

$$V_s/V_L = \sqrt{(1-2\nu)/(2-2\nu)} \tag{2.15}$$

where V_s is the shear wave velocity, V_L the compressive wave velocity and ν the Poisson's ratio. As Poisson's ratio can take values only in the range 0 to 0.5, it follows that

$$0 < V_s/V_L < \sqrt{1/2} \tag{2.16}$$

This relationship can be used as a check on the validity of acoustic wave data.

Unfortunately there are rather few published data on Poisson's ratio for cementitious systems generally and very little data on fresh grouts. For concrete, Poisson's ratio can be calculated from measurements of compressive wave velocity in the bulk material and in resonant beam tests, and some values for grout obtained in this way are given in Figure 3.17. For fresh grouts the author has determined values of 0.45–0.49 for grouts which have not been de-gassed. Substantial decreases must be expected as the grout sets and hardens, as indicated in Figure 3.17.

All acoustic transducers will generate both compressive and shear waves. However, in general they will have been designed preferentially to transmit/receive one type of wave. The attenuation of the waves in the test material and the losses at the material-transducer interfaces may change as the grout hardens. As a result care must be taken when interpreting time of flight measurements. The author has found that, on occasion, when using compressive wave transducers only a shear wave has been received for grouts a few hours old, even though at earlier and at later ages the compressive wave has been dominant.

Uzoegbo (1990) carried out a substantial study of acoustic wave propagation in grouts and concrete and Andrews (1991, 1993) is developing acoustic imaging systems to enable the investigation of detail in mass concrete, etc.

2.18 Concluding remarks

The purpose of this chapter has been to ensure that the reader is aware that cement grouts are not simple homogeneous materials but complex time-dependent multiphase systems whose properties will depend not only on the water/cement ratio of the mix but also on the nature of the cement, the type of mixer and the mixing time employed in the preparation of the grout.

However, despite the complexity of the grout system the basic laws of physics still apply. As grouts are often used to fill voids which cannot be inspected it is easy to adopt a philosophy of 'out of site, out of mind'. This will not work. Grouting can fail for very many reasons but unless the physics of the grout and the engineering of its placement are properly addressed the results will be out of contract! The physics of the grout have been introduced in this chapter. Engineering the placement requires not only an understanding of the behaviour of the grout but also of the void to be grouted—an issue which is seldom properly addressed in grouting specifications.

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3 Hardened properties of Portland cement grouts

P.L.J.DOMONE

3.1 Introduction

The hardened properties of grouts are obviously of paramount importance for their successful use. These will depend on the microstructure discussed in chapter 1, and a knowledge of this is important both in understanding the effect of the various parameters influencing the hardened properties, and when attempting to alter these properties to advantage. Control of the fresh and early age properties discussed in chapter 2 is important in ensuring that the grout can be handled and placed efficiently, and can subsequently achieve properties as close as possible to the full potential hardened properties. In this chapter we shall be discussing these potential properties.

Compressive strength is perhaps the most significant hardened property. Not only is it vital in itself in most structural applications, but it can be related to many other properties of cementitious systems, such as elastic modulus and durability, which are more difficult to measure directly. We shall therefore start by discussing strength, and then consider deformation, both load and environment induced, and finally durability.

As in other chapters, we shall consider the effects of additions of fine aggregate to the neat cement grouts. Although such mixes should strictly be classed as mortars, they are often referred to as grouts, and they have important applications in grouting practice.

3.2 Strength

3.2.1 Strength testing

Compressive strength is usually measured on either cubes or cylinders. UK practice is to use cubes, sometimes as small as 10 mm for neat cement grouts, but more commonly 50, 75 or 100 mm. Only the larger sizes can be used for mixes containing aggregate. Experience of testing many brittle materials has shown that, in general, larger specimens can be expected to give lower average strengths with smaller coefficients of variation. This has been demonstrated for concrete (Neville, 1981), but tests on neat cement grouts have shown generally insignificant differences between results from cube sizes varying from 50 to 100 mm (Harwood and Tebbett, 1981). The exception was strength measured at one day old, which increased with specimen size, probably due to increased temperature from heat of hydration effects.

It is more common outside the UK, and particularly in North America, to measure compressive strength on cylinders with a height/diameter ratio of 2. This is less convenient, as the end surface of the cylinders has to be prepared by grinding or capping before testing, but the measured value is closer to the true compressive strength as the restraining effects of end friction with the test machine platens are reduced towards the centre of the specimen. Tests on neat cement grouts (Harwood and Tebbett, 1981) have shown that the cylinder strength can vary between 80 and 90% of the cube strength. It is obviously necessary to define the method of strength measurement in any specification and when reporting results. Unless otherwise stated, all compressive strength data given in this chapter are from cubes.

The tensile strength of grouts, being typically 10% of the compressive strength, is of lesser importance, and not often measured. When values are required, direct uniaxial testing is difficult, mainly due to difficulties with gripping the specimen; as with concrete, indirect flexural or cylinder splitting tests are preferred. These give results higher than the direct tensile strength.

3.2.2 Factors influencing strength

For cementitious mixes, the principal factors influencing strength and the rate of gain of strength are:

- age
- water/cement ratio (or, more strictly, when using partial cement replacement materials, the water/binder ratio)
- curing conditions
- composition, fineness and type of the cement
- proportions of cement replacement materials
- the presence of admixtures.

For ordinary Portland cement (opc) mixes, typical effects of age and water/cement ratio are shown in Figure 3.1. The effect of increasing strength with decreasing water/cement ratio at any particular age was first recognised for concrete by Abrams in 1918, and cement pastes follow the same basic relationship:

$$f_c = k_1 + /k_2^{w/c} \qquad (3.1)$$

where f_c is the compressive strength, w/c is the water/cement ratio, and k_1 and k_2 are empirical constants.

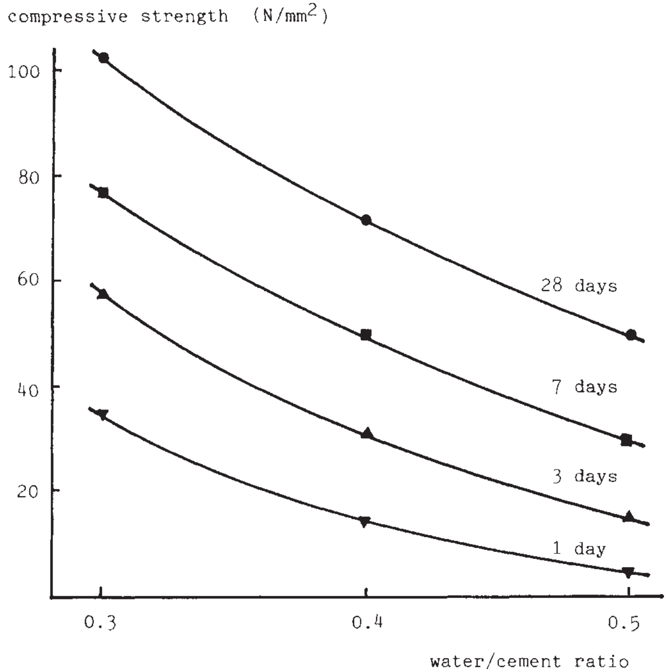

Figure 3.1 The effect of water/cement ratio and age on the strength of ordinary Portland cement grouts cured in water at 8°C (Domone and Thurairatnam, 1986).

The introduction of aggregate will generally lower the strength somewhat, due to cracking being initiated at the cement paste/aggregate interface, which is often a zone of relative weakness, as described in chapter 1. The magnitude of this effect will depend on the aggregate type and size as well as the binder properties.

After placing, continuing availability of moisture from the environment surrounding the grout is necessary to sustain the hydration reactions and prevent drying shrinkage; only then will the full potential strength be developed. The first few days are the most critical, and so good curing practice is particularly important during this time. Low ambient temperatures reduce the rate of strength gain, but can result in significantly higher long-term strength.

Cements with a higher C_3S content (see chapter 1) will gain strength more rapidly, as will finer ground cements; either or both of these effects are used to produce rapid-hardening Portland cement (rhpc). Such cements have typical specific surface areas (SSAs), as measured by the Blaine air permeability test, of 400–450 m^2/kg, compared to 300–350 $m^2/$ kg for opc. The effect of a wider range of fineness on early strength gain of grouts with a water/cement ratio of 0.5 is shown in Figure 3.2. As the fineness increases,

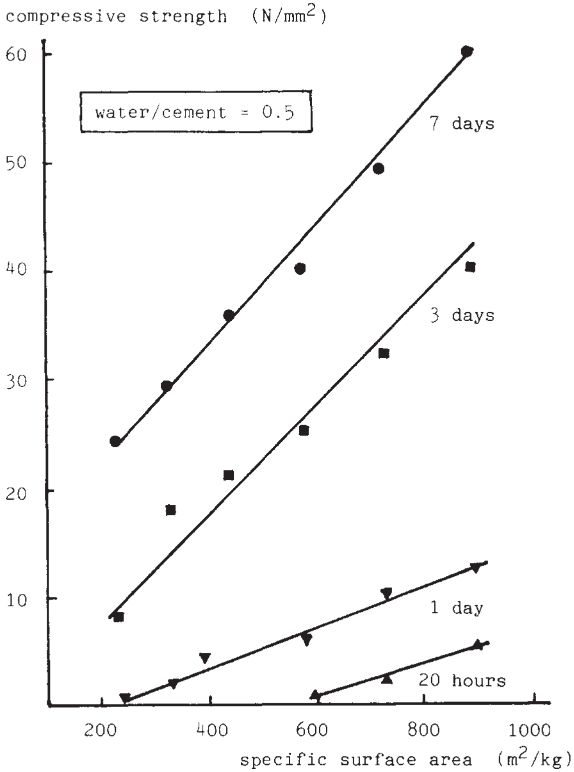

Figure 3.2 The effect of fineness (specific surface area) of Portland cement on the strength of grouts at early ages (Domone and Thurairatnam, 1988).

so does the water demand for fluidity, which imposes a practical limit on the use of the finest cements. For example, in designing a pumpable high early strength grout incorporating a superplasticizer to provide fluidity, Domone and Thurairatnam (1988) found that the highest strength for ages up to 3 days was obtained with a cement of SSA of about 580 m^2/kg and a water/cement ratio of 0.33 (Figure 3.3). At later ages, a grout with a cement of SSA 330 m^2/kg (i.e. in the normal range for opc) and a water/cement ratio of 0.25 provided the best performance.

The most common partial cement replacement materials (CRMs) are pulverised fuel ash (pfa), ground-granulated blast furnace slag (ggbs) and condensed silica fume (csf), sometimes called microsilica. The composition and properties of these materials and their effects on the hydration processes have been described in chapter 1.

Pfa and ggbs have similar size particles to opc, and their use as a partial substitute for the opc results in a slower rate of gain of strength, as shown in Figure 3.4. The reduction in strength is greater with pfa, which is a true

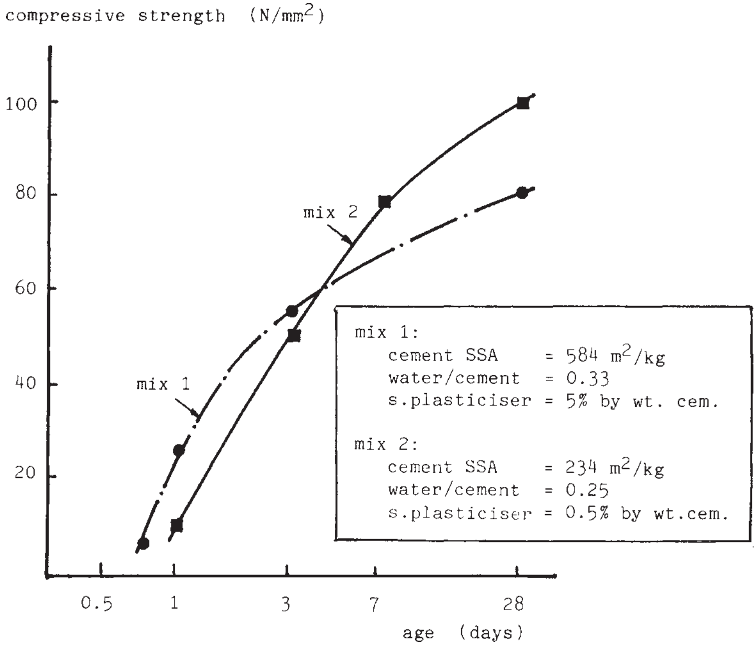

Figure 3.3 The strength gain of grout mixes designed for high early strength and sufficient fluidity for ease of pumping (Domone and Thurairatnam, 1988).

pozzolan; ggbs, which has significant lime content, is largely self-cementing, and therefore it can be used at higher replacement levels and still achieve levels of long-term strength comparable to neat opc mixes. With both of these materials, the rate of heat of hydration output is also slowed, an advantage when grouting thick sections.

Csf is also a true pozzolan, but with a much higher silica content than pfa and with much smaller particles than pfa or opc. The higher silica content means that it is only effective at relatively low cement replacement levels, up to about 20%. There is insufficient lime from the opc hydration to react with higher amounts of csf. The small particle size and high reactivity mean that the pozzolanic reaction in initiated after 2 to 3 days, much earlier than with pfa. In addition, there is evidence that the fine particles stimulate the opc hydration reactions, acting as nuclei for the deposition of hydration products. Tank (1987) has shown strength improvement at all ages from 1 day to 1 year for grouts containing 10 and 20% csf; the 10% csf grout generally had the greater strength enhancement. Comparison with the results of other less comprehensive tests indicates that this effect of csf level may not be typical, but it is clear that relatively low levels of csf are the most effective.

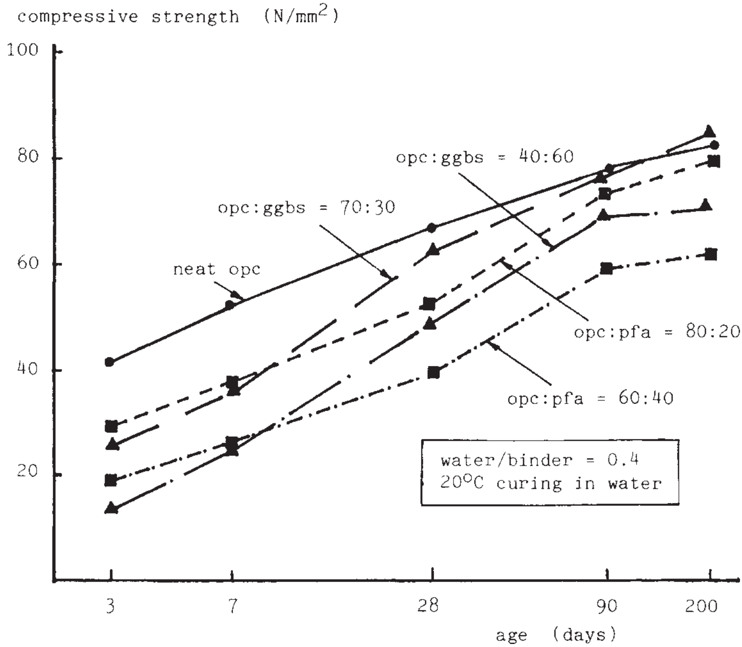

Figure 3.4 The strength development of grouts made with blends of ordinary Portland cement, pulverised ash (pfa) and ground-granulated blast furnace slag (ggbs) (UCL data, previously unpublished).

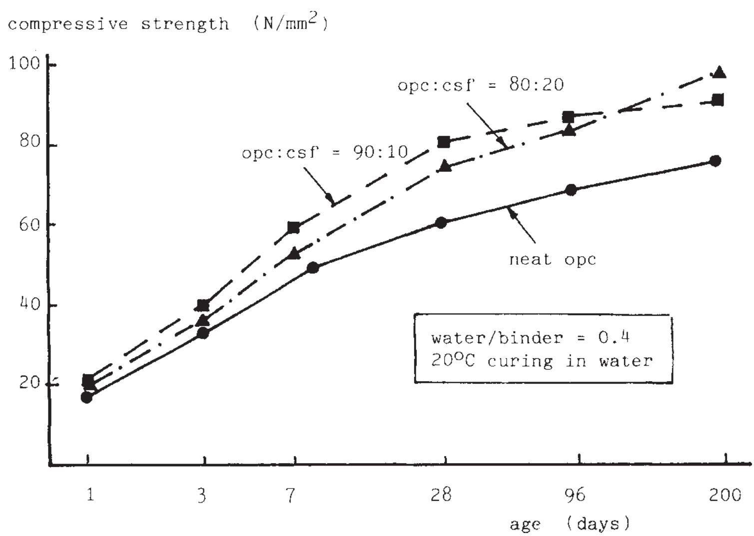

Figure 3.5 The strength development of grouts made with blends of ordinary Portland cement and condensed silica fume (csf) (Domone and Tank, 1986).

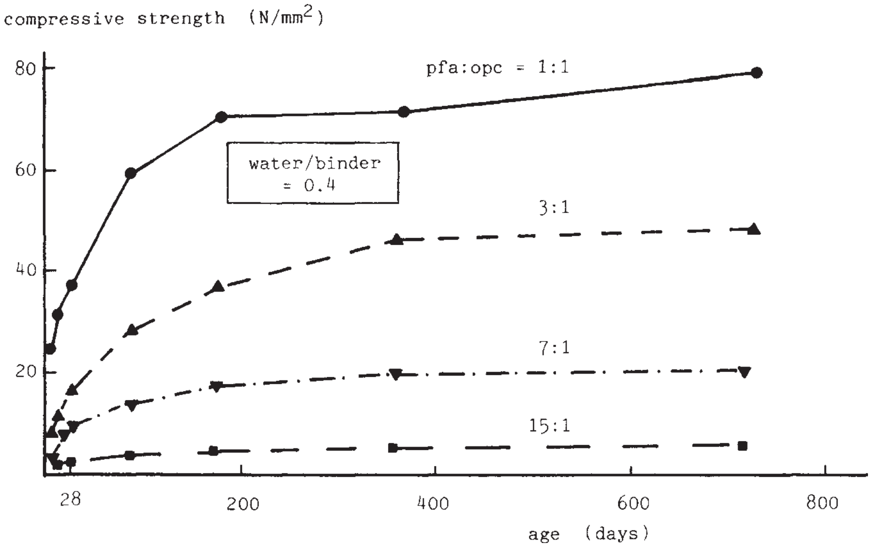

Figure 3.6 The strength development of high pfa content grouts (Hughes, 1991).

In general, all cement replacement materials are proportionally more effective at higher water/cement ratios. Also, mixes containing CRMs are more sensitive to poor curing at early ages than neat opc mixes. The maintenance of the high humidity associated with good curing practice is necessary for the continuation of the pozzolanic reaction and continuing strength gain (Killoh *et al.,* 1989).

As will be apparent from later chapters, there are many applications, such as void filling, for which low-strength grouts are required. One way of achieving this is by using high pfa content mixes. Work by Hughes (1991) has shown that, using selected pfa, a wide range of controlled and reproducible strength properties can be obtained. Figure 3.6 shows the effect of the pfa/opc ratio on the strength development. The pfa content has a marked effect on strength level at all ages, and there is considerable strength gain beyond 28 days, with the 1-year strength often being three times the 28-day strength. This shows the long-term nature of the pozzolanic reaction, which may never be complete.

The addition of significant quantities of gypsum to high pfa content grouts results in considerable strength increases, as shown in Figure 3.7. For 3:1 pfa/opc mixes, the greatest strength increases occurred with gypsum additions of about 0.5 to 1 times the weight of cement. Such grouts also show long-term expansions of about 0.5%.

Admixtures, which are used to alter the workability of the fresh mix (i.e.

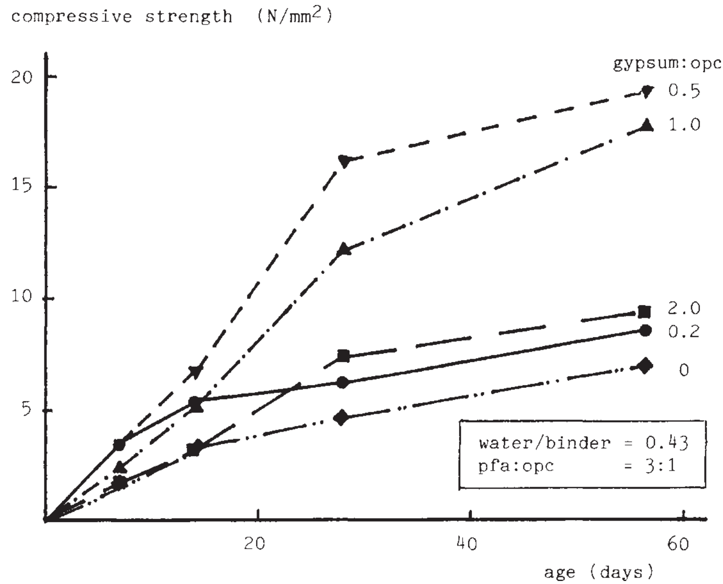

Figure 3.7 The effect of gypsum addition on the strength of high pfa content grouts (Hughes, 1988).

plasticizers and superplasticizers), generally do not in themselves have a major effect on the subsequent strength. The minor effects include

- a small (up to 10%) increase in strength at early ages resulting from the dispersion of the cement particles giving a greater surface area of fresh cement exposed to the mix water
- a reduction in strength due to a small amount of air entrainment with some superplasticizers
- some retardation of strength gain with some admixtures.

These admixtures are commonly used to increase strength by achieving equivalent fluidities at lower water contents and hence lower water/cement ratios.

Retarders and accelerators are used to control the setting time to advantage. Accelerators decrease this and increase the subsequent rate of gain of strength. Retarders increase the setting time and hence delay the onset of hardening; the subsequent rate of gain of strength is normally not altered.

The use of sea water for mixing can act as a mild accelerator (see chapter 1), resulting in an increase in strength of up to 25% in 1-day-old grouts, reducing to negligible increases at 28 days (Domone and Thurairatnam, 1986).

3.2.3 More fundamental relationships

The nature of the porosity of hardened cement, and the factors which influence it, have been discussed in some detail when considering microstructure in chapter 1. Not surprisingly, strength and porosity measurements often show good correlation, typical data being illustrated in Figure 3.8. The total porosity values in this figure include both gel and capillary porosity; only the latter can be reduced to near zero with low water/cement ratio mixes cured at normal temperatures and pressures, and the solid data points were obtained in this way. The data represented by open circles were obtained under higher temperatures and pressures, and serve to show the general principles rather than suggest a practical option for most grouting applications.

The porosity/strength relationship is not unique for all cement pastes, and different curves, albeit of the same general form, will be obtained for say, pastes cured under different temperature/pressure combinations, or for pastes containing pfa (Feldman and Beaudoin, 1976). This indicates some differences in the microstructure of the hydration product in different circumstances, explaining, for example, the different long-term strengths with different curing temperatures mentioned earlier.

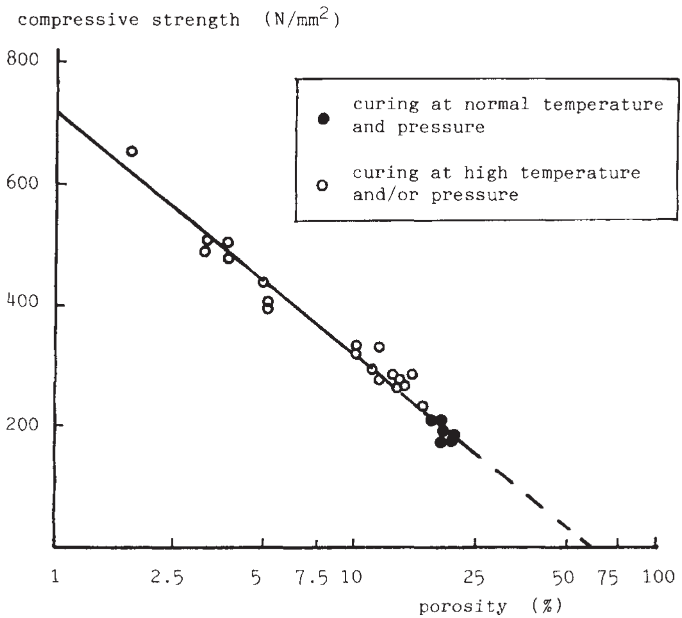

Figure 3.8 The relation between compressive strength and porosity for hardened cement paste (Roy and Gouda, 1973).

More recent research has shown the importance of the relative proportions of different pores sizes, and not just the overall porosity, in controlling strength. In general, it is the larger pores that are the most important; at any given porosity, the strength increases with the relative proportion of fine pores. For example, Odler and Rossler (1985) have concluded that pore sizes of less than 10 nm (10^{-2} um) are unimportant, and Parrott (1985) has obtained good inverse correlation of strength with large diameter porosity, defined as that due to pores in excess of 4 nm (4×10^{-3} µm). In some interesting developments in the early 1980s, Kendall *et al.* (1983) reduced the volume of the larger pores (those greater than about 15 µm) by incorporating a polymer into cement pastes with a water/ cement ratio of about 0.2, extruding the fresh paste and curing initially under pressure. The resulting 'Macro-Defect-Free' cement had compressive strengths of 200 N/mm^2 and above, and flexural strengths of about 70 N/mm^2, i.e the flexural strength increased proportionally more than the compressive strength.

3.3 Deformation

Deformation of cement grouts results both from environmental effects, such as moisture movement or change in temperature, and from applied stress, either short or long term. The general nature of the behaviour is illustrated in Figure 3.9, which shows the strain arising from a uniaxial compressive stress applied in a drying environment. The stress is applied at time t_1, and held

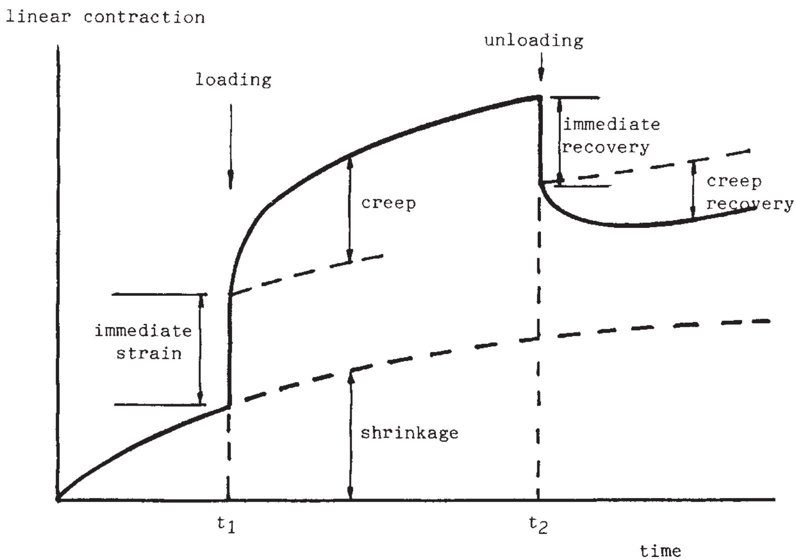

Figure 3.9 The response of grout to a compressive stress applied in a drying environment.

constant until removal at time t_2. Before applying the stress there is a net contraction in volume, or shrinkage, associated with the drying. (Even though this is a volumetric effect, this is normally quantified as a linear strain.) The dotted extension to this curve beyond time t_1 would be the subsequent behaviour without load, and the effects of the load are therefore the differences between this curve and the solid curves. Immediately on loading there is an instantaneous strain response, which for low levels of stress is approximately proportional to the applied stress, and hence an elastic modulus can be defined. This is followed by a time-dependent increase in strain; the increase, after allowing for shrinkage, is the creep strain. Although reducing in rate with time, it does not tend to a limiting value.

On unloading, there is an immediate strain recovery, which is often less than the instantaneous strain on loading. This is followed by a timedependent creep recovery, which is less than the preceding creep, i.e. there is a permanent deformation; however, unlike creep, this reaches a limiting value in due course.

3.3.1 Deformation from moisture movement

With no moisture exchange with the environment, i.e. in sealed conditions, the hydration reactions of a Portland cement result in a small decrease in volume while the grout is still plastic. During hardening, the progressive reduction in internal relative humidity with continuing hydration results in a further net overall volume reduction, called autogenous shrinkage. If water is continuously available from the environment, e.g. the grout is immersed, then some of this is absorbed and there is a small progressive increase in volume. Expressed in linear strain terms, this expansion may reach 0.2% after several years.

Of much greater significance for the structural performance of grouts is the shrinkage and swelling associated with loss or gain of moisture with a changing environment. Movements of 1% strain or more can be obtained, in timescales of days or weeks. The shrinkage should be termed **drying shrinkage,** to distinguish it from **carbonation shrinkage,** discussed later. The behaviour is repeated with successive cycles of drying and wetting, as illustrated schematically in Figure 3.10. Maximum shrinkage occurs on the first drying, and a considerable part of this is irreversible. Further drying and wetting cycles result in progressively smaller amounts of irreversible shrinkage. Also shown in Figure 3.10 is the relatively small, continuous swelling on permanent immersion.

Shrinkage test data should always be interpreted and compared with caution, since they will depend on specimen size and the nature of the environment. The most useful results are obtained from tests on samples of relatively small cross-section allowed to come to equilibrium. A typical

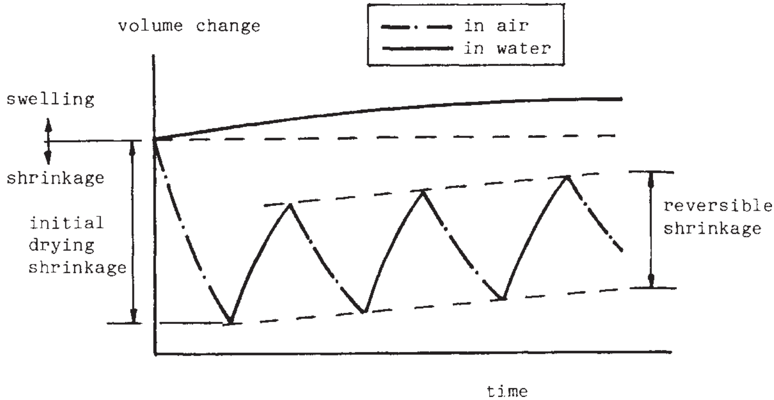

Figure 3.10 Volume changes in grout subject to alternate cycles of drying and wetting.

relationship between shrinkage and water loss (Figure 3.11) shows a distinct change of slope at relative humidities less than about 10%, indicating that there are probably at least two mechanisms of shrinkage. Readers interested in the mechanisms of behaviour should consult more specialist texts.

In general, the stronger the hydrate structure, the less it will respond to the internal forces of shrinkage and swelling, and so, everything else being constant, higher strength grouts can be expected to have lower drying shrinkage. This is demonstrated in Figure 3.12, in which the increasing total porosity of the paste is equivalent to decreasing strength. It would appear that it is the irreversible part of the shrinkage that increases with porosity, the reversible part remains constant.

These effects are, however, complicated by the fact that the influence of the degree of hydration on drying shrinkage is not so simple, for two reasons:

1. Unhydrated cement grains act as a restraint against movement, so reducing the shrinkage at low degrees of hydration.
2. With a high degree of hydration, there is less water in capillary pores, and the earlier loss of water from the gel pores results in greater shrinkage.

It is thus difficult to predict the effect of age, curing conditions, type of cement, etc., on the drying shrinkage of any particular grout.

Cement composition can also have a significant effect on the drying shrinkage, with cements from different sources producing shrinkages varying by a factor of 2 or more in otherwise identical test conditions (Blaine, 1968). The most significant factors causing this variation are probably the

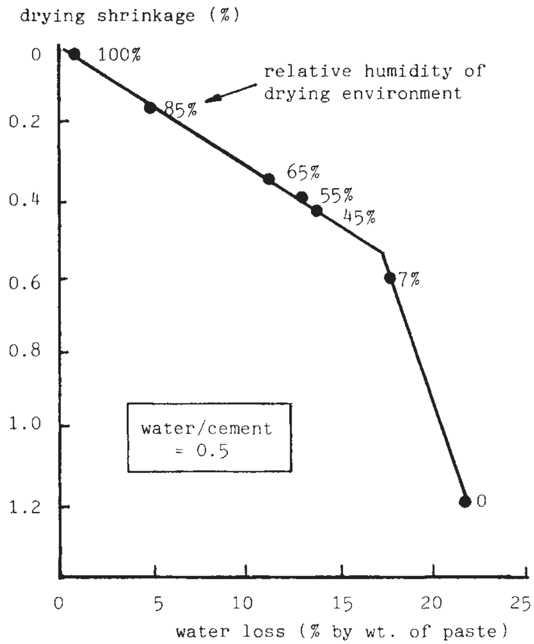

Figure 3.11 The relation between water loss and the drying shrinkage of hardened cement paste (Verbeck and Helmuth, 1968).

relative proportions of the calcium aluminate (C_3A) phase and the gypsum in the fresh cement. These govern the amount and rate of ettringite production, and control of this forms the basis for many of the so-called expansive cements, already discussed in chapter 1.

Most aggregates are dimensionally stable with gain or loss of moisture. Apart from lightweight aggregates, they are stiffer than the cement paste, and therefore restrain its moisture movement. The amount of restraint depends mainly on two factors, the volume concentration of the aggregate and its elastic modulus. These can be combined into the equation

$$\varepsilon_g/\varepsilon_p = (1-g)^n \tag{3.2}$$

Where ε_g and ε_p are the shrinkage strains in the composite grout and the cement paste respectively, g is the volume concentration of the aggregate, and n is constant which depends on the aggregate stiffness, and has been found to vary between 1.2 and 1.7 (L'Hermite, 1960)

3.3.2 Carbonation shrinkage

Carbonation shrinkage results from a chemical reaction within the grout, and is therefore distinctly different in nature from drying shrinkage.

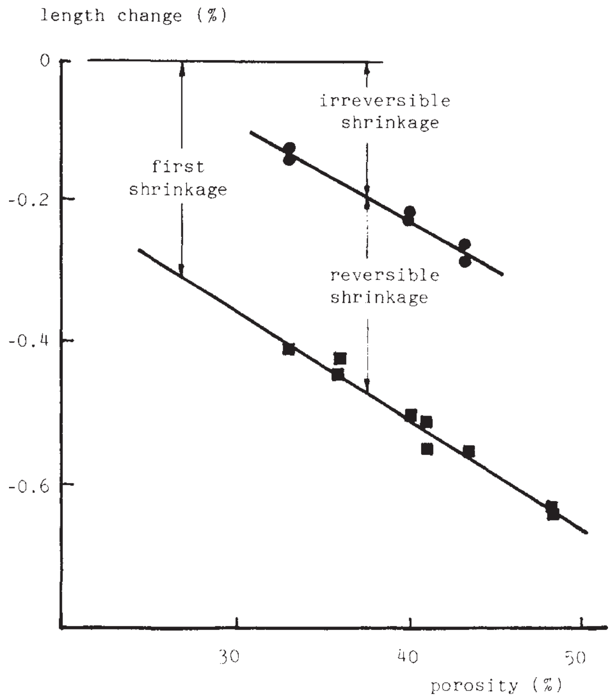

Figure 3.12 Reversible and irreversible shrinkage of hardened cement paste after drying at 47% relative humidity (Helmuth and Turk, 1967).

Carbon dioxide, when dissolved in water to form carbonic acid, can react with most of the components of hardened cement, the most important reaction being with calcium hydroxide to form calcite:

$$H_2CO_3 + Ca(OH)_2 = CaCO_3 + 2H_2O \tag{3.3}$$

There is an accompanying shrinkage, and the grout also increases in weight and strength and decreases in permeability. Even very dilute carbonic acid such as rain water can have a significant effect.

The rate and amount of carbonation depends in part on the degree of saturation of the grout, which is controlled by the relative humidity of the environment. If the grout is saturated, then the carbonic acid will penetrate slowly by diffusion, and the rate of carbonation will be insignificant; if the grout is very dry, then no carbonic acid is available. In concrete, maximum carbonation shrinkage occurs at humidities of 25–50%, and it can be of the same order of magnitude as drying shrinkage. The porosity of the grout is also an important controlling factor. Data for concrete indicate that at water/cement ratios of about 0.5–0.55, provided the grout is well compacted and properly cured, the carbonation front will penetrate at most a few centimetres in 10–20 years, and with higher quality grout even less. However, much greater penetration can occur with low quality grout or in regions of poor compaction. The loss of alkalinity associated with carbonation can lead to problems of corrosion of any steel reinforcement within the grout, as will be discussed later in the chapter.

3.3.3 Thermal expansion

The linear coefficient of thermal expansion of a neat cement grout varies between about 10 and 20×10^{-6} per °C. The value varies with moisture content, reaching a maximum at about 70% relative humidity (Figure 3.13). Moisture movement within the grout therefore probably contributes to the expansion, which is consequently time dependent; for example, the initial expansion on an increase in temperature shows some reduction over a few hours with the temperature held constant. For these reasons, estimates of movements in structural situations from tests on laboratory specimens should be treated with caution.

Most aggregates have thermal expansion coefficients lower than that of neat cement grouts, and therefore the presence of aggregate leads to a reduced coefficient for the composite grout, and to a reduced dependence on relative humidity.

3.3.4 Stress-strain behaviour

(a) Elasticity. Neat cement grouts show some convex curvature in their compressive stress-strain relationship for all levels of loading, and ther

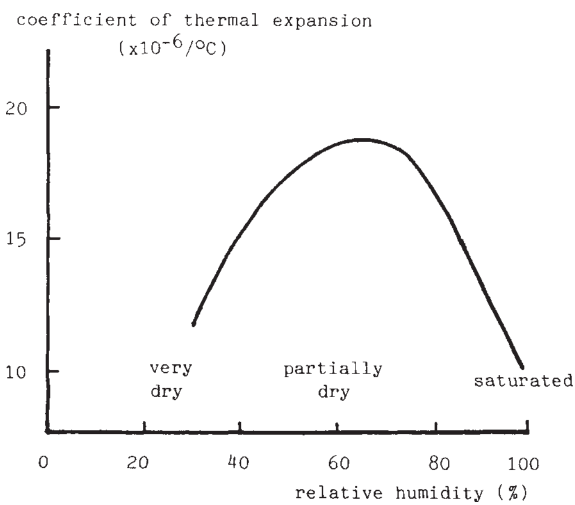

Figure 3.13 The effect of dryness on the thermal expansion of hardened cement paste (Meyers, 1951).

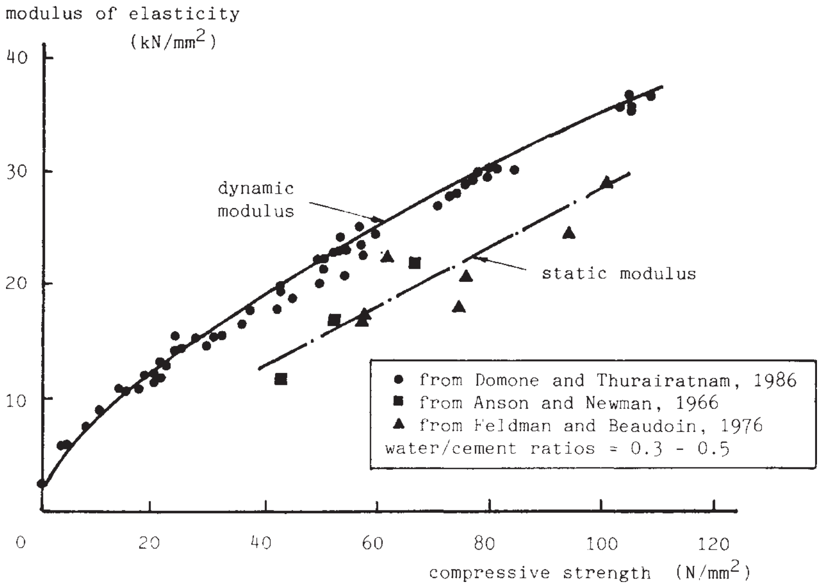

Figure 3.14 The relation between the elastic modulus and strength of Portland cement grouts.

fore modulus of elasticity values can only be defined at particular stress levels. The most commonly measured values are the dynamic elastic modulus, determined by a resonant frequency test (BSI, 1990), which is, in effect, a tangent modulus at zero stress, and the secant modulus at a stress of one-third of the ultimate, determined by a static loading test. The dynamic modulus is somewhat higher than the static value. In general, a good correlation is obtained between either modulus value and compressive strength, as shown in Figure 3.14. The correlation is essentially independent of age, water/cement ratio, cement type and the presence of admixtures. Water-saturated grouts generally have a slightly higher modulus than dried grouts, indicating that in the former some of the load is carried by the pore water. Nevertheless, the skeletal lattice of the hardened cement carries most of the load, and the elastic response is dominated by the lattice properties. As might therefore be expected, the elastic modulus (E_g) is highly dependent on the capillary porosity (p_c), as shown in Figure 3.15. The relationship can be expressed as

$$E_g=E_0\ (1-p_c)^3 \tag{3.4}$$

where E_0 is the modulus when p_c=0, i.e. it represents the modulus of elasticity of the gel hydrates themselves. This is a similar expression to that relating the strength to the porosity of the grout.

Grouts containing aggregates can be considered as two-phase materials for elastic deformation. The stiffness of the composite is intermediate

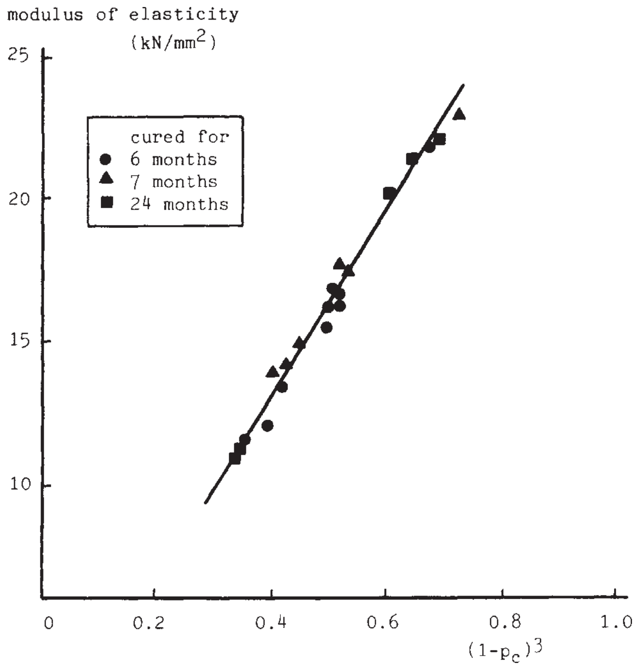

Figure 3.15 The effect of capillary porosity *(p_c)* on the elastic modulus of neat cement grout (Helmuth and Turk, 1966).

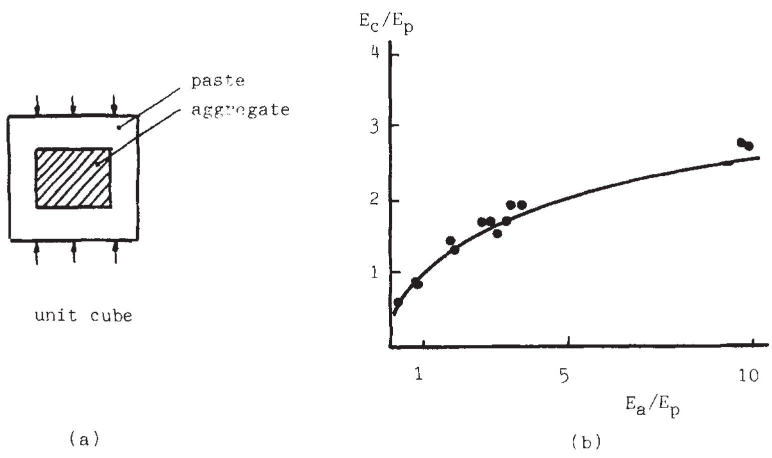

Figure 3.16 The prediction of the elastic modulus of a composite grout *(E_c)* from the modulus of the cement paste *(E_p)* and the aggregate *(E_a)* for a 50% volume concentration of the aggregate (Illston, 1993).

between that of the neat cement grout and the aggregate, and, except for lightweight aggregate, the latter is generally higher. The stiffness will also depend on the volume concentration of the aggregate, and linear analysis of the model shown in Figure 3.16(a) produces the results shown in Figure 3.16(b) which fit measured values reasonably well (Illston, 1993).

The Poisson's ratio of relatively mature water-saturated neat cement grouts varies between about 0.25 and 0.29; on drying this reduces to about 0.2 (Parrott, 1974). Values of the dynamic Poisson's ratio obtained from resonant frequency measurements of elastic and shear moduli show that at early ages a higher Poisson's ratio can be expected (Figure 3.17). The inclusion of aggregate lowers the Poisson's ratio to values between 0.18 and 0.21, with, in general, higher strength mixes having the lower values within this range (Anson and Newman, 1964).

(b) Creep. The magnitude of creep strains can be several times greater than the elastic strains on loading, and therefore they can have a highly significant effect on structural behaviour. Furthermore, although the creep rate reduces with time (except at stress levels approaching the short-term ultimate), it does not appear to tend to a limit. The creep is also substantially increased when the grout is simultaneously drying, i.e. drying shrink age and creep are interdependent. This leads to the definition of creep strains shown in Figure

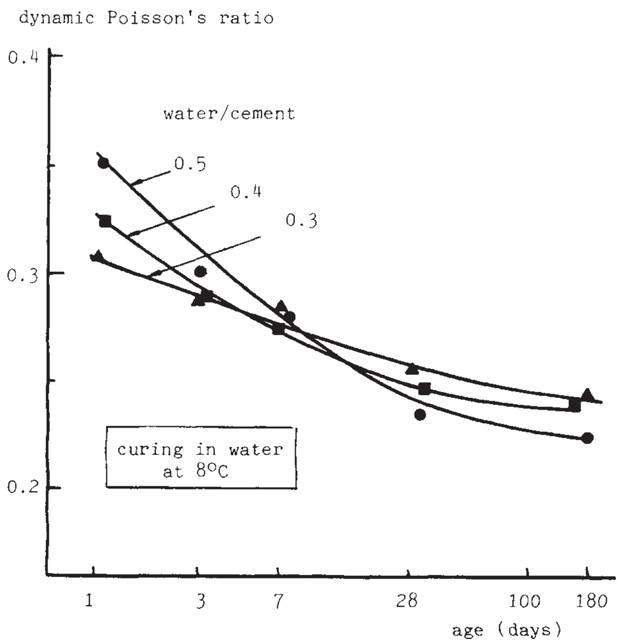

Figure 3.17 The development of dynamic Poisson's ratio in neat ordinary Portland cement grouts (Domone and Thurairatnam, 1986).

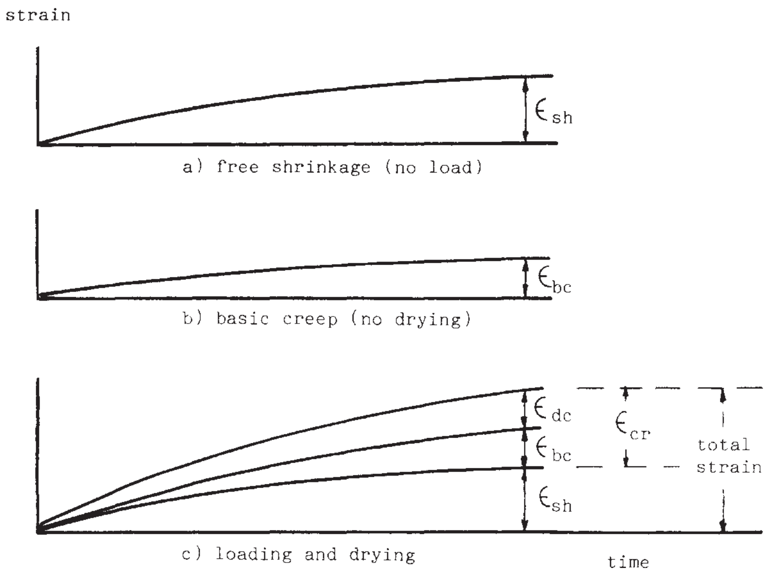

Figure 3.18 Shrinkage, creep and combined behaviour: ε_{sh}=shrinkage, ε_{bc}=basic creep, ε_{dc}=drying creep, ε_{cr}=total creep.

3.18. The free shrinkage (e_{sh}) is that occurring in an unloaded grout specimen when drying, and the basic creep (e_{bc}) is the creep of a similar specimen under load but sealed so that there is no moisture movement to or from the surrounding environment. The total strain (e_{tot}) is that measured on a specimen which is simultaneously drying and under load, i.e. creeping and shrinking, and it is found that

$$\varepsilon_{tot} > \varepsilon_{sh} + \varepsilon_{bc} \tag{3.5}$$

The difference, i.e. $\varepsilon_{ttot}-(\varepsilon_{tsh}+\varepsilon_{tbc})$, is called the drying creep (ε_{tdc}); its magnitude can be similar to or greater than that of the basic creep.

The main factors which influence the magnitude of the creep are:

1. *The moisture content before loading.* A lower moisture content reduces the creep, a further indication of the importance of moisture in the creep process. Completely dry grout has near zero creep.
2. *The level of applied stress.* The creep at any given time increases approximately linearly with stress up to stress/strength ratios of about 0.6; the creep per unit stress, or specific creep, is therefore often a convenient and useful quantity. At levels of stress approaching the short-term ultimate, the creep-stress relationship becomes increasingly non-linear, and eventually creep rupture may occur.

3. *Grout strength.* If all other factors are constant, creep decreases with increasing strength. Hence creep decreases with age at loading and decreasing water/cement ratio.
4. *Temperature.* An increase in temperature increases the creep significantly.
5. *The volume and elastic modulus of any included aggregate.* Most aggregate is essentially dimensionally stable, and the effect on creep is similar to that on shrinkage. There is an equivalent relationship of the form

$$C_g/C_p=(1-g)^n \tag{3.6}$$

   where C_g and C_p are the creep strains of the composite grout and neat cement grout respectively, g is the volume fraction of the aggregate and n is constant depending mainly on the aggregate stiffness.

There appears to be no simple relationship between creep and the cement composition. However, creep is approximately proportional to the level of applied stress and inversely proportional to the strength at the time of loading, and analysis of data from several sources has concluded that creep at a constant stress/strength ratio is approximately constant (Neville *et al.*, 1983). For example, the lower creep of rapid-hardening Portland cement grouts compared to opc grouts loaded under identical conditions of age, stress, etc., can be explained by its higher strength at loading. There is some evidence that mixes containing pfa may exhibit higher creep, but again the generally lower strengths at any age may explain this. Mixes containing ggbs exhibit lower basic creep, possibly due to the greater longterm strength gain during loading, but the total creep under drying conditions appears similar to neat opc mixes.

3.4 Durability

Durability can be defined as the ability of a material to remain serviceable for at least the required lifetime of the structure of which it forms a part. However, many structures do not have a well-defined lifetime, and in such cases the durability should ideally be such that the structure remains serviceable more or less indefinitely, given reasonable maintenance. Degradation of a grout can result from either the environment to which it is exposed (for example, frost damage) or from internal causes within the grout (as in alkali-aggregate reaction). However, it is perhaps more useful to divide the degradation processes into two broad groups:

- those that initially involve chemical reactions which subsequently lead to loss of physical integrity; these include attack by sulphates, sea water and other saline water, acids and alkali-silica reaction;

- those which directly lead to physical effects, such as attack by frost and fire.

In addition, it is necessary to distinguish between the processes degrading the grout itself and those corroding any steel in contact with the grout, in the form of reinforcement, prestressing or containment.

The rate of most of the degradation processes is controlled by the rate at which moisture, air or other aggressive agents can penetrate the grout. The most important transport mechanisms are steady-state flow (or permeation), diffusion and sorption. Each of these will be considered in turn, and the most important features of the main degradation processes then discussed.

3.4.1 Transport mechanisms through grout

As we have seen, cement grouts contain pores of varying types and sizes, and therefore the transport of materials through a grout can be considered as a particular case of the more general phenomenon of flow through a porous medium. The rate of flow will not depend simply on the porosity, but on the degree of continuity of the pores. The term 'permeability' is often loosely used to describe this general property, although, as we shall see, it also has a more specific meaning.

The flow processes depend on the degree of saturation of the grout, as illustrated in Figure 3.19, which represents the various stages of flow through an idealised single pore with a neck at each end.

At very low humidities, the moisture is in the vapour state and is adsorbed onto the dry surfaces of the cement paste (stage (a)). As the humidity increases, the adsorption becomes complete, and flow then takes place as direct vapour movement through the pore due to a pressure or concentration gradient, in the manner of inert gas (stage (b)). The next stage (c) occurs when the humidity is sufficient for water to condense in the restricted part of the pore; this shortens the path for vapour transfer, thus increasing the rate of movement. The condensed water zones extend with rising humidity (stage (d)) and the flow is augmented by transfer in the adsorbed layers. Straightforward liquid flow under a pressure gradient eventually occurs, initially in the incompletely saturated state (stage (e)), and finally in the completely saturated state (stage (f)). In addition, movement of ions or dissolved gases will occur through saturated pores under a concentration gradient.

Three processes can therefore be distinguished—steady-state flow, diffusion and sorption, each of which has an associated 'flow constant'.

1. **Steady-state flow,** i.e. flow or movement of a fluid under a pressure differential. The flow passages through the grout and the flow rates are sufficiently small for the flow of either a liquid or a gas to be **Figure 3.19** Modes of moisture transport through a typical pore in cement grout (Rose, 1965).

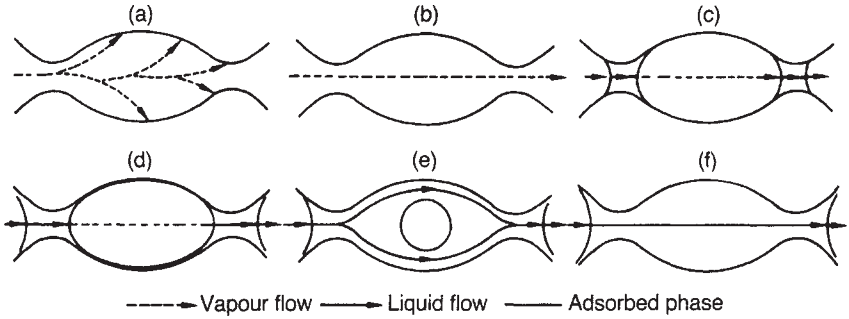

Figure 3.19 **Modes of moisture transport through a typical pore in cement grout (Rose, 1965).**

laminar, and hence it can be described by Darcy's law:

$$u_x = -K.\partial h/\partial x \quad (3.7)$$

where, for flow in the x-direction, u_x is the mean flow velocity, $\partial h/\partial x$ is the rate of increase in pressure head in the x-direction and K is a constant called the coefficient of permeability, the dimensions of which are [length]/[time], e.g. m/s. The value of K depends on both the pore structure within the grout and the properties of the permeating fluid, often water.

2. **Diffusion,** i.e. movement of ions, atoms or molecules under a concentration gradient. This is governed by Fick's law:

$$J = -D.\partial C/\partial x \quad (3.8)$$

where, for the x-direction, J is the transfer rate of the substance per unit area normal to the x-direction, $\partial C/\partial x$ is the concentration gradient and D is a constant called the diffusivity, which has the dimensions of $[\text{length}]^2/[\text{time}]$, e.g. m^2/s.

Many types of diffusion process are of interest, e.g. moisture movement during drying shrinkage, or chloride diffusion from a marine environment. Furthermore, in the case of moisture diffusion (in, say, drying shrinkage) the moisture content within the pores will be changing throughout the diffusion process. There is, however, some justification for considering D as a constant for any one particular diffusion process, but it should be remembered that, as with the permeability coefficient K, it is dependent on both the pore structure of the concrete and the properties of the diffusing substance.

3. **Sorption** (both adsorption and absorption) of liquid into empty or partially empty pores by capillary attraction. It has been shown analytically and experimentally that, for constant environmental con

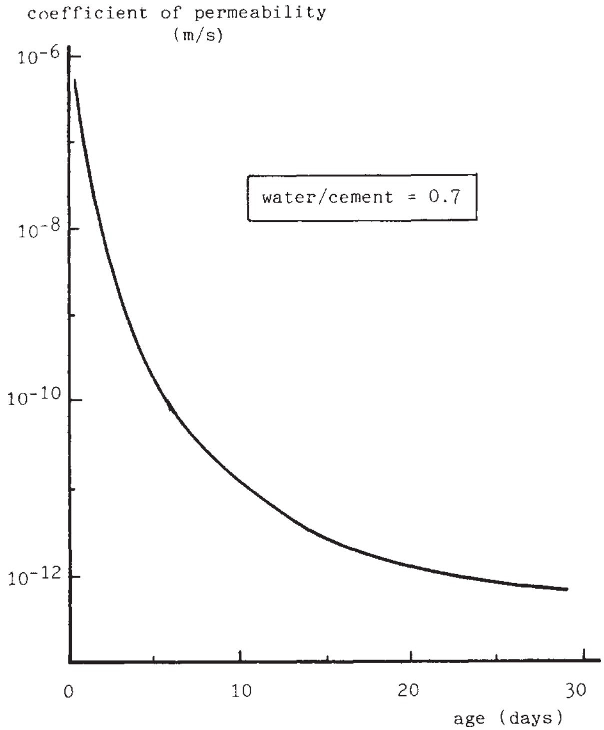

Figure 3.20 The effect of hydration on the permeability of cement paste (Powers *et al.,* 1954).

ditions, the depth of penetration *(x)* of the liquid is proportional to the square root of the time *(t),* i.e.

$$x=S.t^{0.5} \tag{3.9}$$

where S is a constant called the sorptivity, which has the dimensions of [length]/[time]$^{0.5}$, e.g. mm/s$^{0.5}$. S is, in effect, a diffusion constant for this specific situation.

3.4.2 Factors affecting the flow constants for grouts

(a) Permeability. As cement hydrates, the hydration products infill the skeletal structures, blocking the flow channels and hence reducing the permeability. The reduction is high at early ages, when hydration is proceeding rapidly, and it reduces by several orders of magnitude in the first 2–3 weeks after casting (Figure 3.20).

Although permeability and porosity are not necessarily related, there is a general non-linear correlation between the two for cement paste, as shown in

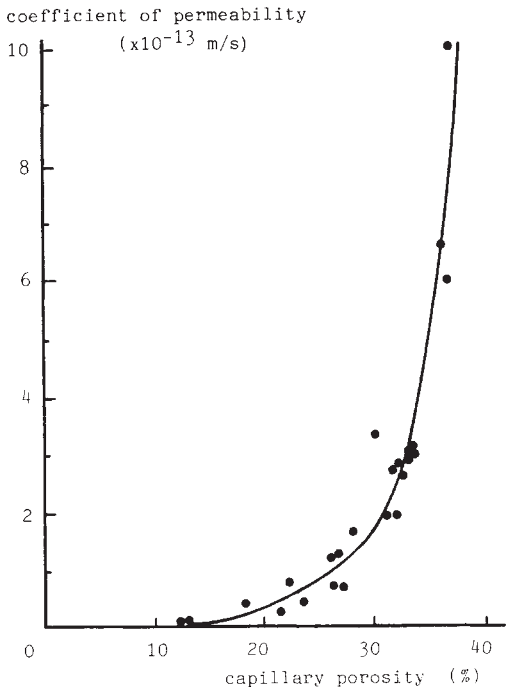

Figure 3.21 The relationship between the permeability and capillary porosity of hardened cement paste (Powers, 1958).

Figure 3.21. The greatest reduction in permeability occurs for porosities reducing from about 40 to 25%, where increased hydration product reduces both the pore sizes and the flow channels between them. Further hydration product, although still reducing porosity significantly, results in much lower changes in the permeability. This explains the general form of Figure 3.20, and also accounts for the effect of water/ cement ratio on permeability shown in Figure 3.22 for a constant degree of hydration. At water/cement ratios above about 0.55 the capillary pores form an increasingly continuous system, with consequent large increases in permeability.

It is clear from the above arguments and those in section 3.2.3 that high strength and low permeability both result from low porosity, and in particular a reduction in the volume of the larger pores. Although higher strength implies lower permeability, the relationship is not linear, and may be different for different curing histories and cement types. However, in general, low permeability is produced by attention to the same factors required to produce high strength. These include using a low water/cement ratio and ensuring proper compaction and adequate curing. The pozzolanic reaction associated with the use of cement replacement materials can

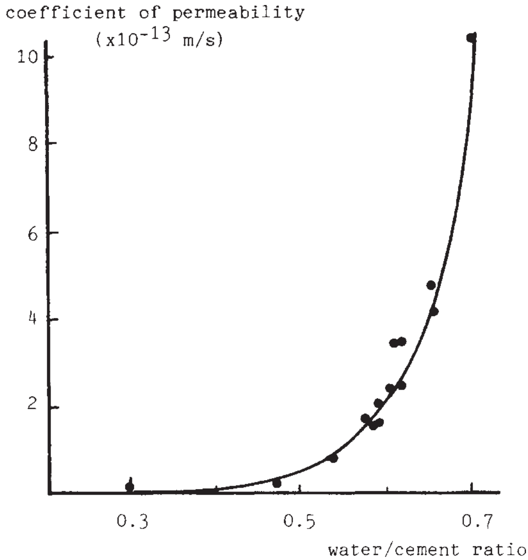

Figure 3.22 The relationship between the permeability and water/cement ratio of mature hardened cement paste (93% hydrated) (Powers *et al.*, 1954).

produce long-term benefits, but longer curing periods are necessary to ensure continuance of the reaction. The avoidance of microcracking from thermal or drying shrinkage strains and premature or excessive loading is also important.

Any included aggregate will influence the permeability. Many of the rock types used for natural aggregates have permeabilities of the same order as that of cement paste. Lightweight aggregates, which are highly porous, can have much higher permeabilities. However, in practice the permeability of a composite grout is often found to be substantially higher than that of either the aggregate or the neat cement grout. This is primarily due to the presence of defects or cracks, particularly in the weaker transition zone at the cement-aggregate interface.

(b) Diffusivity. Diffusivity measurements on cement grouts have generally been carried out on relatively mature specimens. As might be expected, higher water/cement ratios lead to higher diffusivities; for example, Page *et al.* (1981) have found values for chloride ion diffusivity through saturated grout at 25ºC of 2.6, 4.4 and 12.4×10^{-12} m^2/s for water/cement ratios of 0.4, 0.5 and 0.6 respectively. In addition, the diffusivity of the 0.5 water/cement ratio paste was reduced to 1.47×10^{-12} m^2/s for a mix containing 30% pfa and 0.41×10^{-12} m^2/s for a mix with 70% ggbs.

(c) Sorptivity. Few, if any, tests have been carried out to determine sorptivity values for grouts. As with the other flow constants, low sorptivity can be expected with lower water/cement ratios. Tests on concrete have indicated the importance of adequate curing in achieving low sorptivity, particularly in mixes containing cement replacement materials (Bamforth and Pocock, 1990).

3.4.3 Degradation processes

(a) Attack by sulphates and sea water. Any sulphates coming into contact with hardened grout, for example from groundwater, can react with the hydrated aluminate phases in the cement paste to form ettringite:

$$C_3A.C\overline{S}.H_{18} + 2CH + 2\overline{S} + 12H = C_3A.3C\overline{S}.H_{32}^{*} \qquad (3.10)$$

The growth of ettringite crystals causes expansion, and hence disruption.

With some sulphates, reactions can also occur with calcium hydroxide in the hardened grout, forming gypsum, which may cause a loss of stiffness and strength. For example, the reaction with sodium sulphate (using normal chemical notation) is:

$$Na_2SO_4 + Ca(OH)_2 + 2H_2O = CaSO_4.2H_2O + 2NaOH \qquad (3.11)$$

With magnesium sulphate, the magnesium hydroxide formed is relatively insoluble and poorly alkaline; this reduces the stability of the calcium silicate hydrate which is also attacked:

$$MgSO_4 + Ca(OH)_2 + 2H_2O = CaSO_4.2H_2O + Mg(OH)_2 \qquad (3.12)$$

$$3MgSO_4 + 3CaO.2SiO_2.3H_2O + 8H_2O = 3(CaSO_4.2H_2O) + 3Mg(OH)_2 + 2SiO_2.H_2O \qquad (3.13)$$

Thus the severity of attack depends on the type of sulphate; magnesium sulphate is more damaging than sodium sulphate, which, in turn, is more damaging than calcium sulphate. In each case the rate of attack increases with the availability of sulphate, e.g. its concentration in the groundwater, but the rate of increase in intensity reduces above a concentration of about 1%. Also, the rate of attack will be faster if the sulphates are replenished, e.g. if the grout is exposed to flowing groundwater.

Damage usually starts at the grout surface, particularly at any edges and corners, followed by progressive cracking and spalling, eventually leading to complete breakdown. Although this stage can be reached in a few months in laboratory tests, it normally takes several years in the field.

For any given concentration and type of sulphate, the rate and amount of the deterioration increases with:

*See chapter 1 for an explanation of the shorthand symbols used in cement chemistry.

- the C_3A content of the cement; hence the low C_3A content of sulphate-resisting Portland cement
- higher water/cement ratio—higher quality grout is less vulnerable due to its lower permeability—which is a more significant factor than the C_3A content, and is decreased by
- the incorporation of cement replacement materials, which decrease the permeability, reduce the amount of free lime in the hardened cement paste (hcp), and effectively 'dilute' the C_3A.

Sea water contains sulphates, along with other salts, and its action on grout has some similarities to that of pure sulphate solutions, with the addition of some interactive effects. The total soluble salt content of the sea water is typically about 3.5% by weight, the principle ionic contributors to this being 2.0% Cl^-, 1.1% Na^+, 0.27% SO_4^{2-}, 0.12% Mg^{2+} and 0.05% Ca^{2+}. The sulphates react as described above, but the severity of the attack is not as great as for sulphates acting alone and there is little accompanying expansion. This is due to the presence of chlorides; gypsum and ettringite are more soluble in a chloride solution than in pure water, and therefore tend to be leached out of the concrete by the sea water. The magnesium ions participate in the above reactions, and a feature of sea water damaged grout is the presence of white deposits of $Mg(OH)_2$, called brucite. This can have a pore-blocking effect, which may reduce the rate of attack.

The salts in sea water can contribute to two other, potentially much more critical, degradation processes, i.e. alkali-aggregate reaction and corrosion of embedded steel. These are both discussed later.

(b) Acid attack. Hardened cement paste is alkaline, and therefore no Portland cement concrete can be considered acid resistant. However, it is possible to reduce the level of attack by acids by giving attention to low permeability and good curing. In these circumstances, attack is only considered significant if the pH of the aggressive medium is less than about 6.

Potentially harmful weak acids include CO_2 and SO_2 in rain water, and CO_{2-} or organic acid-bearing groundwater from moorlands. The acids attack the calcium hydroxide within the cement paste, converting it, in the case of CO_2, into calcium carbonate and bicarbonate. The latter is relatively soluble, and leaches out of the grout, destabilising it. The process is thus diffusion controlled, and progresses at a rate approximately proportional to the square root of time. The rate of attack also increases with reducing pH.

Cement replacement materials may increase the acid resistance, probably because of the lower calcium hydroxide content as a result of the pozzolanic reaction.

(c) Alkali-aggregate reaction. Alkali-aggregate reaction is a significant

problem in concrete, and grouts containing a fine aggregate are potentially vulnerable. If the aggregate contains certain reactive forms of silica, silicates or carbonates, these may react with the alkalies (sodium, potassium and calcium hydroxide) in the hydrated Portland cement paste. The product is a gel which absorbs water and can swell to a sufficient extent to cause cracking and disruption of the grout. The most common and important reaction in concrete involves active silica—this is known as alkali-silica reaction (ASR) for obvious reasons.

For the reaction to occur, both active silica and alkalies must be present. In its reactive mineral form, silica occurs as the minerals opal, chalcedony, crystobalite, tridymite and volcanic glasses. These can be found in some flints, limestones, cherts and tuffs. Only a small proportion of reactive material in the aggregate (as low as 0.5%) may be necessary to cause disruption.

In unhydrated opc, the Na_2O and K_2O are present in small but significant quantities, either as soluble sulphates (Na_2SO_4 and K_2SO_4) or a mixed salt $(Na, K)SO_4$. There is also a small amount of free CaO, which is subsequently supplemented by $Ca(OH)_2$ from the hydration reactions of C_3S and C_2S. During hydration, these sulphates take part in a reaction with the aluminate phases of the cement, with sodium, potassium and hydroxyl ions going into solution. These may be supplemented by alkalies from external sources, such as aggregate impurities, sea water or road deicing salts.

The reaction between the active silica and the alkalies, to form the alkali-silicate gel occurs first at the aggregate/cement paste interface. The nature of the gel is complex, but it is clear that it is a mixture of sodium, potassium and calcium silicates. It is soft, but on contact with water imbibes a large quantity of water by osmosis and swells considerably. The hydraulic pressure that is developed leads to an overall expansion and can be sufficient to cause cracking of the aggregate particles, the hardened cement and the transition zone between the two.

Continued availability of water causes enlargement and extension of the cracks. The whole process is often very slow, and the cracking can take years to develop in structural concrete. In recent years many cases have been identified, which have generated extensive ongoing research, primarily on concrete. The most important factors which appear to influence the amount and rate of reaction can be summarised as follows:

- *The amount of alkalies available.* (This is normally expressed as the total weight of sodium oxide equivalent=$Na_2O+0.658K_2O$.) There seems to be a threshold level below which no disruption will occur, even with reactive aggregates. This typically corresponds to a lower limit of about 0.6% by weight of cement. The expansion increases rapidly above this level.
- *The amount of reactive silica.* Typical behaviour is shown in Figure

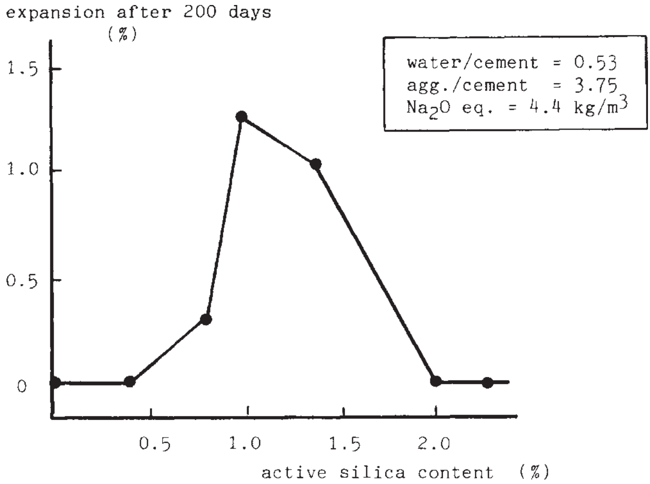

Figure 3.23 The effect of the active silica content of a mortar on the expansion after 200 days due to alkali-silica reaction (Hobbs, 1988).

3.23; this demonstrates the existence of a threshold level and a pessimum level of silica content for maximum expansion.

- *The aggregate particle size.* This affects the amount of reactive silica exposed to the alkali; fine particles therefore produce more rapid and greater expansion, hence the need for caution with the fine aggregate fillers normally used in grouts.
- *The gel composition.* There is evidence that $Ca(OH)_2$ in the gel is necessary, but the overall effect of gel composition is not fully understood.
- *The availability of moisture.* The gel swelling will cease if the internal relative humidity falls below 75%. This will depend on the environment and the grout permeability. Alternate wetting and drying can be most harmful.
- *The ambient temperature.* Higher temperatures accelerate the reaction, at least up to 40°C.

Once started, ASR is almost impossible to eliminate, and although the structural effects will vary from one application to another, it is perhaps preferable to reduce or eliminate the risk of ASR occurring by careful materials selection and mix design. Preventive measures include:

- Avoid the use of reactive aggregates. This is more difficult than it sounds, particularly with mixed mineral aggregates. Ideally aggregates of proven performance should be used.
- Limit the amount of alkalies in the cement, for example by using a low-

alkali cement—i.e. with an alkali content of less than 0.6% by weight, as discussed above—or by combining opc with a cement replacement material. Although pfa and ggbs can contain high total alkali levels, the extent to which this contributes to the total alkalinity of the pore water appears to be small when they are combined with a high-alkalinity Portland cement, and therefore the effective total alkalinity is reduced. Also, ggbs contains a form of silica which reacts slowly with the alkalies to give a non-expansive product, and for mixes with at least 50% opc replacement by ggbs, higher total alkalies can be tolerated. With csf, which is very finely divided active silica, the opposite effect occurs; the reaction is accelerated so that it is essentially completed while the concrete is in its fluid, fresh state, rendering the expansion harmless. However, the exact mechanisms and quantitative nature of the role of cement replacement materials are complex and unclear, and are the subject of continuing research.

- Limit the total alkali content of the mix. Alkalies from all sources (cement, de-icing salts, sea water, etc.) should total less than 3.0 kg/ m_3. Partial substitution of the opc with pfa or ggbs can be used to achieve this figure; at least 25% substitution should be used, and their own alkali content can be ignored, as discussed above.
- Try to ensure that the grout remains dry throughout its life—which is obviously difficult, or impossible in many situations.

(d) Frost damage. Frost action can cause damage to grouts when moisture contained in the larger pores freezes. Free water expands by about 9% on freezing, and if there is insufficient space within the grout to accommodate this then internal, potentially disruptive pressures will result. Successive cycles of freezing and thawing can cause progressive and cumulative damage, which takes the form of cracking and spalling, initially at the grout surface.

The water in the larger capillary pores and entrapped air voids has the greatest effect; the water in the much smaller gel pores is adsorbed on to the calcium silicate hydrate (C-S-H) surfaces, and does not freeze until the temperature falls to about -78ºC. However, after the capillary water has frozen, it has a lower thermodynamic energy than the still liquid gel water, which therefore tends to migrate to supplement the capillary water, thus increasing the disruption. The disruptive pressure is also enhanced by osmosis. The water in the pores is not pure, but is a solution of calcium hydroxide and other alkalies, and perhaps chlorides; pure water separates out on freezing, leading to salt concentration gradients and osmotic pressures which increase the diffusion of water to the freezing front.

The magnitude of the disruptive pressure depends on the capillary porosity, the degree of saturation of the grout, and the pressure relief provided by a nearby free surface or escape boundary. The extent of this pressure relief will depend on:

- the permeability of the material
- the rate at which ice is formed and
- the distance from the point of ice formation to the escape boundary.

In saturated cement paste, the disruptive pressures will only be relieved if the point of ice formation is within about 0.1 mm of an escape boundary. A convenient way of achieving this is by use of an air-entraining admixture, which entrains air in the form of small discrete bubbles at an average spacing of about 0.2 mm.

The capillary porosity of hardened cement paste, and hence its susceptibility to frost attack, can also be reduced by lowering the water/cement ratio and ensuring that hydration is as complete as possible, i.e. by proper curing. Bleeding, which can result in local high-porosity zones, should also be minimised.

(e) Fire resistance. Cement grouts are incombustible and do not emit any toxic fumes when exposed to high temperatures. However, although they can retain some strength for a reasonable time at high temperatures, they will eventually degrade, the rate of degradation depending on the maximum temperature and the overall dimensions of the grout mass.

For temperatures up to about 500ºC the strength reduction is relatively gradual, but thereafter the decline is more rapid, giving almost total loss approaching 1000ºC. At the lower temperatures, if the grout is initially saturated and of low permeability, then the build-up of steam pressure can lead to cracking and spalling. This continues with progressively more tightly held water being given off as the temperature increases. At higher temperatures, in excess of 500ºC, differential expansion between the hcp and any included aggregate results in thermal stresses and cracking, initially at the interface between the two, leading to more rapid loss of strength. At temperatures approaching 1000ºC, the hydrates in the hardened cement break down, resulting in a total loss of strength.

3.4.4 Durability of steel in grout

In many structural situations, grouts are in contact with steel, either in the form of reinforcement or prestressing, or as a containment. High-quality grout provides an excellent protective medium for the steel, but this protection can be broken down in some circumstances, leaving the steel vulnerable to corrosion. It is of great significance that the corrosion products, i.e. rust in various forms, occupy a considerably greater volume than the original steel and can therefore cause cracking of the grout, followed by more rapid corrosion of the exposed steel and eventual loss of structural integrity.

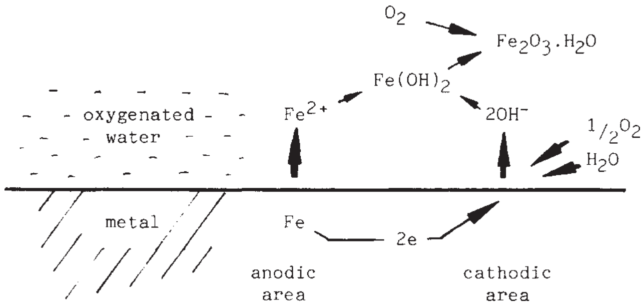

Figure 3.24 The corrosion reactions of iron in moist air or oxygenated water.

In this section the general nature of the corrosion process of steel is first described, followed by consideration of the factors that control its onset and subsequent rate.

Corrosion of steel is an electrochemical process, and for iron or steel rusting in oxygenated water or moist air, the water on or near the metal surface acts as the electrolyte of a corrosion cell. The anode and cathode of the cell are close together, e.g across a single crystal or grain, and the corrosion products, iron oxides and hydroxides, are formed and deposited away from the surface, as illustrated in Figure 3.24, allowing the corrosion to be continuous. For steel in contact with grout, the electrolyte is the pore water in contact with the steel, and this is normally highly alkaline (pH= 12–13) due to the $Ca(OH)_2$ from the cement hydration and the small amounts of Na_2O and K_2O in the cement. In such a solution, the primary anodic product is not ferrous hydroxide, but is the mixed oxide Fe_3O_4, which is deposited at the metal surface as a tightly adherent thin film, and stifles any further corrosion. The steel is said to be passive, and thus grout provides an excellent protective medium. However, the passivity can be destroyed by either

(a) a loss of alkalinity, for example by carbonation, in which the calcium hydroxide is neutralised by carbon dioxide from the air, producing calcium carbonate (as described in section 3.3.2), or
(b) chloride ions, e.g. from road de-icing salts or sea water.

Either of these can therefore create conditions for the corrosion reactions illustrated in Figure 3.24 to occur. The corrosion can be localised, for example in load-induced cracks, leading to pitting, or the corrosion cells can be quite large, for example if anodic areas have been created by penetration of chlorides into a locally, poorly compacted area of grout. However, it is important to remember that oxygen and water must still be available at the cathode to ensure that the reaction continues.

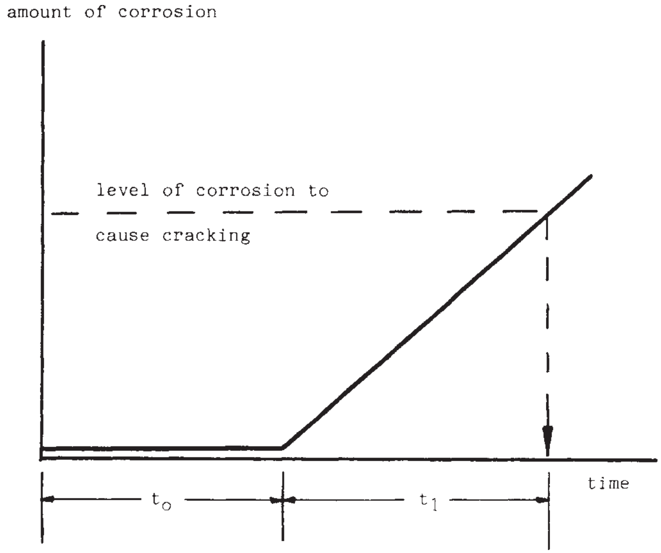

Figure 3.25 The two stages of damage due to corrosion of steel in grout: t_0=time to initiation of corrosion, t_1=time for sufficient corrosion to cause cracking (Browne, 1983).

Since the carbon dioxide or chlorides will normally have to penetrate through the grout to the steel before the corrosion can be initiated (except in the case of chloride-containing admixtures or contaminated aggregates, which are normally avoided) the total time to concrete cracking will consist of two parts, shown diagrammatically in Figure 3.25:

- the time *(t_0)* for the depassivating agents (the carbon dioxide or chlorides) to reach the steel and initiate the corrosion
- the time *(t_1)* for the corrosion to then reach critical levels, i.e. sufficient to crack the concrete, which depends on the subsequent corrosion rate.

Carbonation is a diffusion-controlled process, but it is not of major importance with grouts as in many applications these are not exposed to the atmosphere. Where this is not the case, e.g. for grout used in the repair of reinforced concrete, the rate of penetration of the carbonation front can be reduced to near negligible levels by the use of a high quality properly cured grout.

Calcium chloride, or any chloride-containing admixture, is not normally permitted in any grout which is to be in contact with steel. However, small amounts of chlorides from aggregates can be tolerated, because they participate in a reaction with the aluminate phases of the cement, and therefore no longer exist as chloride ions. Typical ‘threshold’ limits for concretes (expressed as chloride ion by weight of cement) are 0.2% if sulphate-resisting Portland cement is used, or 0.4% if ordinary Portland, rapid-hardening, low-heat Portland or Portland blast furnace cement is used. More chlorides can be tolerated with the latter group as these cements contain higher proportions of aluminates. Similar limits would apply to grouts.

Chlorides from sea water or de-icing salts have to penetrate through the grout in sufficient quantities to depassivate the steel before the corrosion is initiated, i.e. t_0 is finite in these circumstances. The transport mechanisms may be dominated by the permeability, e.g. where grout is permanently submerged in sea water, or diffusivity, e.g. where salts are deposited on to saturated grout, or sorptivity, e.g. where salts are deposited onto partially saturated grout. All processes result in chloride profiles of reducing chloride content with distance from the grout surface. The chloride concentration at a given depth and time will depend on the exposure condition and the grout's permeability, diffusivity or sorptivity. In general, lower water/cement ratios and fully cured grouts will prolong the initiation period t_0.

In practice, it is desirable to ensure that t_0 is as long as possible by careful selection of the grout mix and sound grouting practice. It is difficult, and certainly more expensive, to control or reduce corrosion once it has started. However, if protection against corrosion cannot be guaranteed, then any of the following extra protective measures can be considered:

1. Add a corrosion-inhibiting admixture, such as calcium nitrite, to the fresh grout.
2. Use a corrosion-resistant stainless steel, or epoxy-coated steel.
3. Apply a protective coating to the grout, to reduce chloride and/or oxygen ingress.
4. Protect the steel by applying a voltage from an external source sufficient to ensure that all of the steel remains permanently cathodic.

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4 Other types of grout

P.L.J.DOMONE

4.1 Low-strength silicate cement grouts

4.1.1 The nature of silicate grouts

Sodium silicate is not a compound in the normal sense of the word, but is a mixture of sodium oxide (Na_2O), silica (SiO_2) and water; solutions with weight ratios of SiO_2 to Na_2O between 1.6 and 4 or more, and with solids contents from 28 to 84% by weight, are available. Silicates with weight ratios in the range of 3–4 can react with a number of other compounds to form gels with adhesive properties. In the so-called 'chemical grouting' of soil and rock formations, the gel is formed within a porous soil or rock, thus blocking the pores and reducing the formation's overall permeability and increasing its strength.

The speed of the gelling reaction can vary considerably, depending on the reacting compound. For example, the reaction with chlorides is virtually instantaneous, and has formed the basis of the Joosten process of soil grouting, in which the sodium silicate is first injected, followed by a solution of calcium chloride, the gel being formed within the soil at the insitu interfac e of the two compounds. This is thus a two-stage process, but in recent years it has been replaced by a more convenient single-stage grouting, in which the sodium silicate is mixed before injecting with, for example, esters, amides or bicarbonates. The gel time of the mixture is much longer, and more importantly, can be controlled by varying the mix proportions to occur at an acceptable time after placing.

This non-structural grouting is outside the scope of this book, and has been described in detail elsewhere (Karol, 1990). However, grouts containing mixtures of sodium silicate and Portland cement have been used for structural purposes. Two applications of such grouts, described in chapters 6 and 9, are:

(a) to provide stability to the foundations of offshore concrete gravity platforms, and
(b) to fill the annulus between tunnel linings and the surrounding soil or rock.

In both cases the grout provides a load-bearing interface between the structure (the platform or the tunnel lining) and the soil (or rock), and therefore only a low strength is required, perhaps slightly greater than that of the soil. Indeed, a higher strength is likely to lead to undesirably high contact pressures. As we shall see, sodium silicate/cement grouts are ideal for both purposes.

The platform underbase grouts are based on mixtures of sodium silicate, cement and sea water, whereas the tunnel-lining grouts use fresh water. The basic concept of both grouts is similar:

- the gel provides stability after placing to mixes with a high water/cementing solids ratio;
- the hydration products then form a uniform structure of low strength.

However, the mixing sequence and resulting properties of the grouts, particularly when fresh, are significantly different, and therefore each type of grout is best described separately.

4.1.2 Silicate/sea water grouts

On mixing sea water and sodium silicate a milky white gelatinous precipitate of gel is formed virtually instantaneously. This is therefore a similar reaction to that occurring in the Joosten process mentioned above, except that the more complex mixture of salts in sea water is involved, rather than the single salt calcium chloride.

If the mixing process involves high shear, the gel, which otherwise tends to form flocs, is broken down and fully dispersed. If a relatively low quantity of Portland cement is then added, again with high shear mixing, the gel provides sufficient stability to the fresh grout for the cement to hydrate to form a uniform, low-strength hardened structure.

The fresh, early age and hardened properties of the grout can all be controlled by variation of the constituents and their proportions. Experimental programmes have been carried out on mixes with water/cement ratios varying from 1.0 to 5.0 and sodium silicate contents varying from 1 to 10% by weight of sea water. Rheology, stability, early age thermal effects, strength, modulus of elasticity, and long-term performance have all been investigated (Kennedy *et al.,* 1978; Domone, 1990).

(a) Rheology. With high shear mixing, the gel and cement are well dispersed, and at the high water/cement ratios involved the low solids volume leads to a low-viscosity grout. This is important for the grouting of offshore platforms, as the grout has to be pumped over long distances. Concentric cylinder viscometry has shown that, as with many solids suspensions, the grout approximates to a Bingham fluid, i.e two parameters or constants, yield stress and plastic viscosity, are required to describe its flow behaviour.

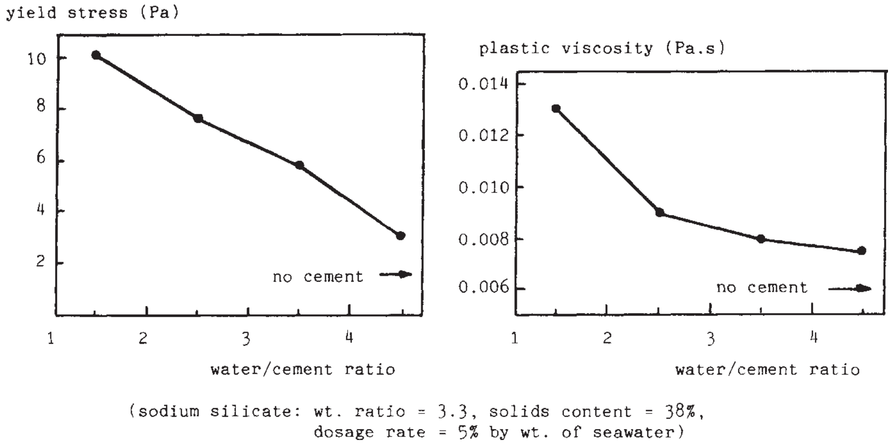

Figure 4.1 Rheological properties of fresh silicate/cement/sea water grouts.

The variation in the Bingham constants with water/cement ratio for grouts made with 5% of sodium silicate by weight of sea water is shown in Figure 4.1. As expected, the addition of increasing quantities of cement increases both constants, but all of the grouts have a low viscosity compared to 'conventional' neat Portland cement grouts of lower water/cement ratios. This facilitates long-distance pumping and flow of the grout into restricted spaces, both of which are often required.

(b) Stability. Despite the high fluidity, the grouts are cohesive and are capable of displacing water with the minimum of interface mixing and dispersion—another important practical requirement. This has been conclusively demonstrated in laboratory and large-scale trials (Kennedy *et al.,* 1978).

The stability of the grout after placing has been measured by observing the segregation and bleeding of a grout column in 50 mm diameter vertical glass tubes, 1.5 m high. Bleed water separation was complete after, at most, 4 hours, and the amount of bleed was strongly dependent on the amount of the sodium silicate. Figure 4.2 shows that at a water/cement ratio of 2.6 a bleed of 2% of the grout height (an acceptably low figure for most applications) could be obtained with a sodium silicate dose of 4% by weight of sea water. The stability of the low-bleed mixes was also apparent from the homogeneous nature of the grout throughout the whole column height. The effectiveness of the sodium silicate in providing stability to the grouts, which have a solids content of only about 10% by volume, was therefore clearly demonstrated.

(c) Strength and modulus. Nearly all the strength of the grouts is due to the hydrating cement, and they have a measurable unconfined compressive strength from one or two days old. Continuous storage in water is essential for

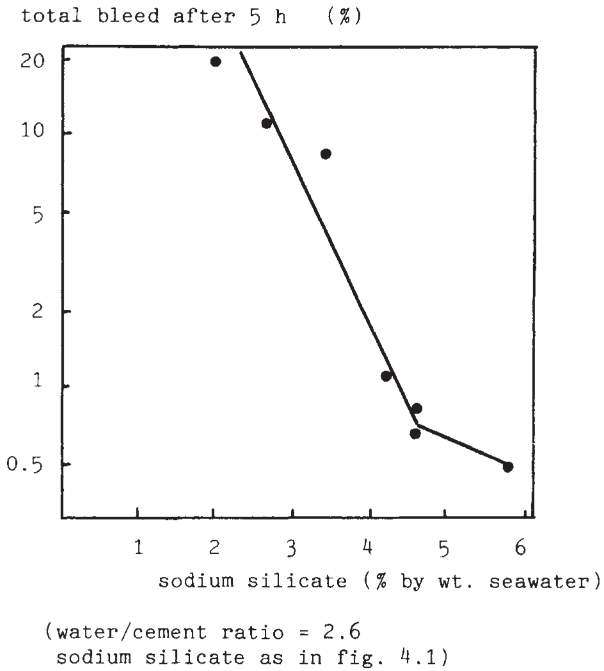

Figure 4.2 Bleed capacity of silicate/cement/sea water grouts (Kennedy *et al.*, 1978).

the maintenance and development of strength, and Figure 4.3 shows the variation in 28-day cube compressive strength with water/cement ratios for two curing temperatures, 8 and 20°C. As with conventional grouts, strength decreases with increasing water/cement ratio.

The modulus of elasticity, measured in a static test, increases with strength (Figure 4.4), which is also similar to the behaviour of 'conventional' grouts at higher levels of strength and modulus.

(d) Long-term properties. Some grouts cubes which were kept for longerterm strength measurements were found to exhibit an expansive form of cracking after several months, as illustrated in Figure 4.5. The cracking was progressive with time, and the weaker, higher water/cement ratio grouts were more susceptible. It also appeared to be more extensive in samples stored at the lower curing temperature of 8°C.

In the sodium silicate/cement/sea water system, it is likely that silicic acid sols provide the initial stability, and that a sodium silica sol is formed by subsequent reaction with the sodium or potassium alkalies present in the cement, or derived from the sea salts in excess of those needed to form the initial gel. The expansive and swelling properties of such gels have been demonstrated by Struble and Diamond (1981), and the process may be

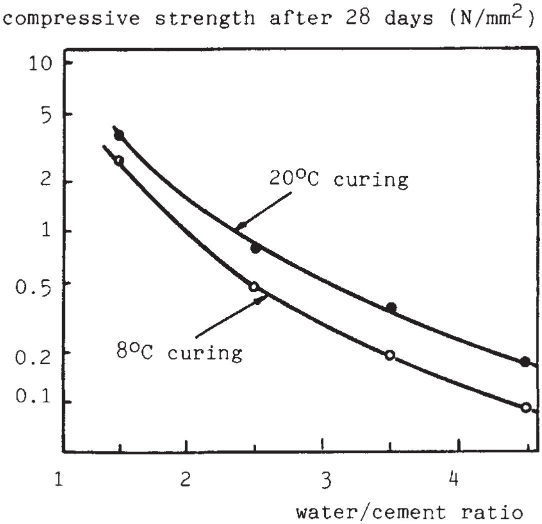

Figure 4.3 Compressive strength of silicate/cement/sea water grouts (Domone, 1990).

similar to that occurring in alkali-silica degradation in concrete, except that, in the case of concrete, the aggregate is the source of the reactive silica. The sodium oxide equivalent (Na_2O+0.658K_2O) of the cement used for the grout in the experimental programme was 0.57%, which is only just below the recommended maximum of 0.6% considered safe for use with potentially reactive aggregates in concrete (Concrete Society, 1987),

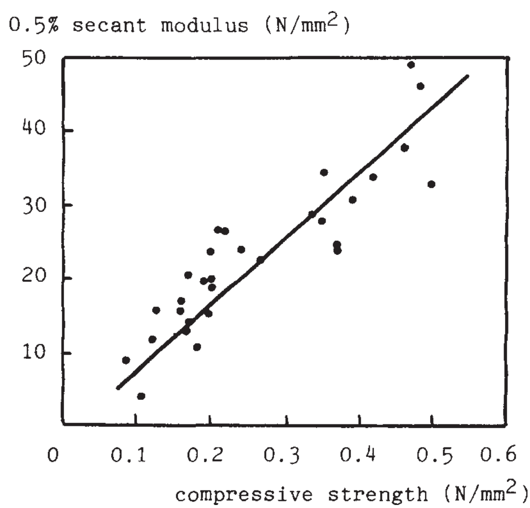

Figure 4.4 The variation of modulus of elasticity with strength of silicate/cement/sea water grouts (Kennedy *et al.,* 1978).

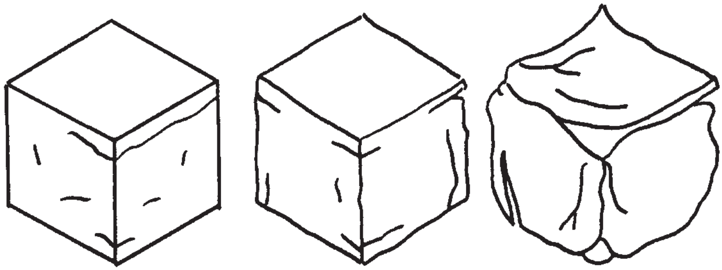

Figure 4.5 Long-term expansion cracking of unconfined silicate/cement/sea water grouts (Domone, 1990).

and therefore only a small contribution of alkalies from the sea salts may cause critical expansion.

However, in the applications described in later chapters, the grouts are used for underwater void-filling purposes, and they will therefore be continuously confined. In such circumstances, any tendency to expand will impose a stress on the surrounding structure, but the low grout modulus means that the magnitude of this stress will be small, and it will also be reduced by creep or stress relaxation within the grout. More importantly, the grout will not be able to crack, and will therefore not degrade. This has been demonstrated by long-term storage of grout cubes cast and stored in perspex moulds under water. After 5 years, the grouts were removed from the moulds, and no sign of cracking could be seen. Also, the compressive strengths (Figure 4.6) showed a significant increase over the 28-day values (Figure 4.3).

(e) Grouts with low-salinity sea water. All the data given above were obtained on grouts mixed with normal salinity sea water, i.e. having a total salt content of 3–3.5% by weight. Some tests (unpublished) have been carried out using water from the Baltic Sea, which had a salinity of 0.65%. In general, similar results were obtained without any modification of the sodium silicate type or content, showing that the degree of salinity is not a major factor.

4.1.3 Silicate/fresh water grouts

In silicate/cement/fresh water grouts, the early hydration products of the Portland cement act as the gelling agent for the silicate, which is added after

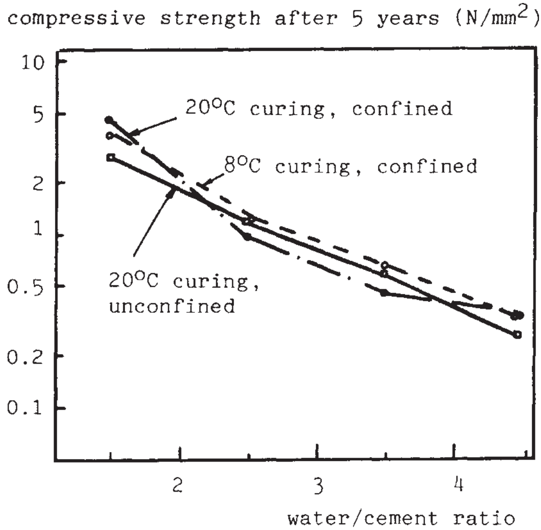

Figure 4.6 Long-term strength of silicate/cement/sea water grouts (Domone, 1990).

the cement. The gelling provides some strength, and therefore the silicate can be considered as acting as an accelerator.

For the Channel Tunnel lining applications described in chapter 9, a blended pfa/Portland cement grout was used, with a pfa:opc ratio of 3:1, and a water solids ratio of 0.45 (Annett and Stewart, 1991). Continuity and flexibility of grouting was essential, and a two-stage grout mix was developed:

1. A grout consisting of the pfa/opc blend, fresh water and a thickening, retarding, anti-washout admixture (coded 802) was first mixed. The admixture was based on a selected stabilised, sugar-reduced lignosulphonate with a polymer-based thixotropic agent. The retarder enabled the grout to be held in a holding tank until required, and the thickening, thixotropic, anti-washout agent ensured stability in the holding tank and immediately after placing, often in running water, behind the lining.
2. This grout was then pumped to the injection point, and a sodium silicate admixture (coded 803) added at the injection nozzle. The resulting grout then gelled rapidly once in place.

Full details of the operation are given in chapter 9. Generally acceptable properties were obtained with dosage rates of 0.5% by weight of the anti-washout (802) admixture and between 2 and 6% by volume of the

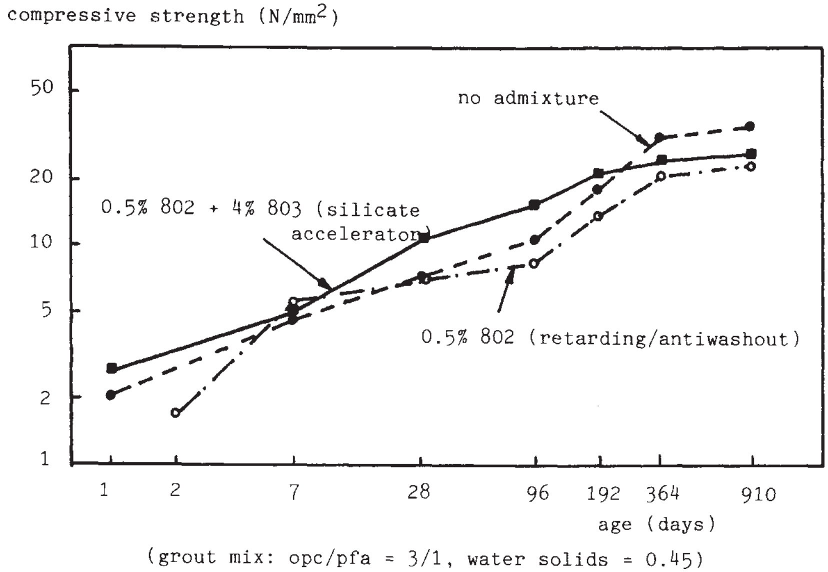

Figure 4.7 Strength development of silicate/pfa/cement grouts for Channel Tunnel lining (data from Fosroc International).

silicate (803) admixture. The most significant quantified properties of the grout were:

1. The viscosity of the grout containing admixture 802 only was low enough for the grout to be pumped with a maximum pressure of 0.5 N/mm^2.
2. The gel time of this grout was approximately 24 h at 18ºC. The addition of the 803 accelerator reduced this to approximately:
   50 min with 3% 803 by volume
   20 min with 4% 803 by volume
   1 min with 5% 803 by volume
3. The strength development of grouts with no admixture, with 802 alone and with 802 and 803 together are shown in Figure 4.7. At early ages, 1–2 days, the retardation from the 802 and the acceleration from the 803 are apparent. The combined admixtures produce the highest medium-term strength, and the delayed nature of the pozzolanic reaction is probably the cause of the significant strength gain of all three grouts at ages beyond 1–2 months. At later ages, 6 months and more, both grouts containing admixtures produce lower strengths than the plain grout. Shorter term tests at a water/solids ratio of 0.6 indicate similar behaviour at lower strength levels, showing that a range of grouts to suit individual circumstances could be produced.

4.2 High alumina cement grouts

The chemical and physical properties of high alumina cement (hac) and its hydration products were described in chapter 1, and the resulting properties of hac grouts briefly mentioned. In this section, the properties, and the effect of admixtures and cement replacement materials on these, will be discussed in more detail,

4.2.1 Fresh properties

The rheology of freshly mixed hac grout is broadly similar to that of Portland cement grouts, and both can be considered as Bingham fluids. Banfill and Gill (1986) have shown that hac grouts have yield values and plastic viscosities slightly lower than opc grouts of the same water/cement ratio, at least in the test range of 0.3 to 0.4. The yield values showed the greater reduction, a result confirmed by Domone and Thurairatnam (1988) who tested grouts with water cement ratios ranging from 0.3 to 0.6. The greatest reductions in the yield value occurred at the lower water/cement ratios, whereas the reduction in plastic viscosity increased with water/ cement ratio.

4.2.2 Setting

The setting time of hac grouts is strongly dependent on temperature. Increasing the temperature from 5ºC results in an increase in setting time until a sharp maximum is reached at about 30ºC (Figure 4.8). The explanation for this behaviour is not clear but is probably due to differing rates of the different hydration reactions that occur with varying temperature. However, for temperatures up to about 20ºC, the setting times are broadly similar to those of Portland cement mixes, allowing hac grouts to be mixed and placed with similar equipment and methods.

4.2.3 Heat of hydration and strength gain

One of the most useful features of hac mixtures is the rapid gain of strength once setting has occurred (Figure 1.17). This is, however, accompanied by a very rapid hydration exotherm. For example, Jefferis and Mangabhai (1990) have measured a peak temperature of 129ºC in 100 mm grout cubes stored in air at room temperature. Any unreacted mix water then boils and the resulting steam pressure may disrupt the grout mass. In general, therefore, water cooling will be necessary to ensure that excessive temperature rise does not occur.

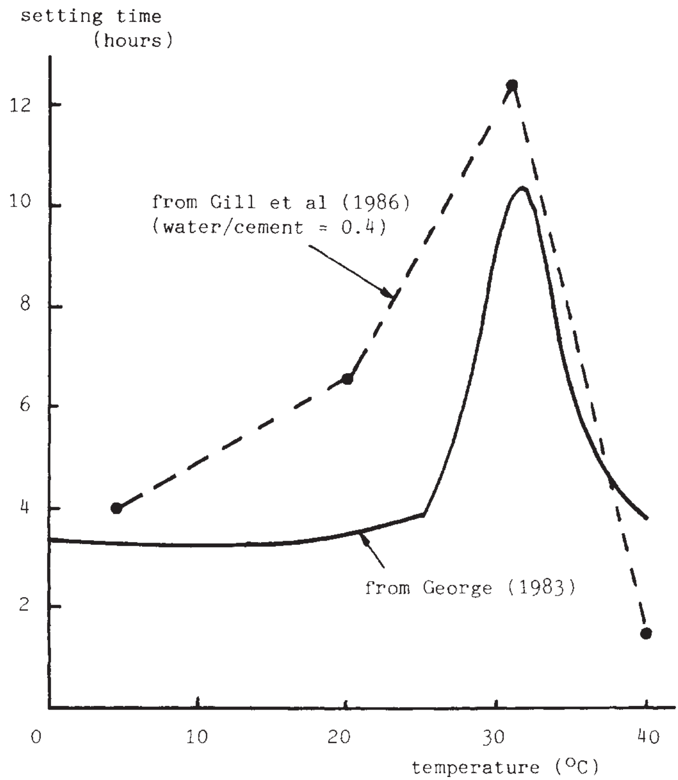

Figure 4.8 The effect of temperature on the setting time of hac grouts.

Also, high thermal conductivity (e.g. steel) formwork must be used as otherwise the cooling will be ineffective, and section thicknesses should be severely limited to ensure effective cooling throughout the grout mass. Plastic moulds should not be used for the preparation of tests cubes as the low thermal conductivity of plastics will seldom allow effective cooling.

The high initial temperatures may also accelerate the conversion processes, which occur during prolonged storage at elevated temperatures and lead to loss of strength (see section 1.8). The quantitative effects of conversion are illustrated in Figure 4.9 which shows the effect of storage temperatures ranging from 5 to 40ºC on hac mortars (cement/sand/water =1.0/1.5/0.4) stored for up to 1 year. The 70.7 mm mortar cubes were monitored by ultrasonic pulse velocity and the pulse velocity values converted to strength values using a correlation obtained from calibration tests, thus giving the strength development curves shown. At 5ºC no loss of strength and no conversion occurred. At 20ºC, conversion resulting in loss in strength started after about 3 months, at 30ºC after about 2–3 days, and at 40ºC all the conversion appears to have taken place before the 1 day strength

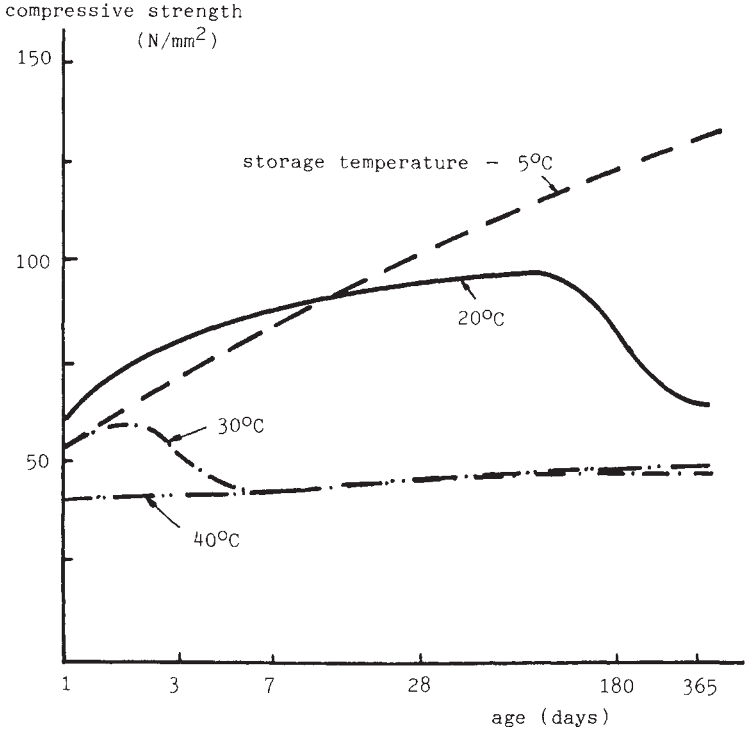

Figure 4.9 The effect of storage temperature on the strength development of hac mortars (Gill *et al.*, 1990) (cement:sand=1:1.5, water/cement ratio=0.4).

measurement. The conversion results in a loss in strength of more than 60%. There is a small gain in strength after conversion.

4.2.4 Elastic modulus

Few data have been published on the elastic modulus of hac grouts. Work by Wimpey Laboratories (1977) shows that for strengths above about 50 N/mm^2 the elastic modulus/strength relationship is approximately the same as for Portland cement grouts (Figure 3.14). For lower strengths the elastic modulus of an hac grout may be lower than that of an equivalent strength Portland cement grout.

4.2.5 Effect of admixtures

Superplasticizers are effective in increasing the fluidity of hac grouts. The magnitude of the increase may be higher if sea salts are present, for example in the mix water (Baker and Banfill, 1990), but, for the reasons explained in chapter 1, the use of sea water for mixing is not recommended.

There are, however, potentially harmful side effects from using superplasticizers with hac (Gill *et al.,* 1986, 1990):

(a) an excessive retardation of set at low and normal temperatures (but a reduction of the excessive setting time at 30ºC shown in Figure 4.8);
(b) a reduction in the subsequent strength development.

Citric acid has also been found to act as a retarder, and calcium hydroxide and lithium salts, notably the chloride, carbonate and citrate, all act as accelerators (Sharp *et al.,* 1990). A lithium based accelerator can be used in combination with a superplasticiser to overcome its retarding effect (Gill *et al.,* 1986).

As with the superplasticizers, many of these admixtures also alter the subsequent strength characteristics of the grout (Gill *et al.,* 1990) and therefore thorough testing of all possible effects must be undertaken before an admixture is recommended for use.

4.2.6 Effect of cement replacement materials

(a) Condensed silica fume (csf). The extremely fine particle size and the effect of csf in Portland cement grouts has been described in chapters 1 and 3. When incorporated in hac grouts, csf accelerates the setting and strength gain process, probably by acting as nucleation sites for the deposition of hydration product.

The retardation effects of a superplasticizer can be reduced, particularly at low water/cement ratios when a relatively high dose of superplasticizer is required to provide fluidity. However, there is a consequent increase in the rate of heat output. Figure 4.10 shows data of rate of heat output and strength gain for a grout with a water/solids ratio of 0.25.

There is also some evidence that the csf reduces the rate of conversion of the hac during curing at temperatures at least up to 65ºC (Bensten *et al.,* 1990).

(b) Ground-granulated blast furnace slag (ggbs). Grouts made with mixtures of ggbs (see chapter 1) and hac have been shown not to suffer from the strength loss associated with the conversion of neat hac grouts (Majumdar *et al.,* 1990). Such grouts do not have the rapid early strength gain of the hac grout, but show no losses due to conversion, and a continuing gain of

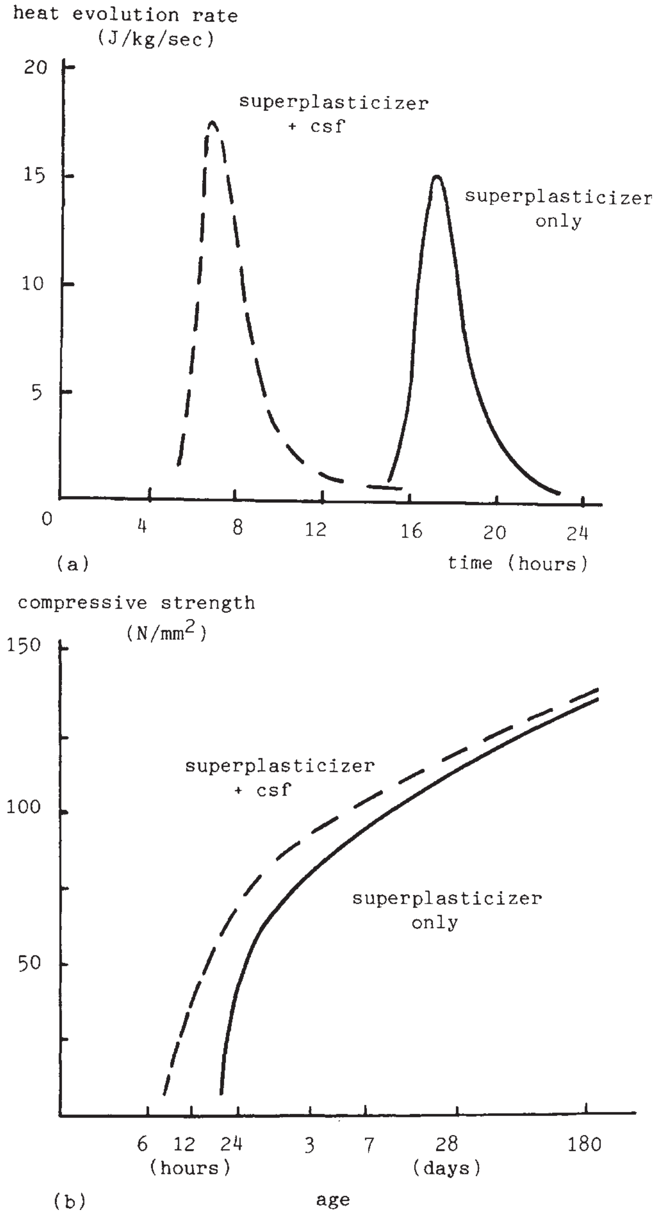

Figure 4.10 Properties of superplasticized hac grout with and without condensed silica fume: (a) rate of heat output during hydration, (b) strength gain (Domone and Thurairatnam, 1988) (water/solids=0.25, superplasticizer=1.44% by wt cement, csf=10% replacement of cement, curing temp=8°C).

strength. As might be expected, the rate of heat output compared to neat hac grouts is also reduced.

It has been shown that the early hydration products in the hac/ggbs system are similar to those with neat hac, but an alternative hydration product called stratlingite (C_2ASH_8) is then quickly formed; this is not susceptible to conversion (Fentiman *et al.,* 1990).

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Part Two
Applications

5 Grouted connections for offshore structures

S.MOXON

5.1 Introduction

Offshore structures for the exploitation and production of petroleum products can be divided into three main types:

- concrete gravity platforms
- steel jacket structures
- subsea templates.

Concrete gravity platforms (Figure 5.1), remain stable on the sea bed by their self-weight, as their name suggests. The base section consists of a number of caissons which, in addition to providing the gravity base, also provide chambers for ballast or oil storage. Two, three or four of these chambers extend into hollow legs which support the topsides' facilities. The caissons and the legs are normally constructed by slipforming and it is usual to move the base into deeper water for slipforming the legs, ballasting it down as the work proceeds to restrict the working height. The whole structure will finally be ballasted down to allow the topsides to be floated over it and connected to the legs in sheltered waters. After connection of the topsides the whole is deballasted to a suitable draught for towing to its offshore location. Once on location the structure will be ballasted down to the sea bed and the voids between the sea bed and the base of the caissons will be filled with a cementitious grout, to ensure even distribution of loading over the whole base area despite the inevitably uneven seabed.

Although termed concrete platforms, these structures contain as much steel, in the form of passive reinforcement, prestressing cables and ducts, as a medium-sized steel jacket. The great advantages of concrete structures are the durability of the material in sea water, the storage capacity they contain, and the reduction in installation time offshore.

Steel jacket structures (Figure 5.2) are not able to remain on the sea bed by gravity alone, but need additional support. This is achieved by tubular steel piles driven through the legs, or through skirt sleeves attached to the legs, into the sea bed (Figure 5.3). The piles are then connected to the jacket either by swaging them to the sleeves or by filling the annuli between piles and leg/sleeves with a structural cement grout. Early jackets used plain tubulars for both piles and sleeves but this connection was

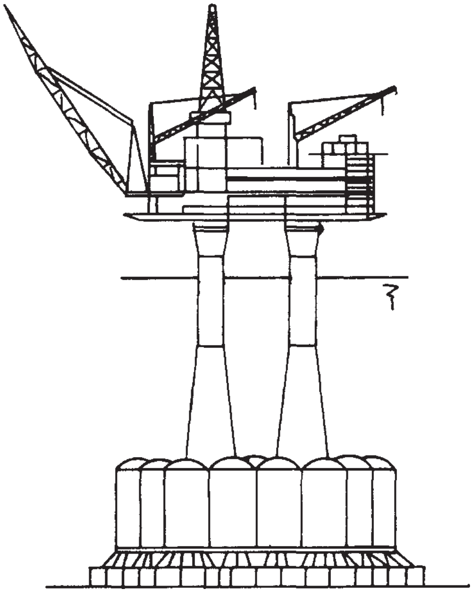

Figure 5.1 A typical concrete gravity platform.

sensitive to the pile surface condition, the roundness of tubulars, eccentricity and possible grout shrinkage; long and inefficient connections were therefore required. By equipping the piles and sleeves with mechanical shear keys over the grouted length most of these problems were attenuated, and more efficient sleeve lengths have resulted.

Subsea templates (Figure 5.4) are used either in conjunction with jackets or with tension leg platforms where a floating deck structure is tethered to the template by rods or tendons. When used for jackets they enable drilling work to be carried out in advance of the jacket installation, and they then act as a guide for locating the jacket which is conventionally piled and grouted. Tension leg platforms have been used where the water depth was considered too deep for a conventional steel jacket mainly on the basis of cost. The templates are provided with a number of sleeves at each corner and are piled and grouted in a similar manner to jackets. The major difference is that there is no structure between the template and the surface to which pipework can be attached; thus remote handling of grouting operations is necessary.

The other uses of structural grouts offshore are for attaching clamps to existing structures either for appurtenances or for the strengthening of joints. Grouts have also been used for infilling of tubulars, particularly above sea level, as a precaution against the buckling of the tubulars in case of fire.

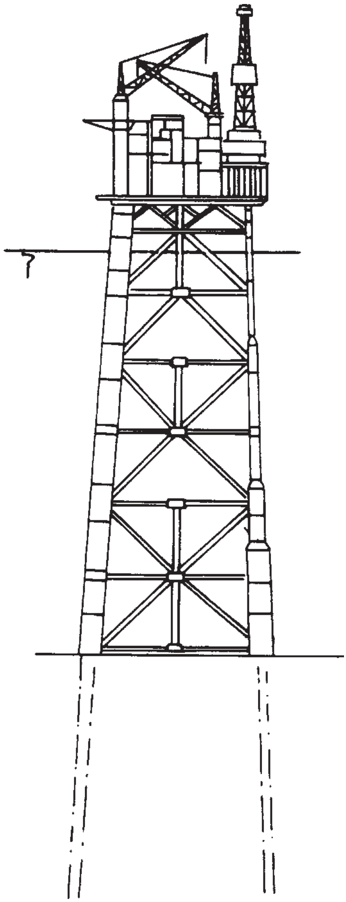

Figure 5.2 A typical steel jacket platform.

This chapter will concentrate on grouts and grouting practice for the second two types of platforms, i.e. steel jackets and subsea templates, which have broadly similar grouting requirements and systems. Underbase grouting of concrete gravity platforms is distinctly different, and is the subject of chapter 6.

5.2 Grout functions and required properties

In both jackets and subsea templates the function of the grout is to form the vital structural connection between two steel members, i.e the pile and the sleeve, and the quality of the bond is clearly dependent on the properties of the grout. It is not possible to measure bond strength in-situ and therefore this must

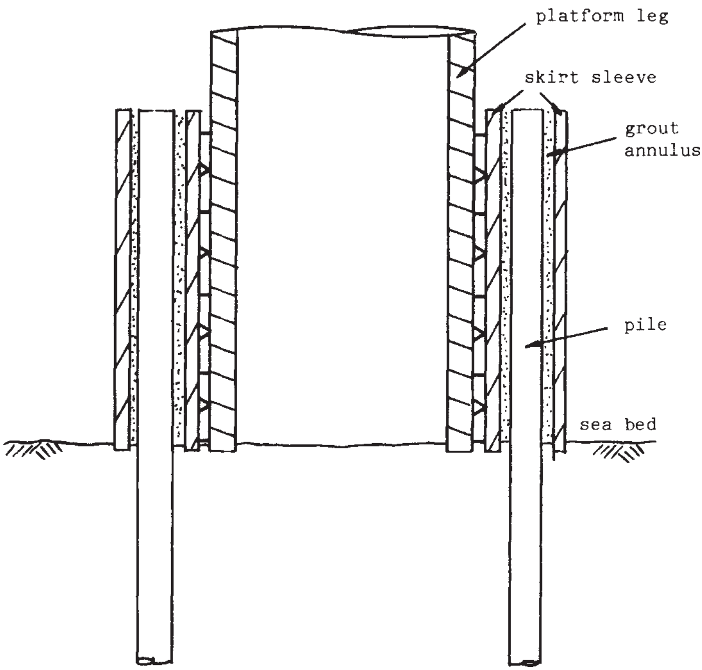

Figure 5.3 The skirt sleeve piling system on a steel jacket platform.

be estimated from another property of the grout. During the late 1970s and 1980s extensive testing of both full-scale and reduced scale pile/sleeve connections was undertaken, for example as reported by Billington and Tebbett (1980) and Carroll *et al.* (1987). A relationship between bond strength, grout compressive strength and pile sleeve geometry (as defined in Figure 5.5), has been established, and incorporated into the Department of Energy (1990) guidance notes as:

$$f_{\text{buc}} = KC_{\text{L}}\,(9C_{\text{S}} + 1100\,h/s)(f_{\text{cu}})^{1/2} \qquad (5.1)$$

where f_{buc} is the characteristic bond strength (N/mm^2), f_{cu} is the characteristic grout compressive strength (N/mm^2), K is a stiffness factor related to the diameters and thicknesses of the pile and the sleeve (see below), C_L is a coefficient depending on the grouted length to pile diameter ratio, i.e L/d_p (it varies from a maximum of 1.0 for L/d_p=2, to a minimum of 0.7 for L/dp greater than 12), C_s is a surface condition factor, h is the minimum shear connector outstand (mm), and s is the nominal shear connector spacing (mm).

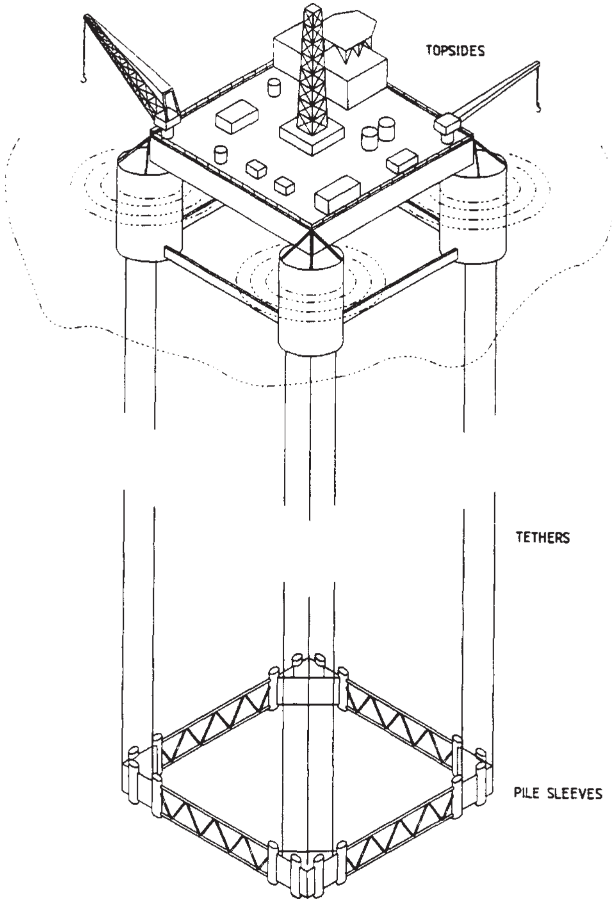

Figure 5.4 A tension leg platform with piling through sleeves on a subsea template.

The stiffness factor, K, is given by

$$K = 1/m(d_g/t_g)^{-1} + (d_p/t_p + d_s/t_s)^{-1} \tag{5.2}$$

where m is the modular ratio of steel to grout.

Thus the characteristic bond strength is directly proportional to the square root of the characteristic grout compressive strength, which is the main parameter used for defining the quality of the grout. The characteristic compressive strength at 28 days is normally used. With a constant quality of materials, this is directly related to the fresh grout (or slurry) density, and therefore measurement of this at frequent intervals during grouting is used for quality control purposes. Details of the methods and instruments employed are discussed in section 5.5.

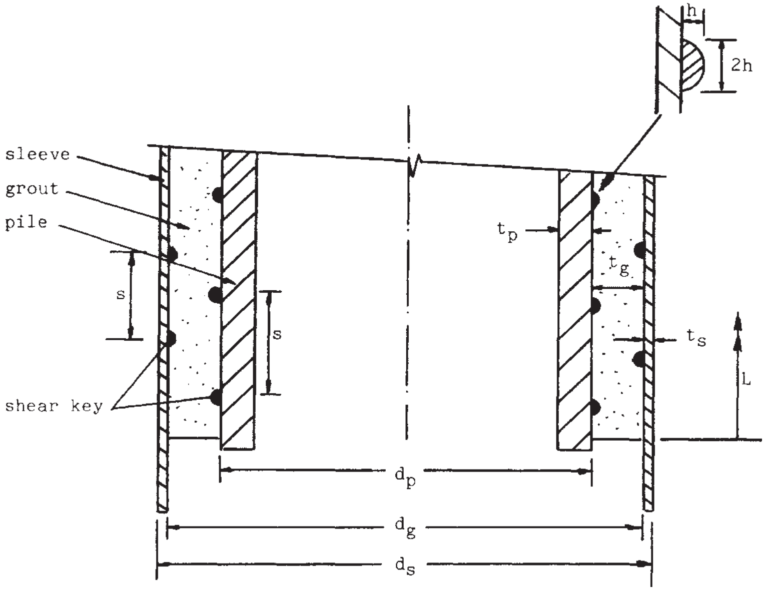

Figure 5.5 The shear key system in a grouted pile/sleeve connection on a steel jacket.

The demands of pumping and placing the grout are not as great as in some other applications such as oilwell cementing, where much greater pipework lengths are involved. The only resistance to grout flow is the friction through the pipework, at the inlet nozzles and in the pile/sleeve annulus. The top of the annulus is usually open and since the column of grout in the pipework is of a density approximately twice that of sea water the pumps are not called upon for a heavy duty. Thus, bond strength is a more important parameter than the rheology of the fresh grout, and the main concerns of the grouting subcontractor are:

- to ensure that the grout can be mixed and pumped at the specified rate, and
- the length of time that the grout can either stand or be recirculated through the mixer and remain in a state such that pumping can be restarted after a break without causing excessive back pressure to be generated.

An important parameter for all applications is the rate of gain of strength with time. For example, this determines the interval required between the placement of a grout plug to replace a failed packer and the grouting of the affected pile, and hence affects the completion of the whole grouting operation. Similarly, it determines the earliest time at which the module support frame and topsides can be placed on the jacket after grouting—an extremely important factor since large offshore lifts are weatherdependent.

In some special circumstances grout properties other than strength may be more important. The two most common examples are:

1. Filling of the tubulars above water entails large volumes of grout without any cooling effect from surrounding sea water, and a standard grout may well generate sufficient heat of hydration to cause thermal cracking within the grout mass. In such circumstances a grout formulation with a low heat of hydration would be used.
2. The grouting of an insert pile placed inside a deep hole drilled into the sea bed. In some foundation beds the use of a standard grout could lead to hydraulic fracturing of the foundation due to the pressure head of the grout itself. A lightweight grout is therefore required.

5.3 Grout constituents and formulations

Offshore pile-grouting practice developed from onshore oilwell practices, and the cement types and the plant used initially were those used for onshore well cementing. Over many years the American Petroleum Institute (API, 1990) had standardised a series of oilwell cements for well cementing, and plant suitable for mixing and pumping these cements in the quantities required was readily available. Thus the offshore grouting industry inherited a background of materials and equipment technology in the first instance.

5.3.1 Cement types *

(a) Ordinary Portland cement. This is not often used for offshore applications in the North Sea except for filling of fabric formwork for grout bags used to support free pipeline spans. This is discussed in more detail in chapter 7.

(b) Oilwell cements. In the early North Sea platforms the Oilwell series of cements was almost universally used. Those most applicable to offshore structures were Oilwell A, Oilwell B and Oilwell G (Bensted, 1983). Oilwell A is intended for use for well cementing from the surface down to depths of 2000 m when special properties are not required. It is available only as an ordinary type. Oilwell B is suitable for the same depth range, but when some sulphate resistance is required. In the USA

*See note on standards on page vii, and descriptions of cement compositions in chapter 1.

it is available in both moderate and high sulphate-resistant types, but only the high sulphate-resistant type has been available in Europe. It is less finely ground than Oilwell A. Oilwell G is almost identical to Oilwell B but the maximum and minimum contents of tricalcium aluminate are specified. This is also a sulphate-resisting cement but it is considered suitable for depths of up to 2600 m.

All the Oilwell series of cements have retarded setting compared to OPC. This is because they were developed for well cementing and needed to remain fluid for a sufficiently long period to be correctly placed downhole.

(c) High alumina cement (HAC). This was initially the obvious choice when high early strength was required, since it develops a high strength in 24 hours. It does, however, have some disadvantages:

1. It can suffer conversion which reduces its strength in some circumstances, although this should not apply to the temperatures experienced in most pile/sleeve situations.
2. It cannot be mixed with sea water, which is an inconvenience offshore.
3. It is incompatible with other cements; even a trace of another cement in HAC or a trace of HAC in other cements will cause a flash set. Storage silos and plant, therefore, have to be used exclusively for HAC.

(d) Encilite. Over the last few years this cement has replaced HAC in the North Sea when high early strength is required. It is a rapid-hardening Portland cement manufactured in Holland to Dutch Standard NEN 3350 (NNI, 1990). Its rate of gain of strength is not as rapid as HAC but is considerably greater than the Oilwell series. It can be mixed with sea water or with other cements (except HAC). It was first introduced to the North Sea by a Dutch offshore installation contractor partly on grounds of cost. Rapid-hardening cements from other countries would be expected to have similar properties.

(e) Blended cements. The most frequently required blend is one which has a lower heat of hydration to reduce the tendency of the grout to crack when placed in relatively large volumes. The most commonly used material is pulverised fuel ash or ground-granulated blast furnace slag. The blend proportions vary, but typically up to 90% of the cement by weight can be replaced with the blending material. The blends are not significantly lighter than a neat cement, but the strength is considerably lower, particularly at early ages.

Lightweight grouts are obtained by using a natural bitumen as the blending material. This is known as gilsonite, and is mined hydraulically in Utah in the USA. It is granular in consistency, black in colour and has a low melting point.

It becomes tacky when held in the hand. It is extremely abrasive and if large volumes are to be pumped then wear on pipe bends will be considerable. The hydraulic mining results in a high percentage of fine material with a particle size of less than 20 μm. These particles increase the water demand so they have to be removed before blending, and because of their lightness this must done by air elutriation rather than by sieving. To ensure that all fines are removed it is necessary to set the cut-off point to 5% more than the permitted fines content, which can result in the loss of a considerable percentage of the raw feed, hence increasing costs.

For offshore applications, blended cements are pre-blended onshore, which makes them all considerably more expensive than neat cement. They are therefore only used when their technical advantages are required.

5.3.2 Admixtures

For onshore grouting applications, and for well cementing both onshore and offshore, a water/cement ratio of about 0.4 is used. For offshore grouting this proportion is still used for HAC-fresh water mixes, but to obtain higher slurry densities (and hence subsequent strength) with the other types of cement, the water/cement ratios are reduced to between 0.34 and 0.36. This clearly increases the viscosity, and with early North Sea operations admixtures in the form of plasticizers or superplasticizers were thought necessary to compensate for this. However, as experience was gained it became clear that the grouts with these lower water/cement ratios could be handled perfectly well without such admixtures, which are therefore no longer in general use.

The one circumstance where an admixture may be used is to accelerate the setting of a grout plug used to compensate for a failed grout packer. The most efficient accelerator is calcium chloride, which is sometimes used in the Norwegian sector of the North Sea. Its use is banned in the UK sector due to the possible corrosive effects of the chloride ions on the steel of the jacket, and thus only chloride-free accelerators are used. However, with recent improvements in packer performance it is often possible to accommodate the relatively few grout plugs within the overall grouting period without reverting to the use of accelerators.

5.3.3 Bleed or free water

Immediately after the grout has been placed, and before setting has started, the mix water that was required to fluidify the mix has a tendency to migrate upwards through the grout, and can form water-filled lenses. These can be trapped within the grout mass on setting, with deleterious effects on overall integrity. Because of this most grouting specifications limit the amount of free water in the grout. However, the standard formulations used in the North Sea have a water/cement ratio sufficiently low for this not to be a major problem. Any excess water collecting on the grout surface tends to be absorbed back into the grout early in the setting and hardening process.

5.3.4 Grout formulations

Grout is normally specified in terms of a characteristic strength at various ages, a maximum free water content and whether admixtures are permitted. From accumulated data it is then possible to select the cement type and the water/cement ratio, and to estimate the resulting slurry density.

The 'standard' grout formulations which have evolved in North Sea practice are given in Table 5.1; their strength development characteristics are shown in Figure 5.6. Of interest are:

1. The encilite grout, although having a higher water/cement ratio, achieves the same 28-day strength as the Oilwell B mix, but its rate of gain of strength is higher
2. The 90/10 blend of ggbs/cement produces a low heat grout with a lower strength and a lower rate of strength gain than a neat cement/ water mix. However, the decrease in strength only becomes marked at a replacement level of over 70%.

Oilwell A cement has not been in use in the North Sea for a number of years but its American equivalent, ASTM C150 (ASTM, 1989) Type 1, is used in the

Table 5.1 Typical North Sea grout formulations

Cement type	w/c by wt	Slurry s.g.	Slurry density (lb/US gal)	Yield (m^3/t cement)
Oilwell B	0.36	2.04	17.01	0.67
HAC	0.40	1.98	16.5	0.71
Encilite	0.39	1.99	16.6	0.70
Blended cements:				
Oilwell B/ggbs 10/90	0.37	1.93	16.1	0.71
Oilwell B/gilsonite 100/45	0.44	1.53	12.8	1.18

Notes:
1. Sea water is used for mixing except in HAC grouts, for which fresh water must be used.
2. The Oilwell B/gilsonite contains 2% superplasticizer by weight of cement.
3. Strength data in Figure 5.5.

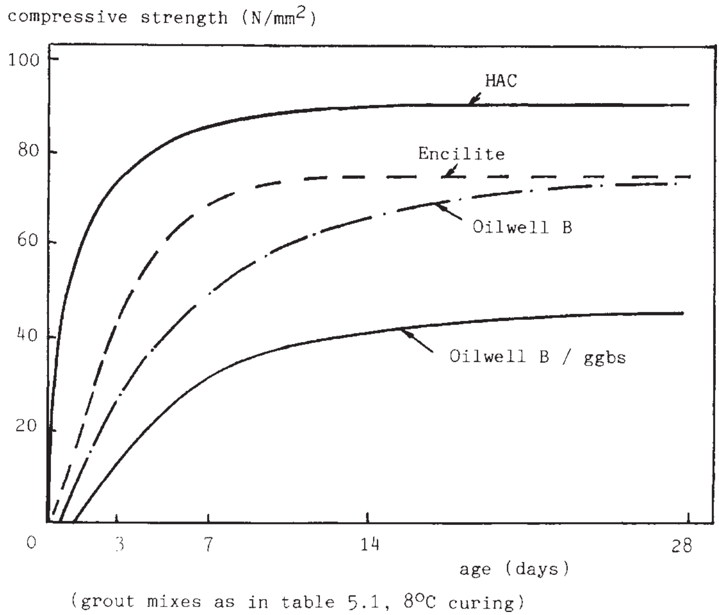

Figure 5.6 Strength development of typical grouts.

Gulf of Mexico. ASTM Type 2, which is the equivalent of API Oilwell B, is also used there. Oilwell G cement has found some use in the North Sea but API Spec 10 (API, 1990) shows that it is so little different from Oilwell B that data for one can be assumed to be valid for the other. However, since the specific gravity of Oilwell G is 3.21 compared to 3.18 for Oilwell B, a slurry density of 2.04 would be obtained with 37% wt of sea water.

5.4 Plant and equipment

The pile/sleeve grouting on a jacket installation is normally carried out by a specialist grouting subcontractor employed by the offshore installation contractor (OIC) for the season. The grouting is normally one of the last platform installation operations and it is therefore on the critical path. Any delays will be expensive, and there is always pressure on the grouting subcontractor to complete within a limited timescale.

5.4.1 Mixing plant

The size and type of plant required is clearly determined by the overall size and scope of the operation and the necessity for completion in as short a time as

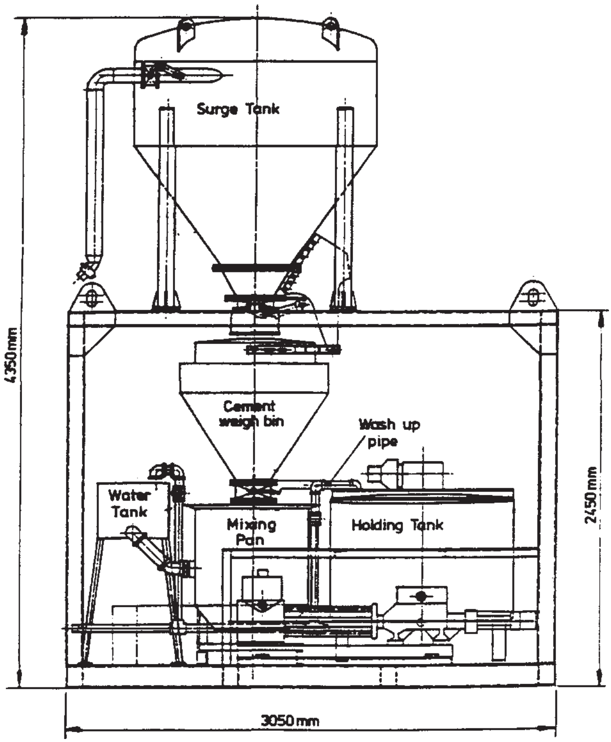

Figure 5.7 A typical small bulk grout mixer.

possible. The subcontractor's plant remains on the OIC's work barge throughout the season, and thus capacity of the plant is that required for the largest operation. Smaller batch mixing plants are only mobilised for smaller repair/strengthening jobs which are undertaken from small work vessels, supply boats or producing platforms where the available space is at a premium.

Figure 5.7 shows a typical small bulk mixer. This is transported offshore in two parts, with the surge tank being mounted at the workplace. The surge tank is supplied with cement from bulk tanks by dry compressed air, and the cement then feeds into the weigh bin and mixing pan by gravity. A holding tank with an agitator is used to provide some storage capacity for mixed grout. This is drawn off as required by a reciprocating pump mounted on the same skid. An even simpler type of mixer is available when the quantity of grout required in any one operation is small. This has no surge tank or cement weigh bin but has a table over the pump unit where a pallet of bagged cement can be placed for hand loading into the mixing pan.

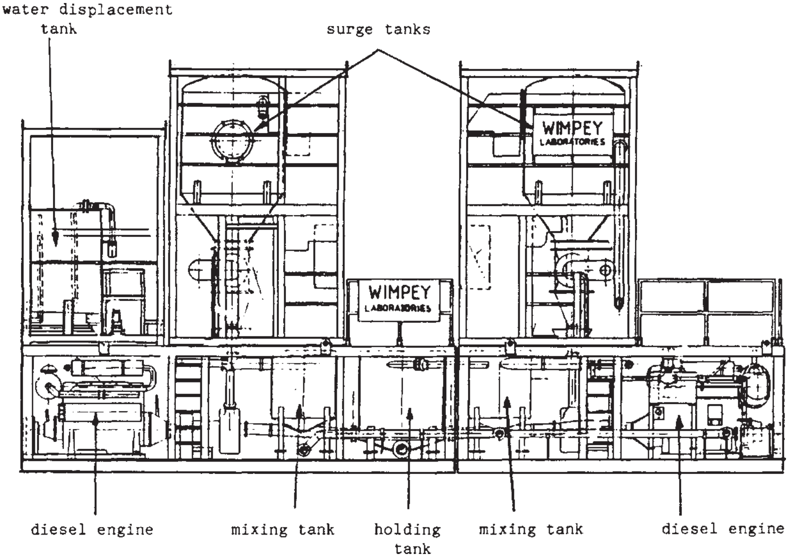

Figure 5.8 A typical large recirculating jet mixer as used for a major platform installation.

Figure 5.8 shows a large-scale recirculating jet mixer typical of that used in a platform installation. This plant is transported offshore in five parts: two base units, two surge tanks and a water displacement unit. The only difference between the base units is that the larger one houses the water displacement tank and the holding tank is common to the two units. Thus the whole of the mixing equipment is duplicated; whichever half is being used feeds into the common holding tank. The pump units, not shown on this drawing, are also duplicated and are located so that the suction hose from each can be easily connected to the holding tank outlet. Such a spread is capable of mixing and pumping up to 1.5 m^3/min of grout with a specific gravity of up to 2.11 (17.6 lb/US gal). This capacity could probably be increased by operating both mixers simultaneously but the second mixer is provided as a standby in case of mechanical problems and the additional capacity is not required in most installations. The mixing skid and pumps are augmented by a number of cement storage silos, a stores/workshop container and a laboratory container with a chilled water tank. The whole grouting spread requires a deck space of 15×15 m.

5.4.2 Grout distribution systems

In North Sea operations the standard grout conduits from the pumps to the inlets on the sleeves are 2 in. nominal bore, although 3 in. is commonly used in the USA. The systems installed on the steel jackets for carrying the grout from the surface to the pile sleeves show many variations but they can be divided into fixed systems and remotely operated systems. In both the arrangement at the pile sleeves is the same. At the bottom of the sleeves, just above the inflatable packer which closes off the annulus, half of a 4 in. nominal bore pipe will form a ring main around the sleeve with a number of penetrations, typically three or four, to allow the grout into the sleeve. This half-pipe will be routed to the top of the sleeve. A similar arrangement with a second ring main approximately 1.5–2.0 m higher up the sleeve forms the secondary inlet to each sleeve. The reason for using a half-pipe welded to the sleeve, which is now almost universal practice, is that it is better able to withstand the strains and accelerations transferred to the sleeve from pile driving.

The differences between the fixed and remotely operated systems occur above the top of the sleeve. The fixed system will have fixed 2 in. steel pipe the whole way from the top of the sleeve to the grouting stations above sea level at the tops of the legs. There may be a single pipe from each inlet to the surface, but since installed pipework is expensive some savings can be made by using fewer pipes up to the surface, each having diverter valves so that they can service more than one inlet. These diverter valves are operated by dropping a suitably sized brass ball down the line and seating it by pressurising the pipe. Multiple-stage diverter valves can be used on jackets in deep water so giving a considerable cost saving.

Remotely operated systems show a similar diversity but these again have two basic divisions. The first type brings all the primary and secondary grout pipes from one leg to one of the horizontal tubulars at a slightly higher level. Each pipe is then fixed in a vertical position and fitted with the male half of a remote grout connector (Figure 5.9). The spacing between the individual pipes is kept as small as possible commensurate with satisfactory access for the female half connector. The female half connector trailing the flexible grout hose from the surface is then attached and locked on to each line as required by a remotely operated vehicle (ROV). Each time a changeover is required the ROV makes the transfer. The second type brings the individual steel lines from the primary inlets on the legs to one or more subsea locations where a valve manifold is mounted (Figure 5.10). The manifold is fed by a single inlet pipe fitted with a male half grout connector. Thus the ROV only needs to make one connection of the surface grout hose but then has to operate the valves as necessary. The secondary lines from each sleeve may be brought to a similar manifold but it is more usual to have the individual secondaries brought to a central location and have each equipped with a male half connector.

Many permutations are available but the primary and secondary lines differ because, unless contingency measures have to be used, the secondary lines will not be employed. An additional variation is the use of divers to connect the female connector and/or to turn valves. This is only possible when divers are

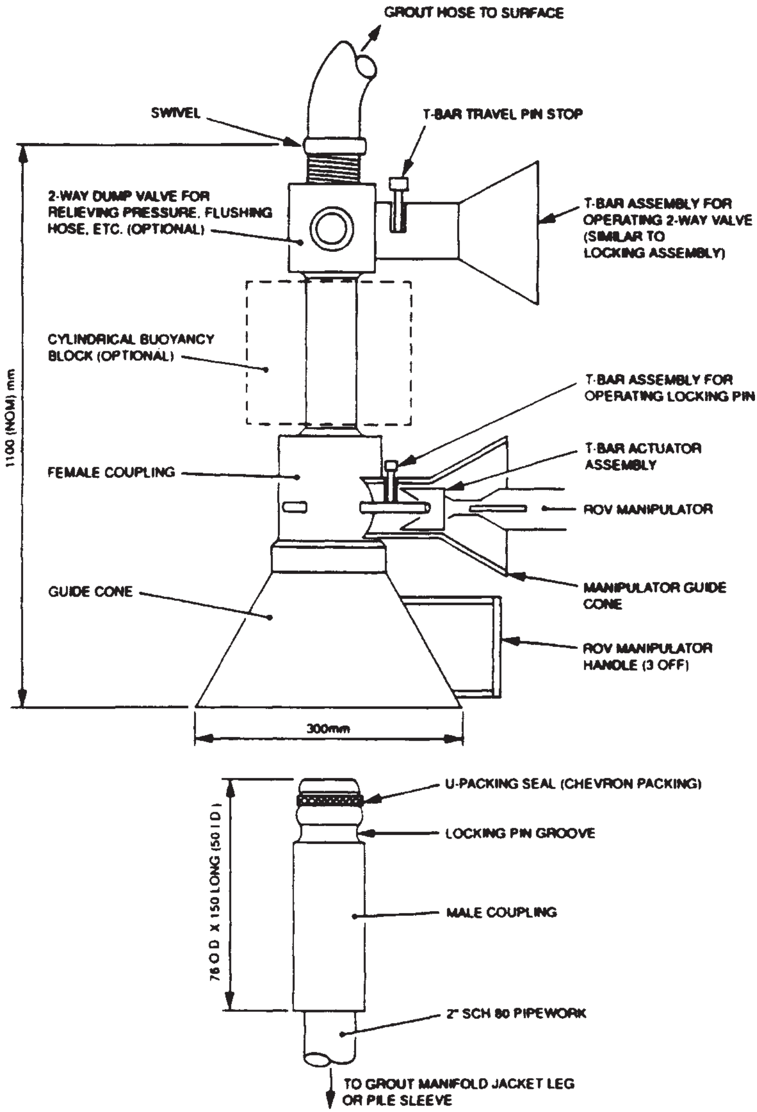

Figure 5.9 A typical remote grout pipeline connector.

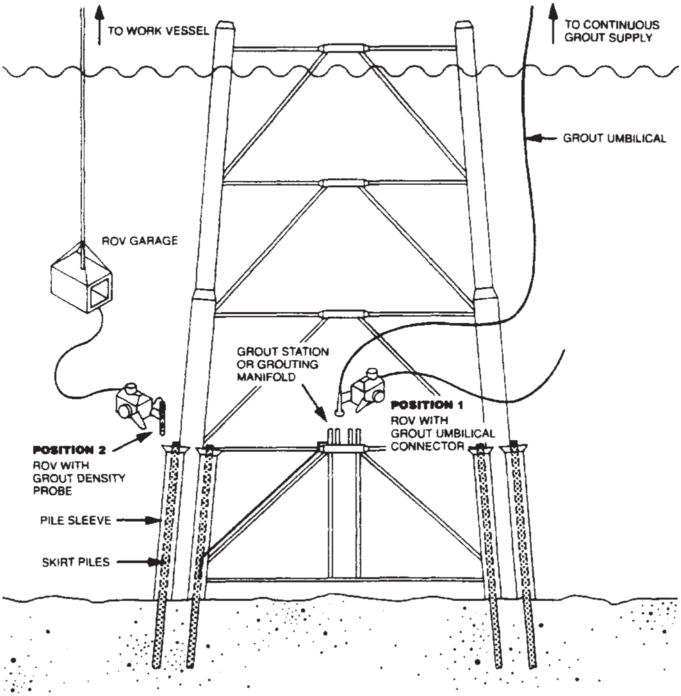

Figure 5.10 Pile grouting and density monitoring with a remotely operated vehicle (ROV).

available and if the depth of the platform is within the safe diving range. The current tendency is to eliminate the use of divers as far as possible on safety grounds.

5.5 Quality control procedures

As the cement grout is the structural connection between the foundation and the jacket it is essential that its quality is uniform and satisfactory. Initially, information such as that presented in sections 5.2 and 5.3 is used to design the mix for a particular application. As mentioned in section 5.2, the parameter that is related to the characteristic bond strength, and which can be readily and accurately measured offshore, is the slurry density. An onshore trial mix is normally required, both to demonstrate the slurry density and to provide test cubes to confirm that the mix achieves the required strength.

The mixing plant is fitted with densometers to give a continuous record of the density of the slurry while being mixed and while being pumped to the jacket. The most commonly used densometer operates on the principle of the attenuation of gamma radiation passed across the pipe when the grout is flowing; denser grout causes greater attenuation. The type of detector used gives a continuous record of density.

After assembly of the plant offshore, a further full-scale trial mix or a series of smaller scale mixes are carried out to calibrate the densometers on the mixers. The calibration is made over a range of densities around the specified value, against density measured with a pressurised slurry density balance, API standard apparatus (API, 1990). A further check is made using the balance each time grout mixing is started. During mixing, the mixer driver uses the densometer as a guide, with regular checks against the density balance.

As each sleeve is grouted a number of samples (usually three or four depending on the volume of the sleeve) of the grout being pumped are taken. From each of these samples the slurry density is measured and a series of 75 mm cubes manufactured. Each cube is allocated a unique identification number which correlates it to the platform, the leg and sleeve, the time of casting and the slurry density. Each cube is covered by a glass plate and stored in a tank of water maintained at the temperature of the position of the sleeves, typically 8ºC for water at depth worldwide. At least one cube from each set will be tested at 28 days, the others will be tested at ages detailed in the specification. These will typically include 3 and 7 days but may also include an earlier age to provide confirmation that the grout has reached sufficient strength to allow the topsides to be set on the jacket.

The mean strength of all the cubes tested at any age must exceed the specified characteristic strength by either

(a) $1.64[0.86 + \sqrt{(2/n)}].s$ for $10 \leqslant n < 100$, or
(b) $1.64s$ for $n \geqslant 100$

where n is the number of test results (which should be not less than 10) and s is the standard deviation calculated from n results.

Thus the grout that is mixed and pumped to the annulus is continuously monitored for quality by means of its density, and the resulting strength of this grout is confirmed by cube testing.

The only further confirmation required is that the grout in the pile sleeve is of the same quality as that pumped down the lines. The only way that a difference can occur is if intermixing with sea water takes place, thus reducing the density of the slurry. This is only likely to occur if the distance between the primary grout inlet to the sleeve and the top of the packer is excessive. On earlier platforms in shallow water, or where divers were on site, samples would be obtained by means of top packers and return lines to the surface or by a diver filling a syringe with grout from the top of the annulus and returning it to the surface for density measurement. For platforms in deeper water neither of these systems is feasible so remote grout density monitoring systems were devised. These, in general, are based on the attenuation of a beam of gamma radiation through the grout, the degree of attenuation being related to density by calibration. Two systems are in common use: one is fixed to the platform while the other is positioned where needed by a remotely operated vehicle (ROV). In the fixed system, gauges are mounted on an overflow pipe near the top of each sleeve and their signal is transmitted to the surface by cables passing through conduits attached to the jacket. In the ROV system (Figure 5.10) the probe is deployed into the overflow pipe by the ROV and the signal to the surface is transmitted through spare cores in the ROV umbilical cable. For particular applications the hard-wire link can be replaced by an acoustic telemetry link or by a winch-deployed cable from the surface.

With these quality control procedures it is clearly possible to effectively control the quality of the grout and ensure that its structural function is fulfilled.

5.6 Case histories

The following interesting examples of offshore grouting practice in the North Sea are separated by a period of 13 years, and illustrate some of the developments that have taken place in this time.

The earlier example, from 1979, is a 32-pile jacket platform with conventional grout pipework to the surface and a fixed grout density monitoring system. The grout composition used was a 100:34:2 Oilwell B cement/sea water/chloride-free superplasticizer mix. The specification requirements were a 3-day strength of 22 N/mm^2 and a 28-day strength of 41.5 N/mm^2. The mix had a theoretical specific gravity (sg) of 2.039 and the minimum sg of slurry which could be pumped to the sleeves was 2.021. On this installation the eight crutch piles (the inner pair of each eight pile cluster on the legs) were driven and grouted first to give overall jacket stability in the design storm conditions. After a break, to ensure that pile driving did not interfere with the setting of the grout, the remaining piles were driven and grouted. During the grouting of the 32 piles the slurry sg varied from 2.025 to 2.085 with a mean of 2.047 and a characteristic value of 2.028.

The 3-day cube results showed a mean strength of 29.7 N/mm^2 and a characteristic strength of 24.1 N/mm^2; the corresponding 28-day strengths were 67.7 and 60.3 N/mm^2 respectively. This gives a bond strength 20% higher than that used in the design calculations. The distribution of the 28-day strength results is shown in Figure 5.11 (a). The grout densities measured at the top of the pile sleeves showed excellent agreement with the recorded

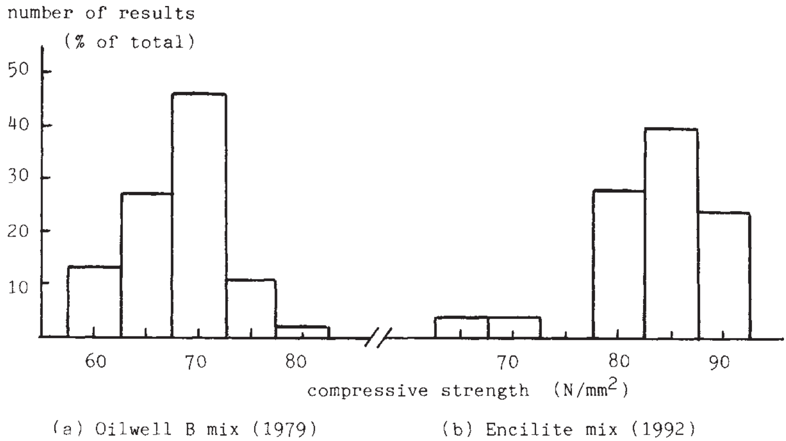

Figure 5.11 Distribution of 28-day cube compressive strengths of grouts from case studies.

density of the slurry when pumped, confirming the quality of the grout in the sleeves.

The second jacket, installed in 1992, is a four-leg structure with a pair of skirt piles at each leg. The grout pipework was again conventional to the surface but the grout density monitoring at the sleeve was by means of a remote probe inserted into the pile-sleeve annulus by an ROV. The specified grout was a 100:36 encilite cement/sea water mix with a required characteristic strength of 35 N/mm^2 at 3 days and 60 N/mm^2 at 28 days. The mix has a theoretical sg of 2.03 and the range observed during grouting was from 2.02 to 2.04. At 3 days the mean strength was 44.73 N/mm^2 and the characteristic strength was 35.65 N/mm^2. The range of 28-day strengths results was from 67.2 to 90.7 N/mm^2, with a mean of 83.54 N/mm^2, and a characteristic value of 73.1 N/mm^2. The distribution of the 28-day results is shown in Figure 5.11(b). In this case the measured characteristic value gives a bond strength 10% higher than the design value.

There are a number of noticeable differences between these jackets. The second, typical of those currently being installed in the UK sector of the North Sea, is a lift-installed jacket with a weight under 10 000 tonnes. Also, the grout specified has a much higher cube strength (and therefore bond strength) than the first example. These two factors, together with improvements in grouting techniques, mean that the number and length of sleeves is reduced so that grouting takes much less offshore time than previously. Thus, it is rarely necessary to drive and grout piles in two stages. However, the quantity of grout used is much less and therefore the frequency of sampling has had to be increased in order to ensure that sufficient results are available for a proper statistical analysis. Much of the progress that has been made in grouting over the last decade can be attributed to quality assurance.

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6 Underbase grouting

P.GEORGE

6.1 Introduction

Underbase grouts are used below the lowest element of a structure to provide a continuous structural link between the structure and its foundations. In the majority of applications this will mean a material formulation consistent with or better than that of the founding stratum, providing durability and long-term dimensional stability.

An early and continuing example of underbase grouting is the use of normal strength grouts (30–60 N/mm^2 compressive strength at 28 days) as void fillers below machine foundations, pre-levelled foundations and stanchion plates. These examples normally involve small quantities of grout, and do not usually present any significant mixing or placing problems.

On a larger scale, mud-jacking of large bases, such as oil tank foundations, represents a form of underbase grouting where the use of cementstabilised drilling mud provides a pressure-sustaining medium which can be easily mixed and placed using existing technology. Grouts formulated for mud-jacking activities were not developed into larger void-filling systems, but were placed by pumping in relatively small quantities.

It was not until the advent of the North Sea oil and gas industry and the use of gravity structures in the 1970s that the need for large volume underbase grouting activities became common (FIP, 1979). The grouts are characterised by their relatively low crushing strength and large structural void-filling capacity in addition to their consequent high production and placement rate capability. This chapter concentrates largely on this specific application of grouting technology.

Gravity structures are designed with steel or concrete skirts which penetrate the sea bed and provide:

- increased lateral stability
- reduced potential for scour and subsurface erosion by penetrating the high erosion-risk zone (in some instances the skirts have been designed to penetrate through weak surface soils, to enable the structure to bear on deeper and stronger founding layers).

The normal installation procedure for a gravity structure involves:

- towing the floating structure offshore and orientating it over the site in correct position while still afloat (Figure 6.1(a))

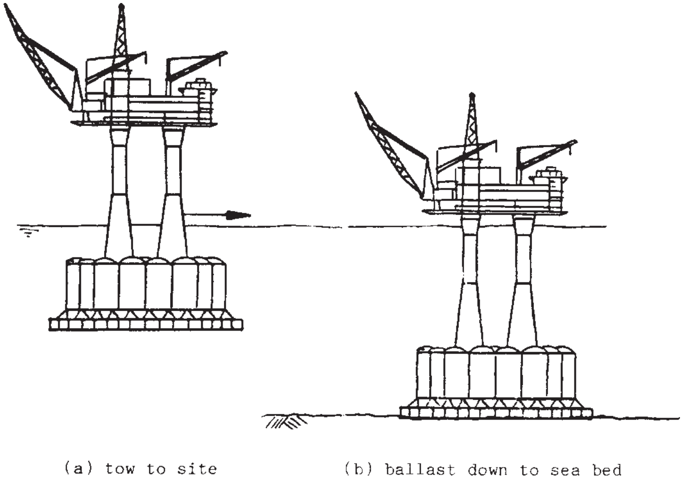

Figure 6.1 Installation procedure for a concrete gravity offshore platform.

- ballasting the structure, normally by flooding the base caisson with water, so that the skirts penetrate the seabed to the required depth (Figure 6.1(b)); on uneven seabeds differential ballasting may be required to ensure that the structure remains level
- grouting the remaining void between the base and the seabed; this may be 1–2 m deep, and stretch for the whole of the base area.

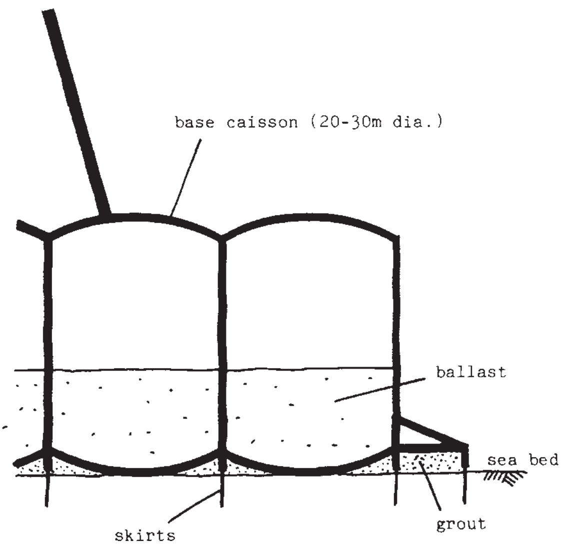

Figure 6.2 Foundation system of a typical Condeep concrete gravity platform.

In most instances the underbase area is divided into a number of isolated compartments. For example, in the Norwegian Contractor's Condeep design the base consists of a series of cylindrical caissons, the walls of which form the skirts. The base of each caisson is an inverted dome cast within the skirts, which therefore forms the upper surface of the underbase grout (Figure 6.2).

Other designs, such as those of Doris and Technomare, have consisted of a similar intersecting skirt system, but with flat base slabs between them.

6.2 Function and required properties

The functions of the underbase grout in offshore gravity structures are as follows:

- to obtain continuity between structure and seabed over the full base area, so achieving uniform stress transfer between the base and the subgrade
- to avoid the risk of soil displacement due to sea water pumping between underbase compartments under dynamic loading conditions
- to reduce the stress regime below the platform, thus reducing settlements due to skirt penetration under storm-loading conditions
- in certain circumstances, to enhance lateral stability.

The base area of a platform is designed to provide a safe bearing pressure on the seabed soils, therefore the grout strength need only be slightly in excess of the soil strength. In the majority of situations where the gravity loads are supported at the grout/seabed level, grouts with a high strength and consequential stiffness can give rise to undesirable excessive local stress concentrations under conditions of differential settlement or variation in seabed properties within the base area.

In geotechnical terms the underbase grout strength needs to be in the range from very stiff or hard clay, to weak or very weak rock, i.e. within the strength range of about 0.4 to 1.5 N/mm^2 (Figure 6.3). Similarly, the elastic modulus of the grout should be equal to or just greater than that of the subgrade.

The grout also needs to be durable to accommodate platform economic life spans of up to 40 years. Because of the stable temperature environ ment, the

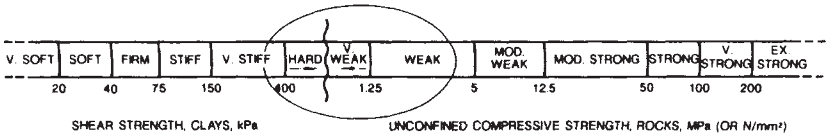

Figure 6.3 Strengths of soils and rocks.

low stress regime and the fact that the material is confined within the skirt compartments, this durability requirement is readily satisfied.

More critical are the fresh and early age considerations, which are determined by the platform installation requirements. Grouting is commenced immediately following ballasting down to the required skirt penetration. Moreover, no other operations can be performed until the grout is in place and has reached its required strength. Time is therefore of the essence.

The following grout properties are important:

1. High **fluidity** for ease of pumping. The grout-mixing plant is either established on the platform deck or more commonly on a support vessel, from where the grout is pumped over distances of several hundred metres at high flow rates. To achieve complete filling of a foundation with up to 50000 m^3 of grout, placing rates of up to 300 m^3/h are sometimes necessary.
2. Sufficient **cohesiveness** to displace the sea water in the underbase compartments without excessive dissipation or dilution of the grout. The ducting system for compartment filling and evacuation of the sea water is carefully designed, but minimum mixing at the grout-sea water interface is still desirable. Also, cohesion aids the uniform filling of compartments and reduces the possibility of mixing with or displacement of very weak soils.
3. **Thickening time** in excess of 2 h, which will allow for a temporary breakdown in pumping without blocking and catastrophic loss of injection pipework.
4. Low **heat of hydration** to avoid setting up high-temperature gradients in the base slab of the platform.
5. Sea water for mixing at as low a **solids content** as possible to enable economies of materials and transport.
6. Low **density** to enable economies of solid material handling and reduce possibility for displacement of weak sea-bed deposits.
7. High **stability** during and immediately after placing to ensure minimal segregation and bleeding. Bleed below 1 to 2% is required.

6.3 Grout formulations

Until the end of the 1970s underbase grouts used offshore varied widely in both constituents and formulation. This wide variation was not necessarily dictated by design criteria, but rather by differing schools of thought and the influence of conventional (land-based) grouting technology.

The main formulations adopted were as follows:

1. Portland cement/pulverised fuel ash/sea water in proportions 0.1/0.6/ 0.3 by weight (Littlejohn, 1981). Stability was ensured by a relatively low water/solids ratio of 0.43.
2. Portland cement/powdered chalk/bentonite/sea water in proportions 0.2/ 0.2/0.08/0.5 by weight (Kennedy *et al.,* 1978). The bentonite produced thixotropy and hence stability; the powdered chalk acted as an inert filler, thus reducing the temperature rise during hydration.
3. Blast furnace cement/sodium silicate/sea water in proportions 0.2/ 0.05/ 0.75 by weight (Boon *et al.,* 1977). On mixing with sea water the sodium silicate produces a silica gel, which holds the cement particles in suspension whilst they hydrate to form a rigid, but fairly weak structure. This grout has an appreciably lower solids content than either of the first two.
4. Ordinary Portland cement/sodium silicate/sea water in proportions varying from 0.3/0.05/0.65 to 0.15/0.07/0.78 by weight (Kennedy *et al,* 1978). This is a similar grout to the previous one, but with the simplification of using opc.

In all cases the common property is low compressive strength. With the exception of grout 1 this has been achieved by the use of ultra-high water/ cement ratios in the range of 2.0–4.0. The solids in grout 1 are a blend of cement and pulverised fuel ash (pfa). The pfa contributes to the ultimate strength of the grout but its activity is low and therefore the desired low strength and heat of hydration are achieved.

A grout of this type has been used on Ninian Central Platform, installed in 1978. This involved one of the earlier grouting operations, which suffered from the large volumes of powder product that had to be handled and blended offshore. In this case it was anticipated that up to 15 000 t of powder, producing 19 000 m^3 of grout, would be handled.

The bleed characteristics of this grout type were inadequate, and sea water and dilute grout were found to remain in the tops of some of the compartments. A secondary grouting stage, or topping-up operation, was therefore necessary, the required volume of the secondary grout being about 3.9% of the primary grout volume.

Grout 2 was successfully used in the two Condeep platform installations of 1975. The low strength resulted from the water/cement ratio of 2.4, but the overall solids content was still high.

The use of liquid sodium silicate to develop cohesion and to control bleeding resulted in stable grouts with high water/cement ratio and low solids content such as grouts 3 and 4. Grouts of type 3 were first used on the Frigg TP-1 platform installation in 1976. This utilised a blast furnace cement, which is a dry blend of ordinary Portland cement and groundgranulated blast furnace slag. Grouts of type 4 use only opc, and have been used for the Condeep installations since 1976. The successful application of these sodium silicate/sea

Table 6.1 North Sea gravity platform underbase grouting history

Operator	Location	Design	Water depth (m)	Concrete volume (m^3)	Grout volume (m^3)	Year of installation
Phillips	Ekofisk	Doris	70	80 000	0	1973
Mobil	Beryl A	Condeep	118	52 000	†	1975
Shell	Brent B	Condeep	140	64 000	†	1975
Elf	Frigg CDP1	Doris	104	60 000	0	1975
Shell	Brent D	Condeep	140	68 000	†	1976
Elf	Frigg TP1	Sea Tank	104	49 000	†	1976
Elf	Frigg MP2	Doris	94	60 000	0	1976
Shell	Dunlin A	Andoc	153	90 000	†	1977
Elf	Frigg TCP2	Condeep	104	50 000	14 000	1977
Mobil	Statfjord A	Condeep	145	87 000	†	1977
Shell	Cormorant A	Sea Tank	149	120 000	†	1977
Chevron	Ninian Central	Doris	136	140 000	5000	1978
Shell	Brent C	Sea Tank	141	105 000	†	1978
Phillips	Maureen	Technomare		Steel	3000	1980
Mobil	Statfjord	Condeep	145	140 000	†	1981
Deutsch Texaco	Svedneck Sea	Doris	20		1200*	1983
Mobil	Statfjord	Condeep	145	130 000	†	1984
Statoil	Gullfaks A	Condeep	135	125 000	25 000	1986
Statoil	Gullfaks B	Condeep	141	100 000	10 000	1987
Norsk Hydro	Oseberg A	Condeep	109	120 000	36 000	1988
Statoil	Gullfaks C	Condeep	206	240 000	35 000	1989
Phillips	Ekofisk PB	Doris	75	105 000	0	1989
Ham. Br.	N. Ravenspurn	Arup	45	10 000	0	1989
Statoil	Sleipner A	Condeep	82	75 000	20 000	1992
Shell	Draugen	Condeep	250	83 000	30 000	1993

*Total for two platforms.
†Record not obtained but for Condeep platforms in range 10–25 × 10^3 m^3.

water based grouts has led to their virtual exclusive use for gravity structure underbase void filling (Table 6.1).

The principal value of the type 4 grouts is the high product volume to powder ingredient ratio, and the consequent ease of material transportation and handling, and production. For example, 15 000 t of powder will produce around 60 000 m^3 of grout.

6.4 Properties of silicate/cement grouts

6.4.1 General

As already outlined, these grouts achieve the required properties by using water/cement ratios in the range 2.6–4.0 with bleeding controlled by sodium silicate. The sea water used for mixing reacts with the sodium silicate to produce an instantaneous precipitate of calcium and magnesium silicates in the form of a thixotropic milky white gel. When cement is added to this, the cement particles are held in suspension while they hydrate to form the hydrate structure which provides the longer term strength and stability.

The nature and properties of these grouts are described in some detail in chapter 4. For underbase grouting of offshore platforms, water/cement ratios in the range 2.5–3.0 are most favoured, with stability provided by sodium silicate contents of 3–8% by weight of sea water. A sodium silicate with an SiO_2:Na_2O weight ratio of 3.3 is preferred.

6.4.2 Fresh properties and behaviour

Concentric cylinder viscometer tests have shown that the fresh grout can be considered to be a Bingham fluid, and mixes with the above proportions have a yield stress of 6–8 Pa and a plastic viscosity of 0.008–0.01 Pa.s (Figure 4.1). The grouts are sufficiently fluid to be readily pumped up to several hundred metres.

Thickening times for such grouts are difficult to assess; however, field experience indicates that the thickening time is in excess of 2 h. This property normally provides adequate time for line flushing and plant cleanout in the event of blockage.

The cohesive nature of the grouts has been demonstrated in large-scale trials of a mock-up of a platform underbase segment 8m×1m×1m (Kennedy *et al.*, 1978) and in other more complex forms simulating underbase compartments. In these trials grout was injected through a hose weighted to remain on the base of the container, thus displacing the sea water in a similar manner to a tremie concreting operation (Figure 6.4). The grout remained in a cohesive body with negligible dilution at the interface with the sea water.

In full-scale compartment-filling trials, a range of seabed soil conditions was simulated, which showed the following behaviour:

1. Grout advanced on broad front from the injection pipe producing a thin flowing sheet over the whole surface within the compartment, effectively building in thin laminations, as shown in Figure 6.4.
2. Grout turbulence was confined to the immediate vicinity of the injection point where no dilution resulted.

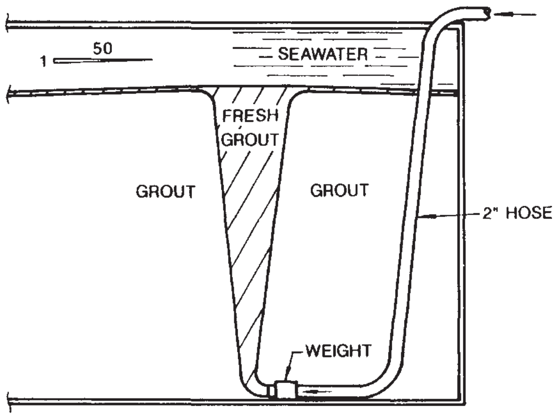

Figure 6.4 'Bottom flow' filling system for grouted compartments.

3. Flow of grout away from the injection point was laminar and no disturbance or mixing with surface soil was indicated. Simulated seabed soils studied included flowable drilling mud through very soft clay to loose sands. It was shown that soil profiles remained unaltered following grout placement.

The stability of the grout after placing has been assessed by observing the segregation and bleeding of a grout column in 50 mm diameter vertical glass tubes, 1.5 m high. Bleed water separation after placing, as measured in a 1.5 m high, 50 mm diameter glass tube, is normally complete after 2 or 3 h. At a water/cement ratio of 2.6 an acceptably low bleed of 2% of the grout height can be obtained with a sodium silicate dose of 4% by weight of sea water (Figure 4.2). The stability of the low bleed mixes is also apparent from the uniform homogeneous nature of the grout throughout the whole column height.

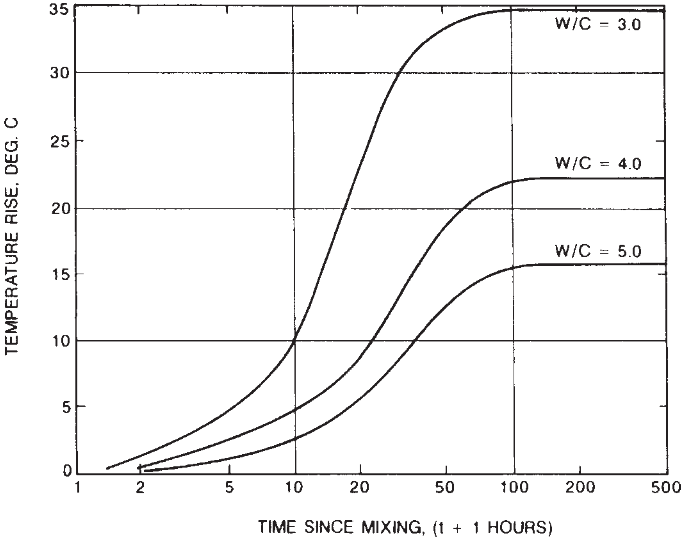

Figure 6.5 Temperature of grouts during hydration under adiabatic conditions (Kennedy *et al.,* 1978).

6.4.3 Early age behaviour

In the first few hours, or the first day, the adiabatic temperature rise produced by the cement hydration exotherm depends on the water/cement ratio (Figure 6.5). When assessed in terms of the cement content, the maximum temperature rises were between 8 and 11°C per 100 kg/m^3 of cement, which are less than the typical value of 13°C for concrete (Bamforth, 1980). The difference is probably due to the different specific heats of the two materials rather than a difference in the amount of heat liberated by the cement. The resulting thermal gradients in the base structure of the concrete platforms have been assessed as acceptable.

After a period of around 18 h a grout with a water/cement ratio of 2.5–3 will develop the consistency of a firm to stiff clay with a density around 13 kN/m^3.

Typically 80% of the 28-day crushing strength develops after 7 days, and 90% after 14 days.

6.4.4 Hardened and long-term properties

Strength data such as those shown in Figures 4.3 and 4.6 enable a grout to be designed with the required properties relative to the seabed for most offshore locations. Most grouts have a water/cement ratio in the range 2.5–3.0, giving 28-day strengths at 8°C (a typical sea-bed temperature) of 0.9–0.6 N/mm^2, and long-term strengths of 1.0–0.9 N/mm^2.

As with most cementitious systems, the stiffness and strength are directly related; the modulus values are about 100 times those of the compressive when both are measured in units of N/mm^2.

The coefficient of permeability of a grout with a water/cement ratio of 2.15, measured using the conventional API test method (API, 1974), has been found to be 3×10^{-9} m/s. This is intermediate between typical values for silt or compacted fine sand (10^{-7} m/s) and concrete (10^{-10} m/s and lower).

An important restriction on the use of these grouts is their need to be contained or confined. As described in chapter 4, unconfined samples of grout stored in water have shown signs of distress after about one year. However, confined samples, duplicating the situations in the underbase compartments, have shown no distress. Also, the continual state of saturation of the grout avoids any problems that might occur from the high drying shrinkage.

The use of these sodium silicate-based grouts for offshore platform underbase and void-filling application has proved to be successful, although no direct evidence of long-term properties of the grout for the, approximate, 30-year life of the structure is available. However, on the basis of existing data (Domone, 1990) and current knowledge and experience, confidence exists that the grouts will perform satisfactorily since:

1. Apart from the first few hours of the grout's life the stability and strength are provided by the hydrated cement matrix, which has proven durability.
2. No long-term shrinkage can occur since the grouts are in a permanently saturated state. Any shrinkage due to cement hydration or silica gel synerisis will occur during the first few hours of the grout's life, while it is still plastic and able to accommodate any movements without cracking.
3. The grout is in enclosed compartments and therefore there is no flow of water either through or past the grout to cause leaching of any of the cementing constituents. The confinement also prevents expansion and the low swelling pressures from becoming a design consideration. These swelling pressures are probably dissipated by creep and relaxation processes.

6.5 Grouting systems

6.5.1 General

In recent offshore grouting operations, the grout mixing and control is carried out on a ship moored to platform. The grout is then pumped to the platform and distributed to the underbase compartments through steel pipework, which is either cast into or connected to the structure of the platform. The main elements of a typical Condeep system are illustrated in Figure 6.6; this would be capable of placing approximately 50 000 m^3 of grout in the underbase

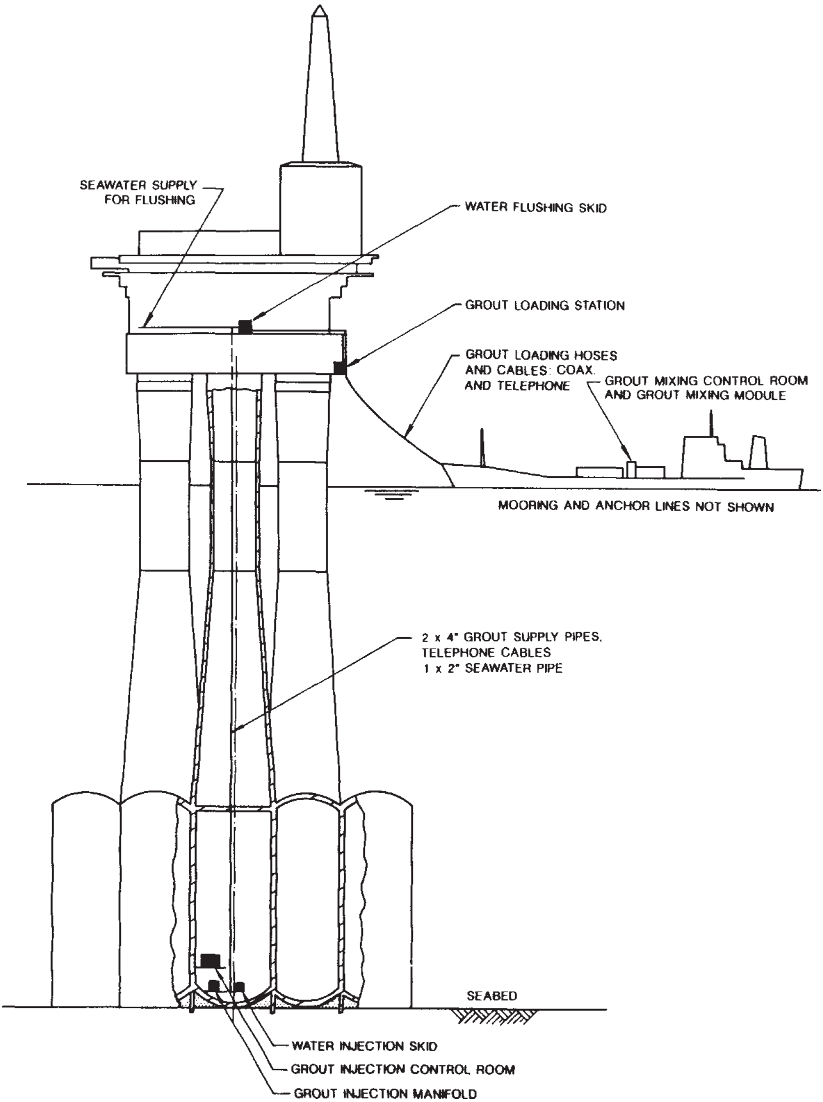

Figure 6.6 The general arrangement of plant and equipment for the underbase grouting of a Condeep gravity platform.

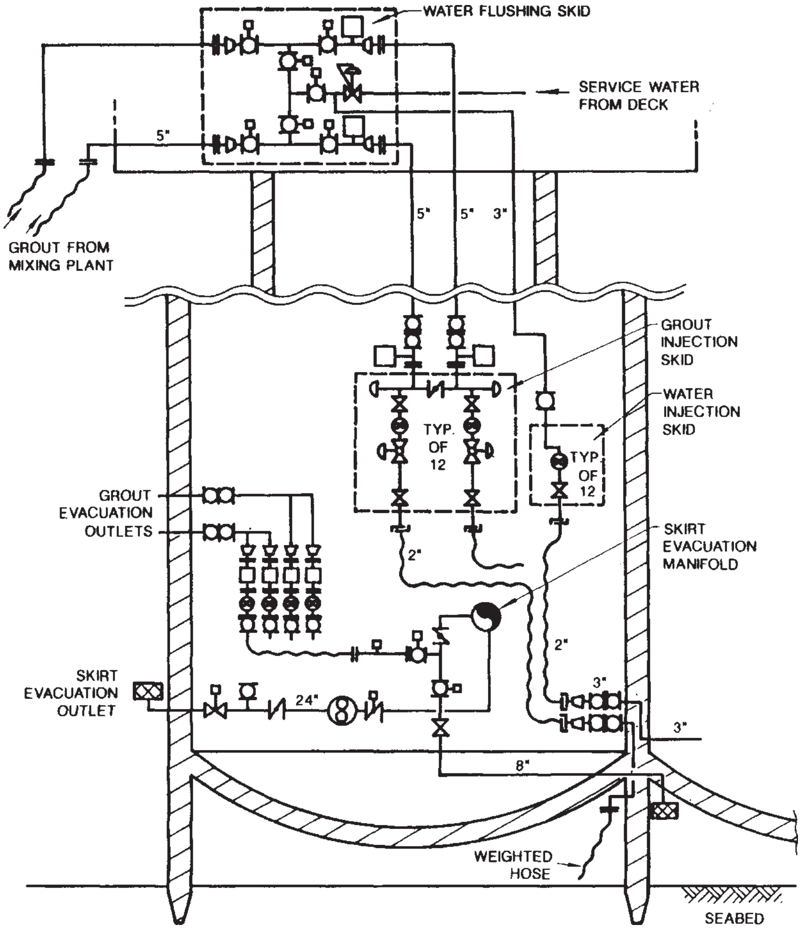

Figure 6.7 The grout distribution network and control system within the platform.

compartments of the platform. The distribution network on the platform is shown in more detail in Figure 6.7. The main features of the system are described in the following sections.

6.5.2 Grouting ship

Materials transport and storage and grout production are now most commonly carried out on a single vessel that has been appropriately modified and equipped. The vessel is moored to the side of the platform and held in position by two anchor lines and two mooring lines. (In some operations in the mid

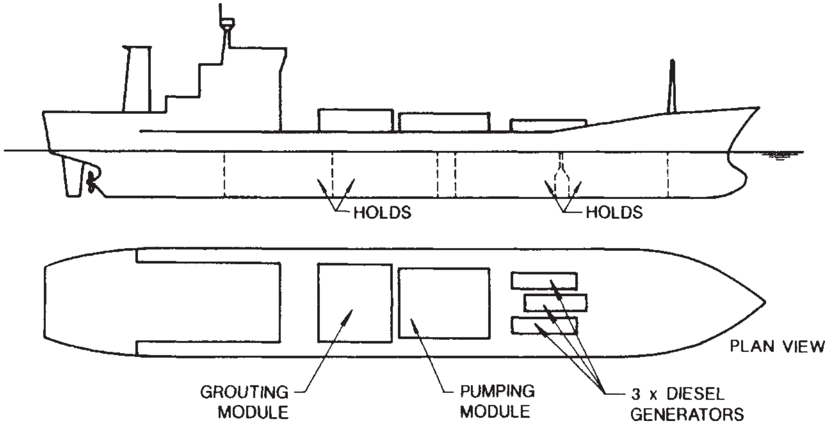

Figure 6.8 The grouting vessel M/S *Conberria*.

1970s both bulk storage and mixing plant were contained in a grouting module mounted on the platform deck.)

Figure 6.8 shows the M/S *Conberria,* the vessel currently used by Norwegian Contractors for offshore underbase grouting operations.

The vessel's main details are:

Length between PP	97.15 m
L.O.A.	106.82 m
Breadth (moulded)	15.80 m
Depth (moulded)	8.70 m
Max. draught	6.74 m
Displacement	8128 t
Dead weight	5882 t
Gross tonnage	352.58 t
Cement capacity	3770 t (bulk density 12 kN/m3)
	5047 t if no sodium silicate on board
Sodium silicate	1236 t (density 13.7 kN/m3)
Fresh water	120 t
Fuel	408 t (density 8.9 kN/m^3)

6.5.3 Grout mixing system

The grout mixing module, located on the grout ship, is a fully automatic continuous mixing plant designed to produce grout with consistent density, and has an output capacity of 75–400 m^3/h.

Buffer tanks for sea water, sodium silicate and cement are located within the vessel above the module. The sea water is pumped direct from the sea into the buffer tank. Sodium silicate is pumped from tanks and cement is fed from holds in the ship to their respective buffer tanks.

A continuous, rather than batch-mixing, method is used. The process is two-stage:

1. A liquid/liquid stage, achieved 'in-line' by metering together of sea water and a proportioned volume of sodium silicate, and mixing these by means of a static helical mixer within the pipework.
2. A powder/liquid stage, achieved by a recirculating mixer which maintains ingredient proportion by monitoring output grout density and adjusting cement input rate. Recirculation is provided by incorporating a pump of several times higher capacity than the maximum design output rate so that individual cement particles pass through the mixing system a number of times prior to discharge. By using a 'hold-up' volume within this mixing cycle, good homogenisation is achieved.

6.5.4 Grout transfer system

(a) Grout loading hoses. Two 5 in. hoses are routed from the bow of the grout ship and up to the grout loading station.

(b) Grout loading station. A grout loading station is located on the platform cellar deck to one side of the platform. The hoses from the grout ship together with a coaxial cable for control of instrumentation and telephone communication cables are terminated there. Steel pipes are routed from the loading station to the water-flushing skid. Hoses from the loading station are pre-installed for lowering to the vessel using platform cranes.

(c) Water-flushing skid. The water-flushing skid is a unit which terminates the lines coming from the grout-loading station and the sea water service system. The sea water is required for flushing grout out of that part of the system running from the grout injection skid in the bottom of the shaft to the overboard dump line on the vessel.

(d) Grout/sea water supply lines. Two 5 in. grout supply lines are routed inside the shaft from the water-flushing skid at cellar deck level to the base of the shaft. Sea water supply to the grouting equipment in the bottom of the shaft is via a 3 in. pipe. The sea water is used for flushing the grout injection skid, feeding the water injection skid and general cleaning.

6.5.5 Grout injection system

The flow of grout to each compartment is carefully monitored and controlled through instrumentation and control valves within the distribution manifold, which is usually situated at the base of one of the main shafts forming the platform leg. Each injection line is fitted with its own flow meter.

The grout flows into each underbase compartment through a flexible hose weighed to lie on the sea-bed. As proved in the full-scale mock-up trials (Figure 6.4), this helps to ensure that the grout remains as a cohesive mass, displacing sea water by gravitational and viscous differentials.

(a) Grout injection skid. The grout injection skid comprises 2 in. lines equipped with flowmeters and remote-controlled valves for controlling the flow of grout to the skirt compartments. The manifold at one end is connected to the grout supply lines and at the other to the embedded grouting lines running to the compartments. The skid is located on the grouting manifold platform at the base of the shaft.

(b) Water injection skid. The water injection skid is located in the shaft. The skid comprises lines equipped with flowmeters for injection of water to the skirt compartments not being injected with grout.

6.5.6 Grout evacuation system

The grout evacuation system consists of two 8 in. pipes running from each skirt compartment, terminating in a skirt evacuation manifold located in the shaft, and a grout evacuation manifold connected to two 8 in. seawater outlets. The skirt evacuation manifold is connected to the grout evacuation manifold by 5 in. hoses.

The grout evacuation manifold consists of 5 in. lines, and each line is equipped with a flowmeter and a density meter for monitoring the grout evacuation.

The 24 in. skirt evacuation outlet provides an alternative outlet for grout. This line is used in an intermediate phase while shifting from one compartment to another, or if more than two compartments are grouted simultaneously. No monitoring of the flow and/or density is carried out in this outlet.

6.5.7 The grouting operation

Immediately after the platform has been ballasted down to the required skirt penetration, the underbase grouting operation starts by flow/pressure testing of the compartments.

When all the skirt compartments have been tested, the grout-mixing module mixes a controlled density grout in a mixing tub. The mixture is pumped via hoses to the grout-loading station located on the cellar deck, to the water-flushing skid and down the shaft.

In the bottom of the shaft the grout is fed through the grout injection skid which controls the rate of flow in the different injection lines. The flow rate is low at the start of filling, but after about 15 min this is increased in small steps, while ensuring that the grout pressure remains within the limits of skirt differential pressure.

The evacuated sea water passes through the grout evacuation system where the flow and density are monitored.

The grouting of a particular compartment is considered complete when the approximate calculated volume has been reached and the evacuated grout has a density of 13 kN/m^3 (or its density is the same as that of the injected grout). The grouting is then normally continued for another 15 min to ensure full and complete filling of the compartment.

6.5.8 Quality control

Assessment is carried out on the freshly mixed grout prior to injection, and on the grout evacuated from filled compartments. Grout density and bleed are measured on site and cubes are prepared for compressive strength testing.

A typical grout quality monitoring programme would include the following:

1. The density of the grout is monitored continuously with nucleonic density gauges and at least once every 30 min by mud balance weighing of grout samples.
2. The strength of the grout is checked once per shift when regular pumping is in progress. Each sample consists of six 50 mm cubes for tests at 7, 28 and 90 days (two at each age).
3. The bleeding of the grout in a 35 mm diameter glass is checked at 1–2 h intervals and every time cube tests are made.
4. The temperature of the components and of the grout is recorded when cubes are made.
5. The specific gravity of the sodium silicate is recorded once per day.

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7 Grouting with fabric formwork

H.LEUENBERGER

7.1 Introduction

Fabric used for the containment of goods in various forms is as old as fabric itself. Initially the construction industry used simple bags as containers for dry sand/cement mixes for underwater construction. The development of pumpable concretes and grouts greatly influenced the development of fabric formwork from simple bags to formwork of intricate shapes and dimensions. Also, the textile industry started developing various types of suitable fabric for applications in construction, and the development of woven double sheet fabric with integrated spacer straps greatly increased the field of application. For example, in the 1960s the protection of the slopes of channels, river and shorelines with these fabrics injected with grout provided a very economical method of erosion control.

The properties and characteristics of the grout were further improved by mixing with high-speed colloidal mixers, and the chemical industry provided various admixtures to increase the stability of the grout against dispersion when directly injected into water. The range of applications within the construction industry for fabric formwork is not almost unlimited, especially for underwater projects. However, 'dry' applications do suffer from some limitations, such as:

- lack of economy as a permanent shuttering
- poor durability of the fabric skin when exposed to ultraviolet rays
- the aesthetics of its appearance.

7.2 Fabric formwork and grout

7.2.1 The relationship between the grout and the formwork

The design of fabric formwork requires a good knowledge of the properties, characteristics and behaviour of the grout. The fabric acts as a very flexible shuttering and the grout must have a high fluidity to flow unaided throughout the formwork from one or more injection points.

The fabric shuttering and the grout, which provides the final structural element, are therefore closely related and dependent on each other. Obtaining the required final shape and structural properties depends entirely on the characteristics and behaviour of each of these two very unstable components, especially at the start-up phase of grouting. These characteristics and properties are the governing factors in the design of the fabric formwork and the grout mix.

7.2.2 Definition of fabric formwork

Fabric formwork is defined as a flexible shuttering tailored from woven fabrics, usually made from synthetic fibres. Generally the fabric used is highly permeable so that air or water is readily displaced from within the form. The resultant shape that the hardened mass takes is the shape of the enclosure, which can be made in many different configurations.

The major factors to be considered when using construction fabrics are:

1. The choice of fabric, which depends on the required permeability, the viscosity of the fill material, the internal construction details, the required stiffness of the form before filling, the hydrostatic pressure acting on the outer skin and the internal restraints on the grout level.
2. The anticipated pressure from forcing the filling material into the formwork.
3. The size, shape and methods of placing and handling of the flexible formwork, and, for underwater applications, the effects of buoyancy and currents.
4. The sequence of injection, if multiple injection points or stage grouting are involved.
5. The position of bleed points or overfill prevention.
6. The provision of overfill compartments to compensate for the settlement of grout resulting from excessive bleed.

7.2.3 Classification of fabric formwork

Different designs and specific applications of the formwork demand the selection of appropriate grout mixes. The formwork can be classified as follows.

(a) Simple container shaped forms. These include simple bag, cushion and sausage shapes, and any forms with only an outer skin and no internal construction. Applications include simple supports, infill work on rip-rap, underpinning of walls and high-weight blocks with no specific demands on configuration. The fill material can vary from a sand/water mix to high-quality grouts or pumpable concrete. The required properties of the filling material are primarily controlled by structural considerations.

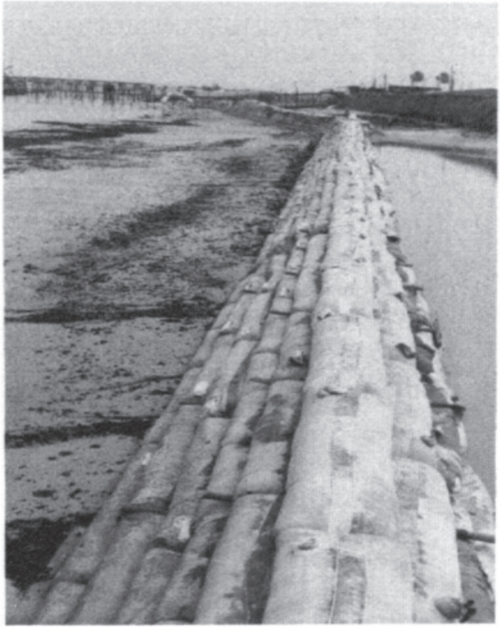

Figure 7.1 Cushion-shaped grout-filled bags used for a tidal zone dam.

Figure 7.1 shows a dam constructed in the tidal zone using cushionshaped bags. The bags in the inner layer are filled with sand and water only. Special filter strips on top of the bags let water filter out and retain the sand. Bags in the outer layer are injected with a sand/cement grout.

Figure 7.2 shows the reconstruction of an old weir using containershaped fabric formwork injected with a sand/cement grout. One of the main reasons for the use of fabric formwork was the inaccessibility of the site to construction equipment and the continually flowing water in the river. The grout was pumped for a distance of 750 m through a 42 mm diameter steel pipe.

(b) Preshaped formwork. This type of formwork is designed to assume a predetermined shape. The outer skin is forced into the required shape by suitable internal construction. This can influence the flow of the filling material within the formwork very considerably, therefore the mix design of the fill material is important to ensure that the final configuration and usefulness of the structure is achieved.

This type of formwork generally proves to be uneconomical for 'dry' applications at accessible locations, but for underwater applications it is a very cost-effective and diver-friendly solution.

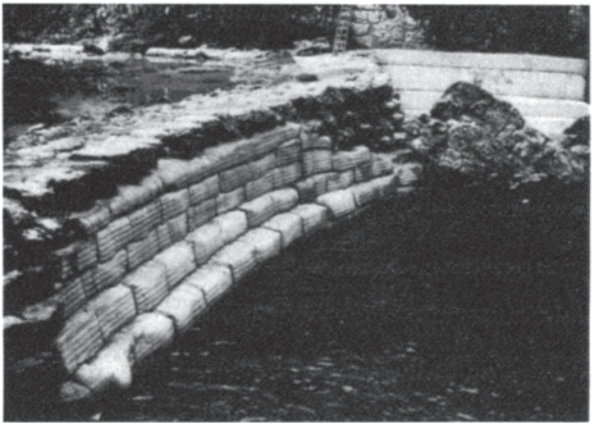

Figure 7.2 An old weir reconstructed with shaped fabric formwork injected with a sand/cement grout.

Applications are very extensive, particularly in the offshore sector, and include:

- supports for pipelines and structures
- weight stabilisation to pipelines and structures
- protection of structures.

The demands on the properties of the fill material are:

- very high fluidity
- low resistance to flow within the formwork
- stability against segregation
- hardened strength as required by the application.

Figure 7.3 shows various subsea applications of fabric formwork for the oil industry and Figure 7.4 shows the construction of a pyramid-shaped pipeline support. The fabric formwork was inject with a neat water/cement grout.

(c)Specially woven fabrics. Double sheet fabrics with integrated spacer threads or with interlaced sheets are produced by the textile industry using special weaving methods. This formwork is ideally suited to producing slabs of a mattress-like texture; the principle of a woven double-sheeted fabric with integrated spacer threads is shown in Figure 7.5. Applications include slope protection of river banks and shorelines, hydraulic engineering, and uses in combination with preshaped formwork.

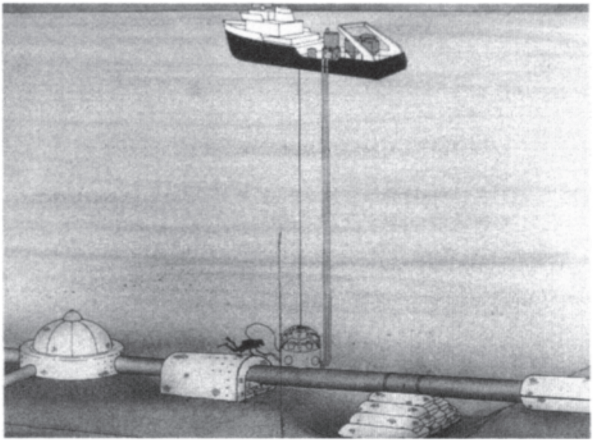

Figure 7.3 Applications of preshaped fabric formwork in subsea applications for the oil industry.

This fabric is mainly suitable for applications with sand/cement grouts or microconcrete. The demands on the properties of the grout are similar to those given above for preshaped formwork grouting.

(d) Limitations of function. In most cases the formwork must be considered as a lost shuttering with no practical function as a structural element, unless the

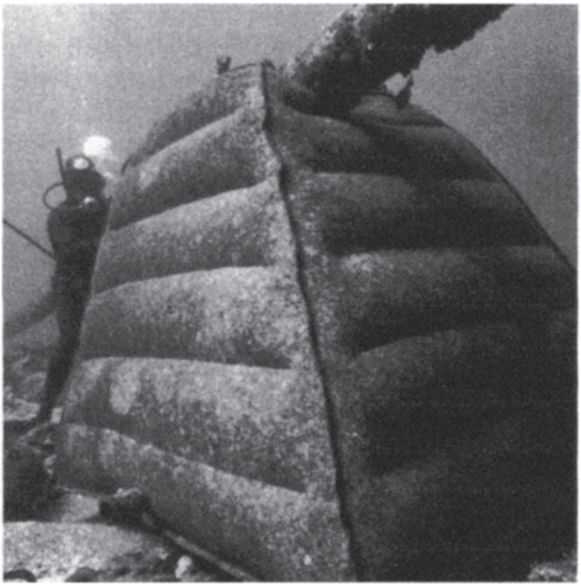

Figure 7.4 The construction of a pyramid-shaped pipeline support.

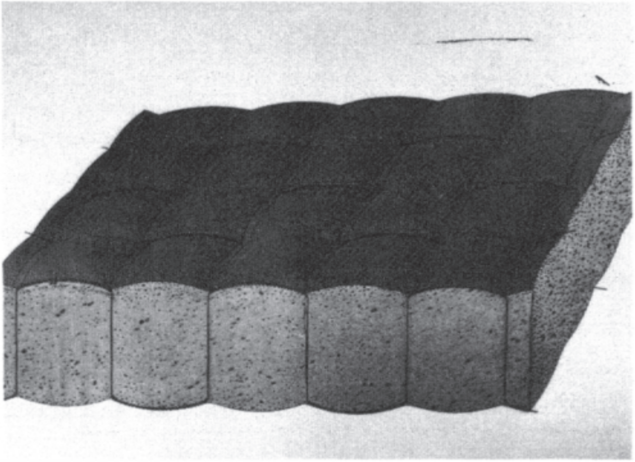

Figure 7.5 Woven double sheet fabric with integrated spacer threads.

design and choice of material provides the required integration of the formwork into the final structure.

Limitations are:

1. The fabric skin cannot be considered as a permanent part of the structure owing to its limited tensile strength and life expectancy.
2. Although great improvements have been made to stabilise the synthetic fibre against chemical attack and ultraviolet rays, the molecular stability of the synthetic fibre is short-lived when compared to the construction industry's life expectancy of structures.
3. Fabric produced from special fibres such as Kevlar is generally not economically viable for use in fabric formwork.

7.3 Grout functions

It is not possible in this short chapter to present all the variations in grout designs for the mixes used in the wide range of applications of fabric formwork. The priorities for the characteristics and properties have to be evaluated so that the mix designs meet the requirements, and the most important criteria are:

1. The location of the structure, e.g. on land, in the dry on land, partly dry, partly in water, under water but onshore, under water offshore.
2. The function of the structure and structural properties required, e.g. compressive strength, tensile strength, density.

3. The properties to suit the formwork, e.g. fluidity, flow resistance, density, bleed, shrinkage.
4. The equipment to be used for mixing, pumping and handling the grout.
5. The logistics, e.g. land bound or offshore execution, any restrictions on choice of materials, transport, storage and handling of components.
6. The methods of grout transfer, e.g. distance of pumping, size of transfer line, rate of injection and contingencies during interruptions.

It is in the nature of fabric formwork that its application is in most cases a substitute for conventional methods because it presents considerable advantages. However, in some cases it may be the only way to achieve the required result in an economical manner.

While the formwork can be handled with very little trouble, the choice of the correct grout mix may prove more difficult. For execution on land or on shorelines, variations of grout properties and characteristics are mainly limited to variations in:

- the water/cement ratio
- the use of premixed cement/fly ash blends
- the use of admixtures.

7.4 Grouting procedure

It is important that the mixing equipment, the pumping equipment, the transfer lines, and the inlet and distribution systems to the formwork are optimised if a good-quality end product is to be achieved.

7.4.1 Grouting in the dry

It should be remembered that a grout mix behaves quite differently in the dry than under water. It is essential that the grouting procedure or the mix design is chosen accordingly. The main criteria are:

1. The size and configuration of the formwork which governs the injection rate and subsequent time for execution.
2. The location and position of slopes which demand special precautions, such as the hydrostatic pressure on the outer skin, the stability of the formwork against sliding and stage grouting.
3. The flow resistance within the formwork owing to its internal construction, which demands various injection points with simultaneous injection.
4. The air temperature and humidity surrounding the formwork, which demands special precautions such as shading or water spraying.

5. The contingency measures in case of interruptions. Precautions have to be taken to ensure that interrupted injection periods can be overcome without impairing the quality of the product, by providing additional inlets or second injection lines.
6. The need for adequate curing. Any grouted structures in the dry should be cured by water spraying. Grouted mattresses on slopes or horizontal should be sprayed even during the injection phase if subjected to direct sun rays.

7.4.2 Grouting under water

The behaviour of the grout during injection is different to that injected in the dry. This results from the fact that normally more fluid grouts with considerably higher cement contents are used, and wetted surfaces have reduced friction.

The main criteria for offshore grouting are:

1. The rate of injection. This must be in proportion to the size of the formwork to be grouted. The procedure must allow for variation in injection rate and subsequent stop-and-start situations. The speed of injection must also take into account the permeability of the fabric formwork, the sizes of the compartment to be grouted and the arrangement of bleed points or overfill preventions.
2. The transfer line. The grout hose has to be carefully selected. As a rule, the internal diameter should be as small as possible to provide a sufficient head loss to balance surge under hydrostatic pressure. The optimum diameter is that for which the transfer friction equals the hydrostatic surge.
3. The inlet point. For underwater injection the grout entry point must always be at the lowest point. If this is not accessible or feasible then an internal distribution system must be installed.

Grout must always be injected into grout. Only at the start-up phase should grout be in direct contact with water. With this system the partly disintegrated grout from the start-up phase will be carried to the top and expelled through the bleed points. Allowing unprotected grout to run or drop through water must be avoided.

The establishment of a procedure must also take into consideration the ability of divers to control the operation. Often greatly reduced visibility, currents and the behaviour of the formwork in a buoyant state and the heavy grout line give handling problems which should be carefully considered when selecting the grouting procedure.

Procedures for underwater grouting in shallow waters with the mixing plant on land and with short transfer lines can be a combination of dry and offshore practice.

7.5 Grout formulation and properties

7.5.1 Cement-based grouts

Cement grouts should be sufficiently fluid to allow efficient pumping and injection. The grout must be sufficiently stable to resist disintegration and erosion after injection. Where the grout is used to fill fabric formwork the characteristics such as fluidity and resistance to flow may be the more dominant parameters in the design. Cement grouts are basically formed from ordinary Portland (type I) cement and water. Other solid materials such as sand, fly ash or clay are added for economy or to obtain special grout characteristics. A broad range of chemical admixtures designated according to their action such as anti-bleed, fluidifiers, accelerators, retarder and expansion agents may also be incorporated. The principal variable affecting the properties of cement grouts is the water/cement ratio. The amount of water determines the fluidity, the rate of bleeding and the ultimate strength of the grout. A typical relationship between these and the water/cement ratio for a neat type I cement grout is shown in Figure 7.6.

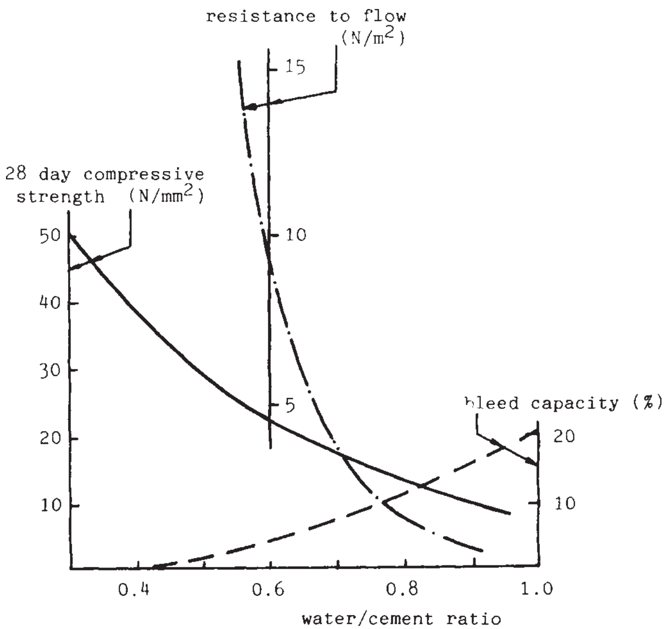

Figure 7.6 The relationship of grout properties to water/cement ratio for an ordinary Portland cement grout (Littlejohn, 1978).

7.5.2 Constituent materials

(a) Water. Water which is suitable for drinking is generally considered suitable for cement grout formulation. Sea water contains about 3.5% of dissolved salts. About 78% of the salt is sodium chloride and about 15% is chloride and sulphate of magnesium. Chlorides and sulphates are the important factors in relation to the chemistry of the cement reaction. All chlorides accelerate the setting of cement and can improve the early strength. Sulphates can retard the setting and development of strength. If structural steel is not involved in the grouted structure, sea water can be used with no adverse effects on the grout quality.

*(b) Cement.** The main type of cements used for grouting are:

- Ordinary Portland cement (opc). This is the cheapest and most commonly used.
- Sulphate-resisting Portland cement. This cement is similar to opc, but is less prone to attack by sulphate due to its reduced tricalcium aluminate (C_3A) content.

(c) Fillers. Fillers, or mineral admixtures, are often used to reduce the overall cost of the grout without affecting significantly the physical properties. Certain fillers give an advantage such as reduced bleeding or heat of hydration. In most cases, especially offshore, the economical penalties of using additional components do not justify their use.

(d) Sand. Sand added to neat water/cement suspensions forms an economical grout. As with concrete, sand is selected with regard to durability, shrinkage and density of the structure. Evenly graded sands with particle sizes in the range of 4 mm down to 75 μm are preferable. For long pumping distances the sand/cement ratio should not exceed 1 to 2.5.

7.5.3 Typical mix designs

In general, there is a very wide range of mixes for injecting fabric formwork. The range varies from sand/water to sand/bentonite/water or more commonly to cement-based grouts. For structural applications in most cases only cement-based grouts are suitable, and mixes vary considerably between onshore and offshore applications.

(a) Onshore mixes. For applications in the dry, generally cement/sand/ water mixes are used, with the proportion of components being chosen for the characteristics required. Typical mixes are shown in Table 7.1.

*See note on standards on page vii, and descriptions of cement compositions in chapter 1.

Table 7.1 Typical grout mixes for onshore applications (proportions by weight)

Cement	Sand	Water	
1	1	0.5	Suitable for long transport lines, and for formwork with internal construction
1	1.25	0.5	
1	1.5	0.5	
1	2	0.5	
1	2.25	0.5	Only suitable for medium and short transfer lines, and with formwork with no internal construction
1	2.5	0.5	
1	2.75	0.5	
1	3	0.5	

Note: The water/cement ratio can be altered to improve structural quality.

(b) Offshore mixes. For offshore applications, cement/water mixes are mostly used, since supplying fillers such as sand outweigh the cost of extra cement. Mixes depend on the type of structure:

- *Pipeline supports.* These do not generally require a high-grade grout specification as the imposed loads are normally low in relation to the overall size. Typical mixes are 100 parts ordinary Portland cement to 42–45 parts sea water. However, there is a potential danger of thermal cracking due to heat of hydration effects with supports of any substantial size, and in this case a grout incorporating pulverised fuel

Table 7.2 Typical batching proportions for grout mixes

Mix No.	Water/cement	Quantities per m^3 of grout			Grout volume per tonne cement (litre)	Grout density
		Cement (tonnes)	Sand (m^3)	Water (litre)		
			Sand/cement/water mixes			
1	0.5	0.833	0.518	416	1200	2.08
2	0.5	0.770	0.597	385	1300	2.11
3	0.5	0.715	0.670	357	1400	2.14
4	0.5	0.666	0.727	333	1500	2.17
5	0.5	0.625	0.781	312	1600	2.19
6	0.5	0.588	0.829	294	1700	2.20
7	0.5	0.555	0.867	278	1800	2.22
8	0.5	0.526	0.902	263	1900	2.23
9	0.5	0.500	0.937	250	2000	2.25
			Cement/water mixes			
1	0.6	1.086		651	0.920	1.73
2	0.5	1.219		610	0.820	1.83
3	0.4	1.388		555	0.720	1.94
4	0.35	1.480		520	0.675	2.00

ash at a ratio to the opc of up to 10:1 may be preferred (Littlejohn and Hughes, 1988).

- *Pipeline protection or weight coating.* This requires controlled density of the cured grout. Typical mixes are 100 parts of ordinary Portland cement to 34–36 parts of sea water and 2 parts superplasticizer. The resulting density is approximately 2000 kg/m^3.

(c) General batching proportions. Typical batching proportions for grouts with and without sand additions are given in Table 7.2.

7.6 Choice of equipment

7.6.1 Mixing equipment

The construction industry has a wide range of grout-mixing equipment suitable for production of grouts with various properties and characteristics. For the injection of fabric formwork it is in most cases not necessary to have a special type of mixing equipment as long as the required grout quality can be achieved either by the mixing process or with the help of chemical admixtures.

As water is involved in the majority of applications of fabric formwork, it is essential that a stable grout can be produced which does not disintegrate if injected into water. While this can be achieved to a certain extent with chemical admixtures, it is preferable to use mixing equipment producing a colloidal grout by mechanical means.

Colloidal mixers are based on the colloidal mill principle of high-speed shearing of the cement in the water to remove any air attached to the particles of cement and to ensure thorough wetting. Colloidal mixers are undoubtedly the most efficient means of mixing hydrating cement. They can produce pumpable neat cement mixes at water/cement ratios down to 0.36 without the use of admixtures. Where admixtures are required for various purposes they can be intimately incorporated in the mix by the high-speed mixing action. Cement or cement/sand or cement/pfa mixes or mixes with any pre-blended cement can also be handled.

Paddle mixers are mixing tanks with rotating paddles or various paddles and the grout is thrown against baffles attached to the side of the tank. These mixers are quite effective for mixing cement slurries down to a water/cement ratio of 0.5. Sand/cement mixes cannot be mixed satisfactorily with these mixers.

Jet mixers are mostly used for a continuous mixing process. The colloidal effect is not very prominent. As this type of equipment is mostly for high output, its use is not very suitable for grouts being injected into fabric formwork.

7.6.2 Pumping equipment

The choice of the equipment for grout transfer and injection into the formwork is important. The criteria are:

- the grout mix
- the length of the transfer line
- the size of the formwork to be injected
- the quantity to be transferred within a certain time limit
- the possibility of controlling the pressure at the entry to the formwork.

Rotary screw pumps are limited in pressure (maximum 20 bar) but are capable of dealing with large volumes up to 20 m^3/h. Sand/cement mixes can be pumped, but the wear on stator and rotor is very extensive especially if a coarse sand is used. They are convenient to use with water/ cement mixes. An advantage is that they give a continuous pulse-free flow.

Piston pumps can generate very high pressure and are well suited to transferring sand/cement grouts long distances. The wear characteristics with such mixes are better than screw pumps. A slight disadvantage for certain types of injection is the pulsating flow of grout.

Any grout pump should have some means of controlling output for the injection of fabric formwork. Hydraulic, air-operated or electric motors with speed variation pumps can give virtually infinite variations in output. For pumping grout down a gradient or vertically to the seabed the pumps should be fitted with stop valves to prevent uncontrolled flow due to surge through the pump.

7.6.3 Transfer lines

The choice of the transfer line is a very important factor for a trouble-free operation.

(a) Grouting on land. On land, sand/cement mixes are used in most cases; the flow characteristics are different to cement slurries and the choice of line depends on the mix design, or vice versa. The main criteria to be considered are:

- mix design
- fluidity and flow resistance of the grout
- the required flow rate
- the length of line and the expected friction
- the expected transfer pressure
- contingency action in case of blockages.

The main recommendations are:

1. For short transfer lines (up to 100 m) the above criteria can, in most cases, be accommodated within a wide range of transfer lines.
2. If the grout has to be pumped over very long distances the criteria have to be carefully considered to achieve a trouble-free operation.
3. Steel pipes give the best result as the pipes do not extend under pressure, as can be the case with rubber hoses or plastic pipes. Steel pipes should not be fitted directly to piston pumps, and a short length of high-pressure flexible hose has to be fitted between the pump outlet and the steel pipe in order to compensate for the pulsating knocks of the pump action. For contingency measures, in case of blockages, it is advisable to fix quick-release couplings at intervals for unblocking the line.

(b) Grouting offshore. As the mixes most commonly used are based on neat cement/water grouts with high fluidity and low flow resistance, the choice of the transfer line is governed less by the mix and more by the operational criteria. These include:

- the length of the line
- the water depth
- surge action producing a vacuum
- the stresses on the unsupported line and couplings during deployment, operation and recovery
- hydrostatic pressure build-up if discharge is blocked
- the weight and buoyancy of the submerged line
- flexibility of handling on the seabed by divers.

The main recommendations are:

1. The diameter should preferably be very small to eliminate any possible surge. In principle, the friction in the line should compensate as much as possible for the hydrostatic surge action of the grout. Grout hoses with 32–38 mm internal diameter are generally used.
2. The hose must have a steel spiral reinforcement to prevent collapse in case of a vacuum build-up owing to surge.
3. Stresses in the line during deployment, operation and recovery should be counteracted by incorporating a steel cable along the line attached to the couplings. The assembly should be tape-wrapped to form an umbilical. Synthetic fibre grout lines are not an ideal solution as the lines themselves have considerable elongations if put under load. The supporting cable counteracts the elongation of the hose and the subsequent reduction of diameter increases the danger of the coupling connections being pulled apart.
4. Hydrostatic pressures in the line are built up if the outlet is blocked. The hose must therefore be able to withstand this pressure, which is about 0.09

bar per metre of vertical drop. The use of small-diameter lines helps to overcome these problems as the diameter and pressure capacity are inversely related.

5. The weight of the hose and its buoyancy when filled with water must be known for deployment. Grout hoses should have a negative buoyancy to prevent them floating uncontrolled in the water during deployment.

7.7 Quality control

There is a wide range of quality control procedures for grout, and the extent of the field control is a decision to be made by the engineer. For onshore work, in most cases it is not difficult and not too costly to carry out a testing programme. For offshore, elaborate testing can be a very expensive item compared to the total cost of the work. Laboratory installations on vessels manned with qualified personnel are usually only implemented for grouting operations with a very close tolerance on grout properties and a high dependence on grout quality.

For most offshore grouting, with the exception of structural grouting on platforms, the grout quality can be controlled by checking the water/ cement ratio with a mud balance.

Cubes can be taken for control of the structural strength. While the density control with the mud balance is essential to maintain a good quality of grout before injection, many factors, from the injection procedure at seabed level, can influence the final quality of the grout when placed. For this reason it is more important to ensure close supervision of the injection than to carry out elaborate testing on the vessel during operation.

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8 Grouts for structural repairs

A.McLEISH

8.1 Introduction

When considering the use of cementitious grout for structural repair purposes attention must be paid to the inherent properties of the grout, and in particular the problems that may be encountered.

Compared with concrete, grout typically has a number of disadvantages when used as a structural material:

- it is expensive when used in large quantities
- it suffers from a high drying shrinkage
- it has a high heat of hydration and coefficient of thermal expansion, often resulting in temperature-induced cracking when used in thick sections
- it has a low modulus of elasticity and a high coefficient of creep

These disadvantages are, in all cases, related to the lack of large aggregate which restrains the movement of the cementitious phase of the material.

The use of grout in structures is therefore generally restricted to small thickness sections such as under base plates and in prestressing ducts. However, although the structural use of cementitious or sand-cement grout for thick sections is limited, two areas are worthy of further discussion:

- the use of grout with preplaced aggregate to form grouted aggregate or preplaced aggregate concrete
- the use of grout filled with small aggregate (typically up to 10 mm) and generally known as flowable concrete.

These uses are discussed in this chapter.

8.2 Preplaced aggregate concrete

8.2.1 General

Preplaced aggregate concrete (sometimes referred to as grouted aggregate concrete) is produced by filling formwork with large aggregate and then pumping cementitious grout into the form to fill the voids between the aggregate particles. To reduce the risk of thermal or shrinkage cracking the cementitious grout usually contains graded fine aggregate.

Preplaced aggregate concrete has been used in a wide variety of applications, including concrete repairs, large volume foundations, high-density shielding to nuclear reactors and producing exposed aggregate finishes. Its use is particularly appropriate in applications where the placing of conventional concrete is awkward, such as under water, in tidal zones, in areas of limited access or in heavily congested structural elements. The references at the end of this chapter discuss the use of preplaced aggregate concrete in a variety of situations and conditions.

The technical and economic advantages of using preplaced aggregate concrete are numerous:

1. It allows the use of large-sized uncrushed aggregate (typically greater than 40 mm except where it is limited by reinforcement or section geometry).
2. The grout mixing process does not involve large aggregate and therefore smaller mixing facilities are necessary.
3. Mechanical vibration is not required.
4. Low void ratios, achieved by careful grading and compaction of the aggregate into formwork, mean that less cement is required compared to conventional concrete.
5. Lower drying shrinkage due to point-to-point contact of the coarse aggregate is especially important in large volume pours. Typically the shrinkage is only 50–70% of that of a conventional concrete.
6. Point-to-point contact of the large aggregate also results in a higher modulus of elasticity and reduced creep. Both these factors are important in repairs where new concrete is required to carry load at an early age.
7. The concreting operation is carried out in two stages, with the cementitious material being required only in the last operation.
8. There is no risk of segregation of the large aggregate.

8.2.2 Materials

(a) Coarse aggregate. The aggregate should be durable, chemically stable and not subject to excessive breakage during handling and placing. To allow easy passage of the grout during injection, bulky rounded or angular aggregate is preferable to flat or elongated.

The maximum size is dictated by handling considerations but should not be more than a quarter of the smallest dimension of the element to be formed.

The selection of the coarse aggregate and, in particular, its minimum size is also dependent on the type of grout to be used. If a sand/cement grout is used with too small a coarse aggregate then the sand can block the passage of grout thus preventing complete filling of voids. If a cementrich grout is used with a large coarse aggregate then the large volume of grout can result in large temperature rises with the risk of thermal and shrinkage cracking.

If sand/cement grout is to be used, the minimum size of aggregate should not be less than 40 mm. Where fly ash/cement grout is used to control temperature rises, the minimum size of aggregate may be reduced to 15 mm. The aggregate should be graded to give a minimum void content, which is usually between 35 and 40% after compaction.

It is very important that aggregates are washed thoroughly before use and that placed aggregate is not allowed to remain ungrouted in the form for a long period. Any impurities remaining in the aggregate will reduce the bond between the stones and the grout and hence reduce the strength of the concrete.

The aggregate should be in a saturated surface dry condition before the grout is pumped in. This will prevent rapid water loss from the grout and difficulty in achieving flow through the section.

(b) Grout. Grouts need to be sufficiently fluid to be efficiently pumped and injected to fill all the voids. Any type of cement may be used though care should be taken if rapid-hardening Portland cement is used in large masses due to the increased heat of hydration.

If a low-speed paddle mixer is used, the maximum size of sand should not be greater than 1.5 mm. If a high-shear colloidal mill is used, then the maximum size of sand may be as large as 5 mm. In either case the maximum size should not be greater than one-eighth or one-tenth the minimum size of the coarse aggregate for a natural rounded or crushed angular sand respectively.

Crushed or natural sands may be used, though well-rounded natural sands are preferable since they require less water to achieve an acceptable grout fluidity. The sand should be hard, dense, durable and free from all impurities which could affect the set of the cement or replace the flow of the grout. The sand is normally medium grade to BS 882 (BSI, 1983). The maximum size of the sand should be in the range 0.05–0.15 times that of the coarse aggregate.

Pulverised fuel ash (pfa) or ground-granulated blast furnace slag (ggbs) can be used as a partial replacement for the cement to reduce the heat of hydration and bleeding, and to improve the workability of the grout.

The addition of microsilica (usually accompanied by a superplasticiser) can result in a high-strength, low-permeability concrete which is resistant to many forms of chemical attack.

(c) Admixtures. In general, for reasons of simplicity and economy, the use of admixtures is avoided unless particular advantages are required.

Plasticizers and superplasticizers will allow a lower water content to be used for a given workability. This results in a finished concrete of higher strength and density, and reduced permeability. Alternatively, for the same water content, the workability is increased facilitating pumping of the grout into the preplaced aggregate.

Air entrainers improve the cohesiveness and workability of the concrete. This minimises bleed where fine aggregate grading is poor and may assist in pumping.

For large pours, or where the rate of grout pumping is low, retarders can be used to delay setting times, reduce the risk of cold joints and allow more time for placement. Retarders are often combined with plasticisers or superplasticisers.

Polymer modifier admixtures can greatly enhance the properties (particularly bond and tensile strength) of the final concrete.

8.2.3 Method of placement

Coarse aggregates should be washed immediately before placing in the forms to remove all dust and impurities as well as to ensure that the surface is moist during grouting. Buckets are commonly used to transport aggregate to the forms though flexible elephant trunks are often used to limit the height of free fall and the degree of segregation. Coarse aggregate has been successfully placed to a depth of 150m through water in the strengthening of steel piles of a North Sea platform.

Grout injection pipes are positioned before or during the placing of the coarse aggregate. These pipes may be plastic or steel of a size ranging from 20 to 40 mm diameter typically, spaced at 1.5–4 m centres on plan. The grout pipes are normally withdrawn as grouting proceeds, or, as an alternative, additional pipes could be positioned to terminate at 1–2 m centres vertically. As grouting proceeds the pipes should remain embedded in the rising grout by at least 150 mm to ensure that air is not entrapped and that a homogeneous mass is produced.

An alternative approach, particularly suitable for repairs, is to build injection ports into the bottom of the formwork. These should be provided with valves to allow the inlets to be closed off after completion of the grouting process.

During grouting it is essential to know the location of the grout surface at all times with a reasonable degree of accuracy. This is done by using ‘sounding’ wells which usually consist of 50 mm diameter slotted tubes through which a ‘sounding’ or measurement line can be lowered. Alternatively, where placement is carried out in the dry, electronically calibrated detector wires can be placed in the aggregate and the rising grout surface monitored.

Grouting should commence from the lowest point so that air (and water if placement is under water) is displaced upwards. There are two basic patterns of injection: allowing the grout surface to rise as a horizontal plane, or establishing a sloping grout surface to advance from one side of the form to the other. The former pattern is most commonly used though the latter is suitable for constructing thick slabs.

If form vibration is required it should be applied at the level of the grout surface. Excessive vibration will cause segregation of the sand from the grout.

If a trowelled or floated finish is required, the grout should be brought up to flood the aggregate surface then any excess surface grout should be removed. A thin layer of 5–10 mm aggregate can then be worked into the surface. Once the surface has hardened sufficiently, conventional practice can be used to produce the desired finish. Care should be taken with the grout injection rate when flooding the surface. The normal rate of pumping used for the majority of the placement may cause soiling and uplift of the surface aggregate. This should be avoided by reducing the pumping rate.

Top surfaces may also be formed by using a ventilated form. This will allow excess air to escape yet restrain any upward movement caused by the pressure of the injected grout. A typical form may consist of a permeable sheet placed on top of the aggregate, backed with expanded metal lath and topped with sheeting boards spaced from 10 to 25 mm apart.

8.2.4 Problems

When constructing formwork for preplaced aggregate concrete the workmanship needs to be of a higher quality than for conventional concrete to prevent grout leakage. This is particularly important in underwater applications or in otherwise restricted locations. Adequate venting must be provided to allow the escape of air and water as grouting proceeds.

Around closely spaced embedded items coarse aggregate may need to be placed by hand. High-pressure air jets may be used to assist in the moving of aggregate.

Care should be taken in the grout mix design to limit the amount of bleed by careful control of the water/cement ratio and the sand grading. If high bleed rates are allowed to occur then the resultant concrete will be of low strength, with increased shrinkage and poor durability. Therefore, trial mixes should always be tested to ensure that the concrete has the desired properties and that the grout can be successfully pumped.

Figure 8.1 shows a placement trial to check on the flow of grout around pipes. This trial used a sand/cement grout with 40 mm minimum coarse aggregate size. A previous trial using a 20 mm aggregate was not successful as the sand prevented the flow of the grout through the aggregate.

In underwater applications contaminants or suspended solids present in the water may coat the placed aggregate and adversely affect the bonding of the

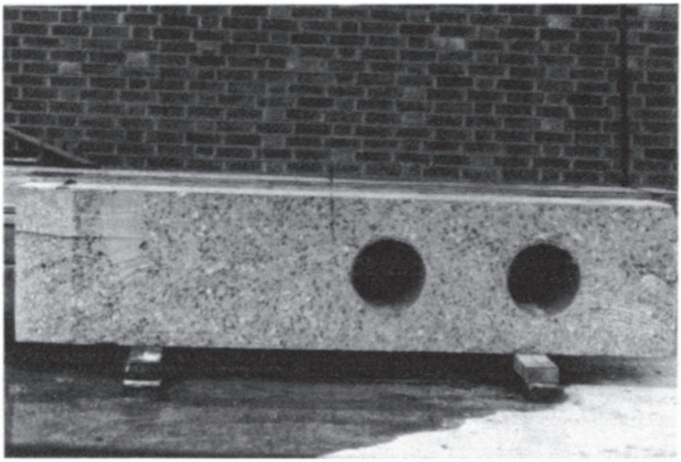

Figure 8.1 Preplaced aggregate concrete trial to check the flow of grout around pipework.

grout. Where such conditions are present, water samples should be taken to determine the possible influence on the quality of the concrete. For moderate contamination, aggregate should be grouted within one or two days of placing. Where contaminants are present such that they cannot be eliminated or controlled, preplaced aggregate concrete should not be used. Even in apparently clean water aggregate should be grouted within a week to 10 days.

8.3 Flowable concrete

8.3.1 General

Although not strictly grout, flowable concrete (consisting of grout and small aggregate) can be designed to have some of the same properties. In particular it can be pumped, it will flow into position around reinforcement without vibration and can be self-levelling. Although it often tends to have a higher cement content than normal concrete, the inclusion of small-size aggregate overcomes to a large extent many of the problems associated with grout listed in the introduction to this chapter. Nevertheless, the mix design has to be carefully considered and control tests carried out to ensure that the flowable concrete has the required properties.

One of the principal uses of flowable concrete is in repairs, and it is this application that is discussed below.

Concrete for use in repair often demands properties not generally considered in detail in concrete for new works. In particular, the flowable concrete for many applications must:

- have small maximum aggregate size to allow flow in the restricted spaces between parent concrete, reinforcement and formwork;
- be self-levelling and self-compacting as access for vibrators is often limited;
- have a high rate of strength gain, particularly where many small localised repairs have to be undertaken;
- be dense and have a low permeability as often repairs result from a combination of low cover and chloride/carbonation attack of the original construction which must be overcome;
- not suffer from any bleed that would destroy the bond of the repair concrete to the parent concrete, thus reducing the structural effectiveness and presenting a leakage path into the reinforcement.

The specifications and test requirements for a specific repair contract recently undertaken are now outlined and used to illustrate the use of flowable concrete.

8.3.2 Requirements for repair concrete

A problem had arisen on a motorway flyover because of deteriorated joints which allowed the penetration of de-icing salts from the road deck above onto the supporting reinforced concrete beams. Repair to these beams was necessary because of extensive cracking and spalling, particularly on the soffit and top deck, due to chloride-induced corrosion.

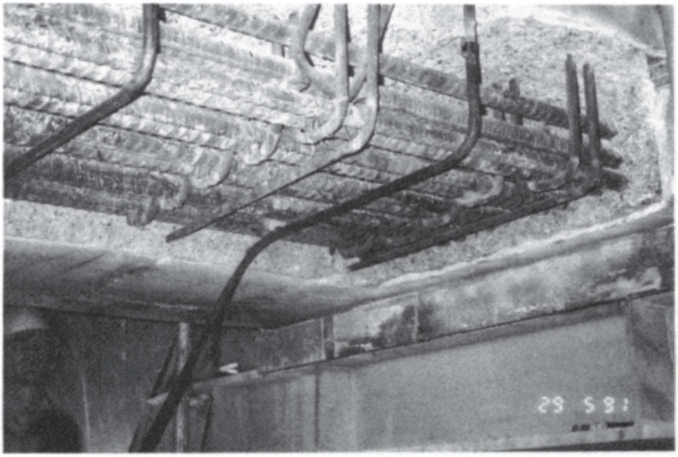

Figure 8.2 Congested reinforcement in the soffit of a beam to be repaired.

The loading conditions applied to the beams required that, to avoid extensive and complex temporary support, the repairs had to be carried out piecemeal such that only small areas could be repaired at any one time. This necessitated concrete with a high rate of strength gain to minimise delays between successive repairs. In many other repair situations, however, this rapid strength gain would not be essential, thus allowing greater flexibility in the design of the flowable concrete mix.

Access difficulties precluded the use of poured concrete and resulted in the choice of a flowable, self-levelling, self-compacting concrete. Congested reinforcement (Figure 8.2) and low cover in some areas dictated a maximum aggregate size of 8 mm. The concrete used for the repair was thus essentially a cementitious grout with the addition of 8 mm nominal size aggregate.

The key elements of the specification for the flowable repair concrete can be summarised as follows:

Minimum cement content	450 kg/m^3
Maximum aggregate size	8 mm
Maximum water/cement ratio	0.40
Minimum compressive strength	30 N/mm^2 at 3 days at 20°C
	30 N/mm^2 at 10 days at 5°C
Maximum compressive strength	60 N/mm^2 at 7 days at 20°C
Flow along test trough	750 mm in 30 s at 20°C (30 min after mixing)
No shrinkage at 7 days	

To ensure good quality control it was decided to use proprietary factory dry batched repair concrete. Details of the various repair concrete mixes used remain confidential to the manufacturers. However, in general the mix constituents were as follows:

opc between 480 and 550 kg/m^3
pfa typically 20% of total cement content or
ggbs typically 30% of total cement content
Microsilica 5% or less of total cement content
Aggregate (8 mm maximum size) between 1220 and 1320 kg/m^3
Water/cement ratio between 0.35 and 0.40
Plasticizer and other undefined admixtures

8.3.3 Testing

Early in the development of the specification it was realised that the required properties of the flowable concrete were highly sensitive to small changes in the constituents and, in any case, were difficult to achieve. A three-stage testing regime was therefore adopted to ensure compliance:

1. Initial compliance tests These consisted of extensive laboratory and field trials of the concrete to ensure full compliance with the specification.
2. Production control tests Each batch of concrete manufactured was tested for flow and compressive strength.
3. Site tests These were routine site tests on flow and strength to compare with previous initial compliance and production control test results.

Additionally two types of placement trials were required.

- The first was a simulated soffit test in which the flow of the concrete around reinforcement was checked through a perspex sheet representing the broken back soffit of the beam being repaired.
- The second was a full-sized repair trial in which a specially cast section of beam was repaired and then cored to check on air voids and debonding at the repair/parent concrete interface. Inspection after 56 days was carried out to check for shrinkage cracking.

Parameters such as shrinkage, bleed, segregation and 'placeability' were therefore checked by realistic trial repairs rather than placing total reliance on laboratory tests.

(a) Initial compliance tests. Laboratory tests were undertaken on a range of proprietary mixes and designed mixes. The tests, which were generally carried out at 5ºC, 12ºC and 20ºC to simulate the range of site conditions, are listed below. In all cases no vibration or other means of compaction was applied as the concrete was intended to self-compact.

1. *Flow trough tests.* The flow characteristics were assessed using the equipment shown in Figure 8.3. Each test consisted of six readings, three taken immediately after completion of mixing and three taken 30 min later. For compliance, none of the flow test times was to exceed the specified time.

2. *Simulated soffit tests.* The flow characteristics were also assessed by simulated soffit tests. The general arrangement of this test is shown in Figure 8.4. The layout of the reinforcement for this test was selected to be the most onerous in terms of achieving placement of the concrete that was likely to be encountered. After the concrete had set, the specimen was saw-cut into two sections which were examined to assess the amount of voidage around the reinforcement and at the repair/substrate interface, bleed at the interface, cracks and any other defects.

3. *Air content.* The air content of the fresh concrete was measured.

Figure 8.3 Assessment of flow characteristics with a flow test.

4. *Compressive strengths.* These were measured at 3, 7 and 28 days. In addition to meeting the minimum and maximum strength criteria, the difference between the minimum and maximum test results were restricted to 20% of the mean strength. The maximum strength requirement was applied as an indirect means of controlling the brittleness of the repair concrete. Previous trials had used very high-strength concrete which, during break-out to repair adjacent areas, had fractured.

5. *Chemical tests.* Chemical tests were undertaken to determine the cement, chloride and alkali contents of the concrete.

6. *Expansion and shrinkage.* During development work on the flowable concrete, short-term expansion and drying shrinkage tests were undertaken to check on the performance of the concrete admixtures. These were subsequently omitted from routine approval testing because no correlation was found between the results obtained from the laboratory testing and the actual performance in the full-scale mock-up crossbeam.

(b) Production control tests. Following the initial compliance tests various concrete formulations were accepted as being suitable for use in repairs to the

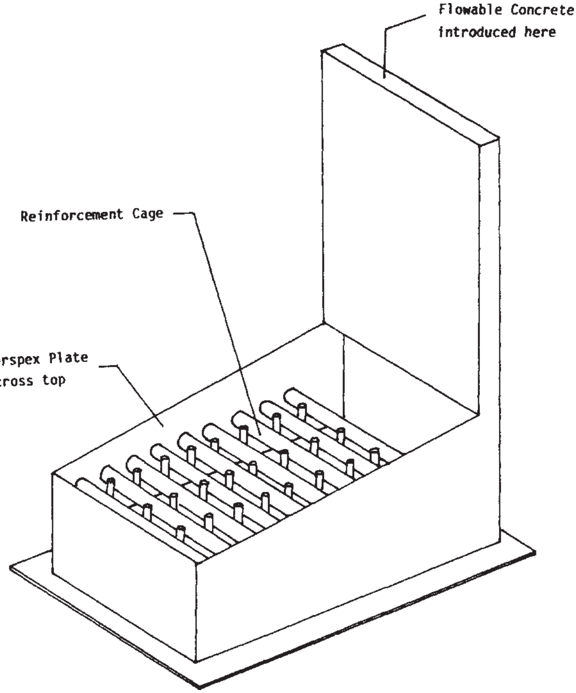

Figure 8.4 Simulated soffit repair test.

structure. During the course of the repair works production control tests were carried out on each new batch of concrete supplied by the manufacturer.

Production control tests were restricted to flow trough and compressive strength testing both carried out at 20ºC only. These tests proved to be a very valuable method of detecting poor quality or defective batches of material and providing a check on the manufacturer's quality control.

(c) Site tests. While the production control tests demonstrated the suitability of each batch of repair concrete, further site tests were carried out on site on every pour. For each pour of concrete a flow test was carried out and the compressive strength gain was monitored by testing cubes stored alongside the repair areas at ambient temperature.

8.3.4 Method of placement

The consistency of flowable concrete makes it ideal for pumping and this is generally the most efficient method of placement for all but very small

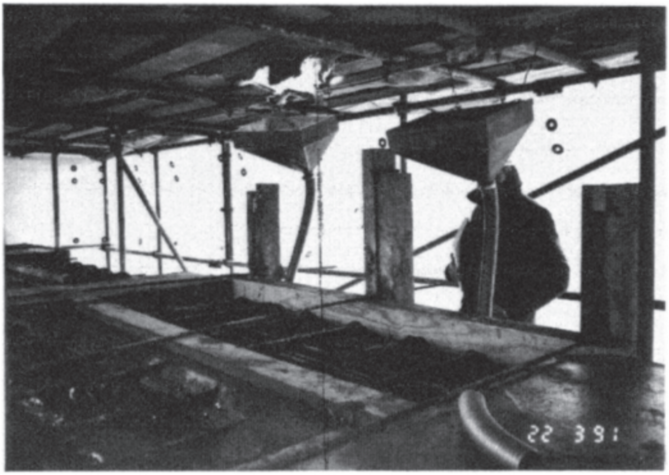

Figure 8.5 Funnel and tube system for placing small quantities of flowable concrete in a repair.

quantities. The inclusion of only small-sized aggregate allows a smaller diameter pipeline to be used. The concrete can either be pumped into the top of the formwork, or introduced through a valve in or near the soffit. This latter approach is particularly appropriate for repairs where the introduction of the concrete at the bottom of the lift minimises the risk of trapped air which could reduce bond to the substrate and provide a path for

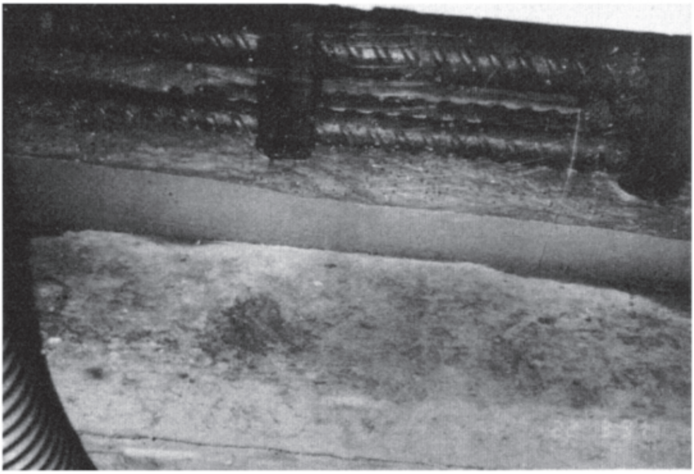

Figure 8.6 A perspex-faced shutter to enable the flow of repair material to be viewed.

water penetration. It is also the best method of placement for underwater application.

A convenient method of placement of small quantities of flowable concrete is by means of a funnel and tube (Figure 8.5). A transparent perspex face to the formwork allows the placement of the concrete to be monitored and provides a check on compaction and absence of voids (Figure 8.6)

8.3.5 Problems

One problem that can occur with flowable concrete is cracking due to drying shrinkage. For repairs the concrete generally has a high cement content and, even where a shrinkage-compensating admixture is used, is susceptible to long-term drying shrinkage. It is cast against a substrate which is often much larger in mass than the repair, which is therefore effectively totally restrained against shrinkage. Crazing and fine cracks have been recorded in several instances using some repair concretes, although this was not serious enough to cause a durability problem.

Another problem encountered is brittleness of the repair concrete. The requirement for a high early strength can lead to extremely high 28-day strength and brittleness of the concrete.

For repairs requiring small batches of concrete, quality control of strengths and flow properties can be difficult to achieve.

When used for soffit repairs, bleed of repair concrete can result in poor bond with the substrate and an increased risk of water penetration to reinforcement. Flowable concrete used for repairs must be designed, and use constituents (particularly sand) which minimise the amount of bleed that occurs.

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9 Grouting of tunnel linings

M.F.ANNETT

9.1 Introduction

During a recent survey of contractors, consultants, designers and clients (UTRC, 1991) some 20 areas relating to the construction and maintenance of tunnels and underground structures were identified as causing concern and needing further investigation and research. The first two areas were the waterproofing and grouting of tunnels. As well as being interrelated, both have historically been troublesome; they have often caused construction delays and considerable overruns on costs, with solutions eventually being found at great cost, after long delays and with substantial lost construction time.

A major cause of these problems is a lack of detailed consideration of the grouting specification at the design stage. For example, in three recent major contracts, the specification has called for pea gravel to be placed behind the tunnel lining, this then being grouted with a 1:1 neat Portland cement/water grout. Apart from the unsuitability of the grout, the pea gravel is difficult to inject behind the lining and it is virtually impossible to pump in the grout if water is present. In each case, trials or construction practice have shown the specification to be totally impractical; it has finally been rejected for alternative methods of grouting, but only after needless cost and lost time.

There is now a considerable amount of expertise and knowledge on successful grouting methods and practice, and adequate consideration and forward planning would certainly reduce, and probably eliminate, 90% of problems such as the one just described.

This chapter describes the most common types of grouting used in new tunnels and shafts, and in the refurbishment of existing structures, such as brick-lined sewers. The choice of the method depends on the results of a prior ground investigation. The grout requirements and properties, and the mixes used to achieve these, are discussed and some consideration is given to grouting plant and site practice. The information presented mainly relates to the author's experiences, and it is hoped that sufficient detail is given to ensure that the designers and engineers responsible for producing grouting specifications are made aware of the need to give adequate attention to this important subject.

9.2 Ground investigation and design

In any tunnelling operation, knowledge of the ground to be traversed is of prime importance. Of great significance is the position of the tunnel relative to the water table and, if water is expected, the effect it will have on the stability of the ground. The need for a full study of the ground with an extensive programme of site investigation, even for relatively simple tunnels, cannot be over-emphasised. This is particularly critical in regions subject to drought and, hence, wide fluctuations in the water-table level.

The properties of the ground are important factors in the consideration of:

- design of the tunnel lining
- design of any temporary support required prior to the installation of the lining
- assessment of construction method, including any temporary works
- assessment of the time and cost of the construction
- assessment of the consequences of construction on other operations.

In any one case there is no single correct answer, and the tunneller has to use his best judgement to arrive at the optimum solution. A wrong decision in any of these prime criteria can lead to the cost of the tunnel being significantly increased.

An additional key item is the consideration of the method of grouting the tunnel annulus. This will influence the rate of advance of the tunnel, which in turn is directly related to overall cost.

9.3 Planning and specification

A report by the International Tunnelling Association (ITA, 1991) has highlighted the damaging effect that water can have on tunnels during their working life. The opening statement is that 'Water is the tunneller's enemy', because:

- it causes problems during excavation
- it introduces additional expense into the tunnel lining and ground support
- it frequently causes ongoing problems during the working life of the tunnel.

Adverse experiences over many years have confirmed that not all of the degrading effects of water are obvious at the time of tunnel design. Indeed, some problems arise only after the tunnel has been in use for a significant period of time. The problems and damage caused by the effect of water on tunnels during their working life may be classified as:

(a) external effects on the surroundings of the tunnels, but not affecting the structure
(b) structural effects, influencing the structural adequacy of the tunnel and its lining
(c) functional effects, influencing the structures and fittings within the tunnel.

These comments strengthen the case that, when tunnel grouting is required, it must be fully considered during the preliminary survey and design thinking; the suitability of both the grouting design criteria and the grouting techniques must be assessed and specified. In this process, it is necessary to assume that water will be encountered to a greater or lesser degree.

Grouting of the linings of deep or shallow tunnels is generally required in soil types ranging from medium to soft rock, sands and alluvial gravels. Grouting is not normally required in hardrock tunnels, which are either finished with a shotcrete lining, a permanent in-situ concrete lining or are merely scaled and left untreated. At the other extreme, grouting of tunnels in deep clays is sometimes not required, particularly if the clay has an adequate stand-up time. However, settlement considerations can make grouting necessary.

9.4 Methods of grouting

9.4.1 Definitions

Grouting of tunnels and shafts can take place at various stages of the construction operations:

1. **Pre-grouting** is required immediately in advance of driving the tunnel or shaft. The main purpose of pre-grouting is to prevent water inflow during excavation and during the subsequent service life. In some cases, it is used to consolidate the ground or rock to minimise the amount of structural lining required, and for this reason it is also sometimes known as **consolidation grouting**.
2. In severe cases of excessive groundwater flow, a second stage of pregrouting, termed **intermediatory grouting,** is required.
3. After the tunnel or shaft has been driven, the annulus between the lining and the surrounding soil or rock is filled with grout. This is called **primary grouting,** and it is carried out as soon as possible after the lining has been placed. The grout ensures efficient and uniform load transfer between the tunnel lining and the surrounding strata, and also further reduces any water ingress into the tunnel.
4. In some cases, **secondary grouting** (also known as **post-grouting**) is

required to supplement the primary grouting. It need not necessarily immediately follow the primary grouting; for example, in repair or maintenance purposes, it may be carried out at any time in the life of the tunnel.

The combined operation of primary and secondary grouting is sometimes called **cavity grouting**.

9.4.2 Tunnel pre-grouting

An effective reduction of the water inflow into a tunnel during construction can be achieved in a number of ways, such as:

1. A fan pattern of holes can be drilled ahead of the tunnelling operation and the rock grouted prior to excavation (Figure 9.1). Care needs to be taken to ensure that the drill hole is outside the intended line of excavation; this prevents lost drill steels from affecting the tunnelling operation. The length of drill hole should not be allowed to exceed 6 m as problems can be experienced with longer holes.
2. If the surface terrain permits (i.e. access is not blocked by buildings, etc.), the drill holes can be made from the surface to pre-grout the rock ahead of the excavation (Figure 9.2). Although this does not interrupt the tunnelling, it is costly, requiring a lot of drilling, and each hole must be surveyed as it is drilled to ensure that it is in the proper location. Also, it may well strike other underground services, particularly in urban areas.
3. A pilot tunnel can be driven and the rock grouted from this heading (Figure 9.3). This is not always feasible and a major disadvantage is the

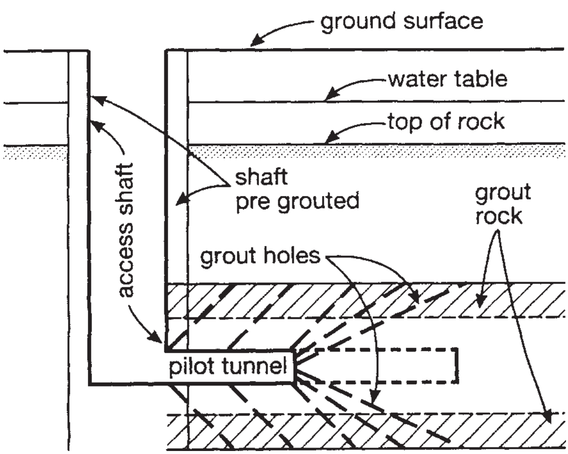

Figure 9.1 Fan pattern grouting ahead of a tunnel excavation (Driscoll, 1990).

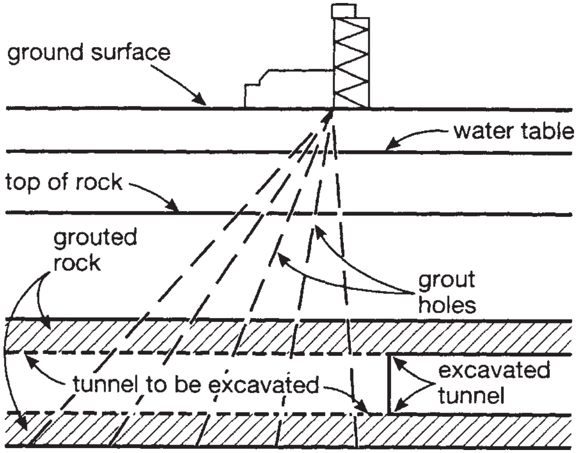

Figure 9.2 Drilling and grouting from the surface ahead of a tunnel excavation (Driscoll, 1990).

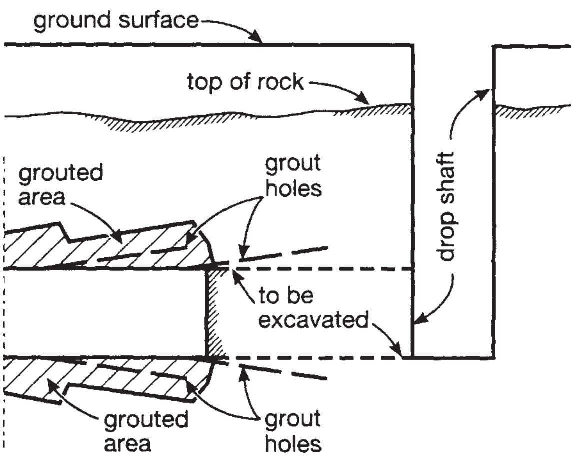

Figure 9.3 Pre-grouting of a tunnel from a pilot tunnel (Driscoll, 1990).

need to support the strata, which may cause its own problems with the main drive following behind. Also, it may cause problems with safety and construction techniques, and, in effect, becomes a post-grouting situation.

9.4.3 Shaft pre-grouting

Pre-grouting of shafts is performed by grouting from the surface prior to excavation. If necessary, this may be followed by grouting from within the shaft during excavation. The main objective is to reduce the groundwater

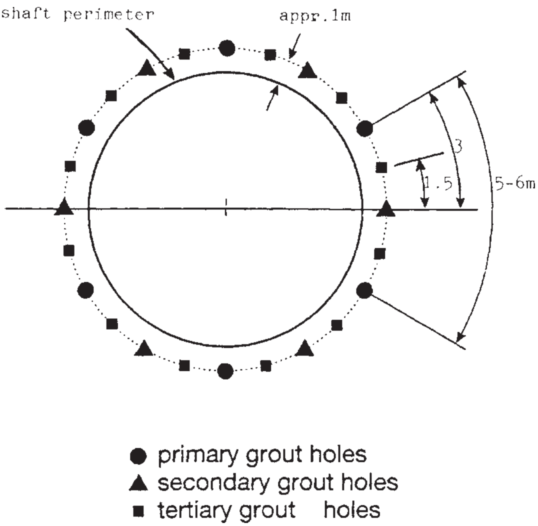

Figure 9.4 The pattern of grout holes for shaft pre-grouting (Driscoll, 1990).

inflows to levels that do not significantly interfere with the construction activities within the shaft. This can be quantified prior to excavation by establishing an inflow rate of equivalent rock permeability towards which the grout programme can be directed.

Grout holes at the surface are positioned using a split-spaced method. The first series of holes, the primary holes, are drilled in a circular pattern around the shaft periphery at a maximum predetermined spacing (Figure 9.4). Surface holes are preferably drilled by rotary methods using a tricone bit or diamond core bit. Percussion drilling methods tend to produce rock flour that partially clogs the rock discontinuations exposed in the borehole, and reduces the subsequent grout takes.

After the primary holes are grouted, the need for further grouting is determined from the results of the water-pressure tests performed during the grouting programme or by analysis of pump test data from a test well located in the centre of the shaft. If grouting is to continue, secondary grout holes are drilled midway between the primary holes. If still further holes are required, these are drilled and grouted at intermediary locations.

In grouting from the surface the downstage method of packers is used. The hole is advanced in, typically, 6 m stages in the sequence illustrated in Figure 9.5. Each stage is washed, water-pressure tested, and then grouted to refusal by means of a packer located at the top of the stage. The process is continued until the required depth is reached, and the hole is then filled with grout.

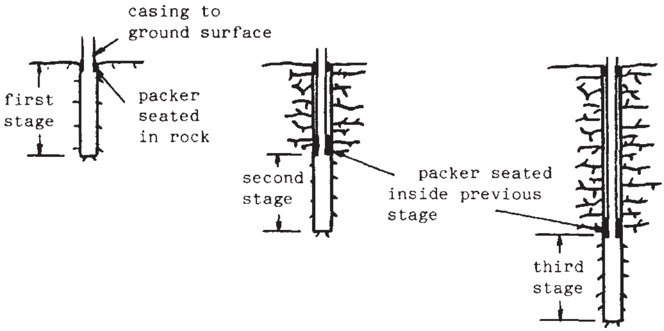

Figure 9.5 Downstage grouting with a packer (Driscoll, 1990).

The primary source of water inflow to rock excavations are near-vertical joints, which provide only a limited opportunity for interception with vertical boreholes. Additional grouting from within the shaft excavation may therefore be necessary following successful closure of grout holes drilled from the surface. If such grouting is necessary, water-producing discontinuities should be grouted in a systematic manner in advance of the excavation. If grouting is delayed until inflows become severe, the grouting and excavation levels may merely force inflows upward along fracture lines, to exit from points which are no longer easily accessible.

As with all grouting operations, the pumping pressure should be continuously monitored. Normally, when grouting to an 'open face', pump pressures of between 0.5 and 1 bar are experienced; when the void is full, these will jump quickly (within seconds) to about 10 bar. The pumping must then be instantly stopped, ideally with a preset cut-out mechanism on the grouting module.

9.4.4 Primary grouting

It is not the aim of this section to describe the tunnel-drilling operation in any detail. With the steady growth of tunnelling over the last 20 years and the development of new technologies—tunnel-boring machines, roadheaders and a variety of slurry machines—there is now an enormous selection of techniques and equipment from which to choose. Progress has been considerable since the time of some of our illustrious forebears, such as Brunel or Bartlett, who developed and patented the first bentonite machine in the UK the 1960s.

However, cost is always a critical criterion in tunnelling, and current trends indicate that segmental linings used in conjunction with tunnelboring machines are likely to be the preferred method, at least during the 1990s. To achieve stability of the lining, to prevent heave, and to cope with any water ingress or ground collapse on to the lining, the annular void behind the lining must be filled as soon as possible, preferably as the excavation proceeds.

Primary grouting design is a highly individualistic interpretation of assessing the requirements of an unknown void in varying circumstances with unknown consequences. Failure to grout during construction can result in surface settlement in days or weeks, but other effects of inadequate grouting—such as heave within the tunnel, tunnel flotation or cracking of the linings—may indicate a number of years later that problems are inherent. For example, in a tunnel which has not been grouted for several hundred metres, it may suddenly become apparent that the invert level has altered by some 100–200 mm, rings are squatting or the tunnel segments are out of alignment.

9.4.5 Post- or secondary grouting

Post-grouting of shafts and tunnels is a difficult, time-consuming and costly operation, but is commonly required. There are two basic approaches:

- feature grouting, which is required in cases of medium and high water inflows, and
- pattern grouting, for low water inflows.

9.4.6 Selection of procedures

The grouting procedures will vary according to the job, the policy, the objective, the geology, the contractor and field personnel, and individual judgement and preference. The techniques, which can be varied, include drilling, washing, pressure testing, selection and adjustment of mixers, grouting pressures, flushing the holes and washing the pumps system during grouting, use of delays, intermittent grouting, determining the need for additional grout holes, treatment of surface mix, and maintaining up-to-date records of drilling, grouting and monitoring.

This list shows that, regardless of how well conceived and designed the grouting programme, its success depends on the field techniques used and on good judgement by field staff. Grouting techniques may not be subject to contractor quality control and therefore should be directed by the programme field staff. For this reason, an experienced engineer or geologist, supported by adequate field staff, should supervise the grouting programme.

It should now be apparent that the grouting of tunnel and shaft linings can be a complete and absolute nightmare with significant cost overruns from the original budget. The cost of grouting is a very low percentage of the total construction costs (often of the order of 0.01% of the contract value), but like the bolts which secure the wheels of a car, grout failure can result in disaster. In particular, the comments on the detrimental effects of water on tunnels, both during construction and on the long-term performance, show that grouting merits far more attention than has previously been the industry's norm.

9.5 Grouting in particular situations

9.5.1 Grouting of shield-driven tunnels

There is an optimum period for grouting the annular gap between the tunnel lining and the surrounding strata for shield-driven tunnels. The grout must be placed in time to prevent the ground from collapsing on to the lining, causing uneven loading and possible deformations, but it must avoid grouting the end of the shield and impregnating and nullifying the effectiveness of the tail-seals. The grouting sequence must therefore be carefully phased in with the tunnel progress.

With the slurry-tunnelling concept, such conflicting requirements are further complicated by the buoyancy of the lining within the void until it is grouted. Even though the added weight of the trailing sledges, ancillary tunnelling equipment, etc., behind the tunnel-boring machine (TBM) offsets this tendency to some extent, the overall tendency of the lining to float remains.

Good grouting practice is to grout from the invert upwards, irrespective of whether the tunnel is being driven in 'free' or compressed air. However, such grouting in tunnels driven using the slurry technique tends to aggravate the buoyancy effect, with the added risk of unevenly loading the lining and causing distortion as well as gaining access to the tail-seal system at the bottom of the tail skin. As a result, it has now been established that slurry-driven tunnels should generally be grouted from the ring shoulder position, with, in some cases, additional slurry being supplied from behind the TBM to stabilise the void until it has been grouted.

9.5.2 Grouting of brick-lined tunnels

Brick-lined tunnels that have been built in the last 100–150 years are now commonly suffering long-term deterioration and, therefore, require maintenance and repair. The main problems are cavitation and voiding, in extreme cases with void heights above the lining in excess of 1 metre. The effects of water percolation through the brickwork may be further aggravated by freeze/thaw damage, causing deterioration of the face bricks in the form of spalling or even complete loss of bricks. The whole structure is usually totally water saturated, and therefore the grouting of voids and cavities behind the lining is a necessary first step, before attempting any crack injection of the lining. This requires a careful survey and assessment of the entire tunnel lining, and grouting should ideally commence at one portal, working progressively to the other end of the tunnel. If the grouting is started from the middle of the tunnel, the water naturally tends to ingress through the strata from either side of the grouted section and it is very difficult to assess the effectiveness of the grouting. Inspection using radarscanning techniques can give inconsistent results, and diamond coring is still the most effective method of assessment.

Subject to the client's requirements, it is fairly basic practice to establish a pattern for the drilling of injection and vent holes and then to grout from the bottom of the tunnel upwards. Traditionally pfa/opc grouts or neat opc grouts have been used for this work, and it has been a standard practice to use a high water/cement ratio mix and pump until rejection occurs.

However, there are many tales about how people started grouting and weeks later discovered that the grout was coming out miles away! The use of admixtures is therefore becoming more and more common, particularly those based on long-chain polymers, which give excellent anti-washout properties together with volume stability and virtual elimination of bleeding and separation.

While undoubtedly being beneficial, these have limitations in traditional applications as the pumping life is usually short and can cause blockages in lines and pumps. As a result, accelerators have also been considered to reduce the setting times and lower the possibility of washout by water flow. While valid in concept, this approach was not fully developed until the recent advent of the Hydraulic Variable Control System, which deliberately extended the setting time of the selected grout material to longer than 24 hours with a retarder incorporating long-chain polymers, and then added an accelerator at the point of injection to counteract the retardation.

The grouting of brick linings should always be carried out at very low pressures, i.e. 0.5 bar, with an upper limit of 5 bar when grout uptake is completed. Pressure gauges should be inserted at appropriate points in the line to ensure that the operators can always respond immediately to observations. In regions of high water pressure, relief pipes should be provided, and the grout pump pressures maintained at between 1 and 2 bar above the water pressure.

9.5.3 Sewer renovation

Sewer renovation is normally carried out by fitting a thin lining of glass reinforced plastic (grp) or cement (grc) of slightly smaller diameter than the existing lining, and then injecting grout behind this to provide structural support. Alternatively, a resin impregnated cloth system may be used (prc), held in position by internal water pressure. The general comments on the principles of grouting brickwork linings apply equally well to this operation.

During the last ten years, the basic properties required from the grout have been strength, durability and economy, and suitably proportioned pfa/opc grouts satisfy these requirements. Opinions differ on the compressive strength requirements, but 8 N/mm^2 is considered to be the desirable minimum. Normally a 3:1 pfa/opc grout will achieve this in 28 days.

Admixtures can have varying effects on the strength development of such grouts, and this will need to be established by trials. For example, overdosing of certain retarding admixtures can lower the strengths, even after 28 days.

Some other points to consider in relation to general sewer grouting practice are:

1. While thinner grouts with high water/cement ratios are cheaper and have greater penetration, they have extended setting times, reduced strengths and may impose higher stresses during installation. Thick grouts are expensive, difficult to pump and have reduced penetration. A target water/solids ratio of 0.4–0.45 usually produces an acceptable ‘creamy’ consistency which is effective for both pumping and strength requirements.
2. The tunnel lining should be isolated into sections by mortar stop-ends and each section can then be grouted in one shift. Thus the grout should completely fill the void and an evenly distributed grout pressure should be applied while curing takes place. This is particularly important with all thin lining systems, since these are not very stiff and are normally strutted to provide temporary support during the grouting and curing stages.
3. The quantities of grout used are obviously dependent upon many factors, but particularly the annular space between the new lining and the old sewer. It is, however, interesting to note that, even in the case of the prc lining, which is in nominal full contact with the brickwork, records show that it is possible to inject 10 tonnes of grout over a 120 m length of tunnel. This is therefore the amount required to reinstate the original brickwork lining and fill the voids behind it. Assessment of grouts used in other schemes indicate similar quantities after allowing for the annular volume.

It is therefore unlikely that any one approach will ideally satisfy all the requirements for a given scheme, and material and the method of installation chosen in each case will be a compromise.

9.6 Grouting specifications

Grouting specifications are all too often open to a very liberal interpretation of requirements. A performance type specification, which allows the specialist grouting consultant and/or contractor to use the most effective solution for the particular application, is preferable to a prescriptive type. The following examples of useful clauses from such specifications illustrate this.

1. Cavity grouting is to be carried out in two stages, primary and secondary grouting.
   - Primary grouting will be an initial void filler and be to a pressure of not more than 1 bar above the surrounding hydrostatic pressure.
   - Secondary grouting shall be completed as soon as is practicable, but within 14 days of the ring build or 50 m from the face, whichever is the most critical. Secondary grouting shall be at a pressure not greater than 6 bar, consistent with completely filling all voids.

2. *Method statement*
   Whatever the solution proposed for filling of the annular void, the Contractor shall give a detailed description of the proposed device and method of injection and obtain the consent of the Engineer. The proposals shall include details and location of the mixing plant and grout pump(s), mix design and constituents, pumping rates and pressures, injection points, the methods of monitoring, recording and controlling the sequence and timing of grouting, the method of preventing grout leakage, and details of the experience of the personnel and supervisors.

3. *Mix*
   The grout shall be a mixture of Portland cement to BS 12 and water with a water/cement ratio in the range of 0.35–0.5 by weight as appropriate to the circumstances. The Engineer may allow plasticisers or non-shrink agents in the grout mix or the use of other additives, excepting those containing calcium chloride. For the purpose of this clause, additives shall include bentonite and pulverised fuel ash but shall exclude sand.

4. *Grout characteristics* The characteristics of the grout and the working procedure shall satisfy the following requirements:
   - In the short term, the grouting shall prevent settlement phenomena prejudicial to safety of the environment.
   - In the long-term, the grout shall be a factor for water-tightness and durability of the tunnel.

The grout shall have the following characteristics:

(a) be prepared as near as possible to its injection point
(b) be initially of suitable viscosity to fill the void created during the shield penetration
(c) set quickly to avoid settlement
(d) be formulated correctly in order not to block the tail seal
(e) provide a long-term homogeneous, stable and low-permeability ring around the tunnel lining
(f) preferably, be placed from the invert to the shoulder.

Accordingly the Contractor shall particularly study:

- The grout composition and types of additives
- The working out conditions, viscosity and shrinkage characteristics, and injection pressure
- The setting and rheological characteristics of the grout
- The long-term durability and strength of the grout, and its compatibility with the lining segments
- Quality control procedure and tests (in laboratory and on the working sites). In particular, the volume of grout injected for each ring, compared with the theoretical volume of the annular void, shall be controlled and recorded. If the amounts injected are shown to be insufficient or the grouting imperfect, secondary grouting as a complementary treatment shall be performed as soon as possible, at the Contractor's own time and cost.

9.7 Case study: Grouting the Channel Tunnel

9.7.1 General grouting requirements

The Channel Tunnel linking the United Kingdom and France is one of the greatest civil engineering feats of our time. The tunnel was driven through the Lower Chalk Marl which was badly jointed and had a zero stand-up time in places. Some areas were very wet, having ingress of saline water of the order of 116 litre/min at pressure levels of up to 5 bar. Effective grouting of the tunnel linings was therefore important for the successful operation of the tunnel. The tunnel in fact consists of three separate tunnels—two running tunnels of 7.5 m internal diameter and a service tunnel of 4.5 m internal diameter. In the UK sector, precast concrete segments were used for the lining, and the grout was required to fill a 20 mm annulus between the lining and the chalk. Tunnelling rates of approximately 250 m/week were programmed, and the initial projected grout volume for the UK sector of the tunnel was some 55 000 m^3, with each complete ring of precast concrete lining segments requiring some 0.85 m^3 injected into the annulus. Any delays in the grouting operation would have had a dramatic impact on the completion dates of the project. The volumes of grout involved, the logistical supply requirements and the large size and speed of the operation meant that planning, control and successful execution of the grouting was vital.

There had long been a need for traditional cementitious grouts to be modified to meet the demands of such fast, mechanised tunnelling. The grouts required sufficiently early setting characteristics to take the invert load of the segment trains within 1 h of grouting, i.e. setting should occur within 15 min of the grout being pumped into place behind the tunnel linings. This was achieved by the addition of an accelerator at the injection point. Good anti-washout properties were required to cope with water ingress and the large overall volume of grout, which was often placed in relatively small batches and therefore required a long pumping life prior to injection. Also, the UK sector of the tunnel had open-jointed linings with no gaskets, and so any flow of the grout through the joints would cause major dispersion or 'fluffing up' problems, which were likely to be aggravated by the wet ground conditions.

As mentioned previously, grouts currently in use for wet conditions incorporated long-chain polymer-based admixtures with accelerators added at the mixing station. These grouts quickly thicken and become unpumpable and were therefore incompatible with long pumping lines. In the Channel Tunnel grouting, although mixing was generally undertaken as close to the point of injection as was feasible, the demands of the project required the development of an enhanced performance grout, based on an opc/pfa blend.

9.7.2 The client's specification

The main points in the specification for the grouting of the articulated precast concrete tunnel linings issued by the consultant (Mott McDonald) in September 1989 were:

1. *Properties*
   - The minimum strength, measured on 100 mm cubes, should be 1.0 N/mm^2 at 1 day, and 8 N/mm^2 at 28 days.
   - The initial set should be achieved within 45 min of injection at a temperature of 20ºC.
   - The final set should generally be achieved in a maximum of 6.5 h at a temperature of 20ºC, unless there are other conflicting requirements.
   - The grout should not bleed significantly during hydration.

2. *Testing*
   The above properties should be determined, on any of the grout mixes used, at least at weekly intervals during grouting, or on each 30 m^3 of grout, whichever was the greater.

3. *Verification of grouting*
   - Proof drilling to expose the excavated ground should be carried out in the crown of the running and service tunnels and/or elsewhere if required. All voids encountered should be fully grouted, and all drill holes filled using non-shrink cementitious material or similar.
   - Grout in the invert segments of running tunnels should be verified by proof drilling through grout holes.

4. *Grouting method*
   - The grouting should be carried out in the following stages:

     (a) grouting of the invert segments
     (b) grouting of the remaining segments up to the shoulder
     (c) grouting of the key void
     (d) grouting of the crown.

     Where appropriate, one or more of the stages could be combined.
   - The grout should be pumped into the annular space starting from the lowest grout hole in the segments and successively progressing up the segments. Grouting should be progressed uniformly on both sides of the tunnel to maintain symmetrically balanced pressure on the tunnel lining.
   - The injection of grout should be continued until the grout emerges from the highest point in the section being grouted.
   - Proof grouting should combine the injection of grout through the drilled holes at a pressure not exceeding 0.5 N/mm^2, as measured at points of injection. It should be deemed as complete if the quantity of grout injected does not exceed 300 kg per group of 5 rings. If greater quantities are required the proof grouting shall be repeated.

     (Note: this relatively low pressure was a consequence of the segmental non-bolted design of the lining.)

9.7.3 Development of the grouting mix and method

Extensive laboratory and full-scale site trials were carried out to develop and prove the grout mix and the production and placing techniques. The solution that was finally chosen dealt specifically with the need to provide a correctly grouted invert in the marine running tunnels, where ground conditions were such as to generate grout washout situations. The grouting system and materials proved to be flexible and capable of dealing with both fast and slow rates of progress.

The base grout, designated GP3, was a 3:1 pfa/opc mixture, to which an anti-washout, anti-bleed, volume stable retarding admixture, designated 802, was added. This is based on a selected stabilised sugar-reduced lignosulphonate plasticiser with a polymer-based thixotropic agent. It is a free-flowing powder and was added at the mixing tanks to the base mix at a rate of 0.5% by weight. The retardation enabled long-term mixing and circulation of the grout; thus the operatives always had grout available on demand and the age-old problems of blocked lines, pumps and valves were virtually eliminated. The grout was easily pumped, but retained cohesive properties to reduce washout when placed under water.

A liquid accelerator, designated 803, based on a modified silicate to provide rapid gelling and set of the grout once in place, was added to the GP3/802 mix by means of a metered in-line mixer in the grout injection nozzle. The dosage rates varied between 2 and 6% by volume, dependent on conditions, as discussed below.

The grout had the characteristics of a long-chain polymer grout, but as it was much less viscous it was therefore capable of being pumped at the low pressures required. It developed the early strengths required by the specification, and had adequate anti-washout and thixotropic properties, reducing fluffing to an absolute minimum. The properties are discussed in more detail in section 4.1 of chapter 4.

The mix was varied slightly for different parts of the annulus:

1. For the invert, the use of an accelerated anti-washout grout was essential and a 4% admixture dose rate provided positive displacement of water and silt during grout injection. Experience proved that the TBM rams could be removed within minutes without loss of grout from the invert.
2. From the knee to the shoulders, where excessive water ingress was evident, a 5% accelerator dose rate was used. As a general rule a 4% dose rate was adequate in static water displacement conditions; in dry conditions the accelerator could be further reduced to 2.8%.
3. For the crown, to reduce grout seepage through the joints and in the case of heavy water ingress, a predetermined grouting pattern was essential. The use of 4% accelerator dose rate reduced cavitation and water entrapment and had the advantage of requiring nominal fluffing of the joints.

The dry grout powder was held in 4.5 m^3 static bunkers mounted on the TBM sledge, which in turn were supplied by transit cars from storage silos in the lower site at Shakespeare Cliff on the UK coast. The plant finally adopted for wet conditions consisted of paddle mixers, mono pumps and metering pumps. More sophisticated weigh-batching equipment was primarily used in dry conditions. It was found that in wet saline conditions, the robust and easily operated Hydraulic Variable Control System eliminated problems previously experienced with electronic metering equipment.

The gelling properties and ability of the grout to displace water behind the lining was dramatically demonstrated both in site trials and later in the marine running tunnels. The grout was successfully placed in conditions of heavy water ingress at estimated pressures of up to 10 bar. Further details of the whole development have been published elsewhere (Annett and Stewart, 1991).

The concept of using a flexible variable control system for the metering and dispensing of a retarding admixture, with subsequent acceleration behind the lining provided at the nozzle on injection, has been successfully demonstrated with other grout mixes, including:

- pfa/opc grouts.
- mortars, comprising pfa/opc blends with sand and rounded aggregate of size 5 mm and smaller, with the incorporation of some microsilica in some cases
- bentonite/opc grouts.

In all cases an extended pumping life of at least 6–8 h has been achieved with the consequential benefits of virtually eliminating blocked lines and pumps. The ability to achieve early load capacity within 60 min of the material being injected behind the lining has been of great benefit both from the stability of tunnel linings, to the load transfer at the build area from the TBM sledges and the flat bed wagons bringing materials and segments to the erection area.

9.8 Concluding remarks

At the start of this chapter, the importance of giving adequate consideration to the grouting of tunnels and shaft linings at the design and specification stage was emphasised. It is hoped that, if nothing else, the relatively brief and general comments in the chapter have explained this requirement. However, even with the ‘best’ specification, first-time success cannot be always guaranteed. Each job is unique, and the grouting operation always has a learning curve, no matter how experienced the engineers and operatives who have the responsibility for the work.

The whole concept of grouting would appear to be much less well understood by the industry than might be expected for this essential and repetitious requirement. All those concerned must be prepared to learn from past experience, and continuing investment on research and development is essential in this recognised area of potential difficulty in the construction of underground structures.

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10 The grouting of prestressing ducts

S.A.JEFFERIS

10.1 Introduction

One of the major applications of structural grouts has always been the filling of prestressing ducts in concrete structures. In this application the purpose of the grout in the duct is generally two-fold:

(a) to protect the stressing tendons against corrosion
(b) to provide an effective bond between the prestressing tendons and the structure

The strength required of the grout to develop the necessary bond is often quite limited and readily achieved with Portland cement grouts. The prevention of corrosion requires that the tendons are isolated from aggressive agents such as the combination of water and oxygen. This may be achieved by completely surrounding the tendon with grout. However, it is now recognised that this is very difficult to achieve and there is a move to ensure that the duct itself is also completely sealed against the ingress of aggressive agents so that the grout becomes a second line of defence (Concrete Society Design Group, 1993). In addition to providing a physical barrier between the tendons and aggressive agents, Portland cement grouts have the important feature that they are strongly alkaline and thus will inhibit the corrosion of steel provided that this alkalinity is not neutralised by reaction with atmospheric carbon dioxide. In a sealed duct there should be no possibility of carbonation. However, if there is a break in the duct carbonation can be a serious issue as the thickness of grout around the tendon may be quite modest (see section 10.3).

Thus the ideal grout should have adequate strength and completely surround the tendons with a durable alkaline material which prevents the ingress of corrosive agents. In practice it can be very difficult to achieve the full covering of the tendon. There are a number of major factors which may lead to an unsatisfactory final product including:

- Inadequate grouting techniques:
  poor workmanship, insufficient training of grouting operatives
  lack of quality control of grout preparation and injection
  failure to appreciate the properties of the grout
  injection of an insufficient quantity of grout to fill the duct or to sweep out trapped air or water

- The nature of the void to be filled:
  failure to appreciate the nature of the grout flow in the void
  failure of the grout to displace all air or water from the duct
  lack of venting to release trapped air
  leakage of grout from the duct or the anchorages or stressing points
  loss of grout volume due to filtration of water into the tendons, at anchorages or stressing points or from leaks in the duct itself
- Properties of the grout:
  bleeding (water segregation)
  shrinkage and/or cracking of the grout during setting and hardening
  inappropriate rheology
  failure of the grout to penetrate all the interstices of the tendon/duct system
  the chemistry and fineness of the cement

As grouting materials have been discussed in detail in earlier chapters, this chapter will be devoted mainly to the physical processes which occur in duct grouting.

10.2 Grouting practice

In the foreword to the Fédération Internationale de la Précontrainte (FIP, 1990), *Grouting of tendons in prestressed concrete, Guide to good practice,* the following statement is made:

> 'The importance of proper tendon grouting for the achievement of durable corrosion protection and for efficient bonding of the prestressing steel to the concrete structure has long been recognised. However, the execution of self-evident procedures tends to be taken rather lightly. In recent years, many prestressed concrete structures have been inspected, and surprisingly often tendons were found which were not properly grouted or even in some cases ungrouted.'

While in the past 'the execution of self-evident procedures' may have been 'taken rather lightly' it is to be hoped that the current greater awareness of quality assurance will ensure that consistent and documented practices are followed. More importantly what still may be taken lightly is to assume that good practice is self-evident. Cement grouts are extremely complex fluids and even the simplest duct is actually a very complex void. To assume that appropriate grouting practices are always self-evident could be to fail to recognise the complexity of the situation. Of course much experience of grouting exists and a corpus of knowledge as to what represents good practice has been established. However, small changes in duct profile or grout behaviour may require major changes in grouting practice and thus full-scale trials are an essential part of most grouting operations.

It should also be noted that it can be relatively easy to ensure good workmanship when the product of the work is visible and easily inspected but it is much more difficult to ensure that appropriate procedures are followed when the product is inaccessible. In duct grouting the grouted void is invisible to the operator and short of destroying the duct it can be checked only by non-destructive techniques such as γ-radiography or acoustic wave techniques. Neither of these techniques is currently available in a form which can be applied during grouting. Thus the operator has to rely only on experience gained from grouting trials and the published literature.

Thus to ensure good workmanship it is essential that the operatives have been properly trained, that the duct and the injection and venting points have been appropriately designed, and that appropriate plant is available on site to mix and pump the grout.

Despite the fact that duct grouting has been widely used for a substantial time it must be noted that there has been regular concern about the effectiveness of such grouting. In an investigation of post-tensioned concrete bridges in the United Kingdom, Woodward (1981a; Woodward and Miller, 1990) found voids in over half the ducts examined and in many instances tendons were exposed. In Japan, Kabuta and Ohta (1978) found voids in 35% of the ducts examined and 10% of the ducts were less than half filled with grout. The corrosion of tendons as a result of ingress of aggressive agents has been reported on a number of occasions, for example in Wales a bridge collapsed in 1985 (Woodward and Wilson, 1991) as a result of corrosion of tendons. In September 1992 the United Kingdom Department of Transport announced that:

> 'pending a review of standards, no more grouted-duct post-tensioned concrete bridges would be commissioned by the Department' (Concrete Society Design Group, 1993).

10.3 The duct

There is a tendency for specifications to focus on the properties of the grout but not address the nature of the void in any detail. It is easy to see why this is so, for the grout is the permanent material and the void should exist only until it is filled with grout. However, an understanding of the way in which a grout may behave as it is pumped into a void is fundamental to achieving full grouting. For example, no matter what the properties of the grout, a void which is inadequately vented can never be fully grouted.

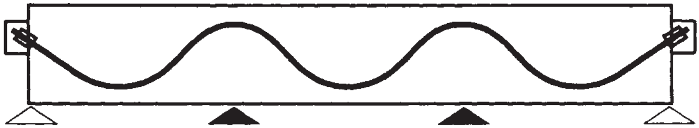

Figure 10.1 Schematic diagram of a duct in a bridge deck beam.

Typically a duct may be from 25 to over 100 mm in diameter. Although for the smaller ducts the tendon must be inserted prior to concreting, the minimum diameter for ducts in which the tendon is installed after concreting is in the order of 40 mm. Typically the tendon will occupy from 40 to 50% of the gross cross-sectional area of the duct. Thus for a 100 mm duct with the tendon bundle in the centre, the annular space to be grouted may have a width of, at most, 14 to 18 mm. However, the tendons are unlikely to be axisymmetric with the duct unless it is entirely straight. For example, the ducts in a bridge deck beam may be 'W' shaped in elevation as the tendons will follow the line of tension in the structure (see Figure 10.1). The stress in the tendons will pull them down to the invert of the duct at high points and up to the soffit at the low points. The space to be grouted is therefore complex and failure to appreciate its profile and varying geometry can lead to the failure of the grout to displace all the air from the duct and thus to the trapping of air voids within it.

10.3.1 Trapping voids in vertical ducts

It could be assumed that the trapping of air voids in ducts is a problem that occurs only in inclined or, more particularly, undulating ducts. However, voids can be trapped even in vertical ducts.

Consider grouting of a vertical duct. In principle, for upward injection in a vertical duct the grout should have a near horizontal surface and advance uniformly up the duct. However, if the tendon bundle is eccentric so that the gap between the tendons and duct wall is not equal in all directions it is possible that the grout may follow a preferred flow path in the wider section of the annulus so that the narrower section is filled at a slower rate or perhaps not at all if the grout has a yield stress. As the grout advances along such a preferred path it may at some stage spread so that it once again occupies the full duct cross-section, particularly if the grout reaches a region where the tendons are more central. In this way an air void may be closed off. Such a void can be removed only if later flow sweeps it from the duct or if the duct is pressurised and the trapped air can escape (perhaps along the tendon bundle). If the air cannot escape the void cannot be fully removed by pressurisation. Void formation is thus possible in the simple geometry of upward flow. It is much more likely to occur in inclined or undulating ducts, particularly in regions where the duct slopes downwards in the direction of flow.

In vertical ducts the risk of trapping voids can be reduced by limiting the injection rate so that the grout has time to spread laterally under gravity to fill any potential voids before they are closed off by the advancing grout. The FIP (1990) states that the rate at which grout is pumped into the duct can vary from 5 to 15 m/min but notes that a low rate is recommended for vertical ducts. However, this recommendation is probably not to reduce the risk of trapping voids but rather to reduce the injection pressure which can be large in tall ducts due to the contribution of the hydrostatic pressure.

While limiting the injection rate may help to limit the occurrence of gross voids it will not ensure that the grout fully penetrates all small interstices such as those of the tendon bundle and the contact points between tendon and duct wall. For this it may be necessary to use an admixture such as a superplasticizer to reduce the yield stress of the grout (see section 10.5.1).

In undulating or downward sloping ducts there is a very substantial risk of trapping voids and the situation cannot be improved by reducing the injection rate. In such ducts, as grout is injected it may satisfactorily fill the sections where the flow is upward provided the injection rate is again small compared with the rate of spreading of the grout across the duct by gravity induced flow. However, where the flow is downward the grout will not tend to spread across the full cross-section of the duct but will flow along the invert and run down to the next low point where it will build up and trap a void as shown in Figure 10.2. Clearly this type of break-away from full duct flow to preferential flow along the invert will be more likely to occur with Theologically thin grouts (for example superplasticized grouts) than thicker more paste-like materials. In principle it should be possible to design a grout that is sufficiently stiff that it can be squeezed along the duct rather like a piston and so continue to fill the full duct crosssection. However, Jefferis and Forrester (1991) found that break-away occurred even with the stiffest grout that could be usefully pumped. As already noted, stiff (high yield stress) grouts may show poor penetration of interstices. Thus attempts to control break-away by thickening the grout are not recommended. Furthermore, the trapping of air voids will be exacerbated by the shear rate-dependent and time-dependent rheology of cement grouts (and especially thick grouts) as both these properties are likely to encourage flow along preferred paths because in regions where the grout is moving slowly it will have a higher apparent viscosity and the longer residence time will mean that the grout is older and thus stiffer.

It is therefore recommended that with regard to the elimination of voids

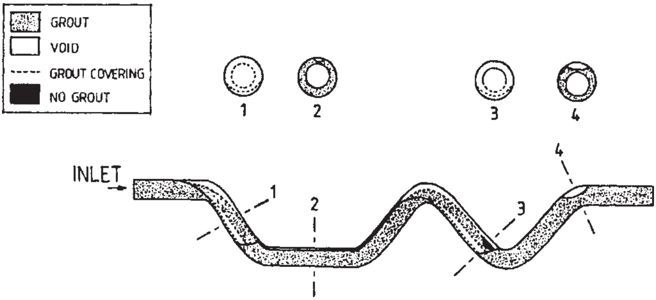

Figure 10.2 Voids trapped in the descending limbs of an unvented duct.

attention is focussed on venting the duct rather than trying to develop an ideal rheology.

10.3.2 Venting the duct

It is essential that any duct is properly vented so that air can escape as the duct is grouted. If the grout flow is always upward (i.e. in vertical ducts or ducts which are inclined upwards in the direction of flow), a single vent at the top may be sufficient provided that the grout injection rate is limited so that no voids are trapped as discussed in section 10.3.1. Where there is downward flow it is still common practice to set vents only at the high points. Thus in the undulating ducts typically found in bridge decks it has been normal practice to set a single vent at each high point (or crown) in the duct. However, it has been found that a single crown vent is not sufficient. Such vents are effective in allowing air to escape as the grout moves up the rising limb before the crown but air can still be trapped in the descending limb as shown in Figure 10.3. To remove this air it is necessary to include a second vent part way down each descending limb (descending in the direction of flow). The sequence of

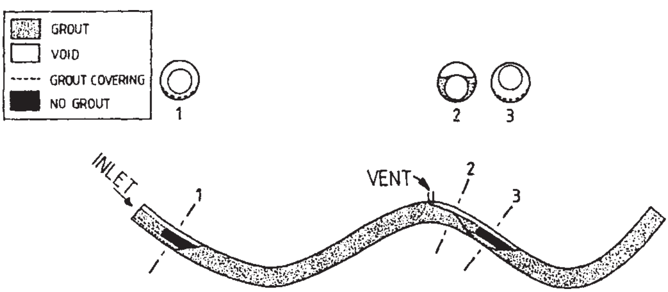

Figure 10.3 Voids trapped in the descending limbs of a duct with a single vent at the crown.

opening of these two vents also is important. The crown vent must be opened only after a significant quantity of air-free grout has been allowed to discharge from the downstream vent. If the crown vent is closed before the downstream vent a void may remain at the crown.

The ideal situation for grout injection is always to grout from low points to adjacent high points so as to ensure that there is never downward flow. However, for many structures installing the necessary injection points would present serious practical difficulties. For example, it could mean injecting from the underside of a bridge deck.

If only crown vents are used (without associated downstream vents) then grouting may be improved by injection from both ends of the duct so that all sections of the duct are subjected to upward flow at some stage of the injection process. Substantial quantities of grout will need to be wasted at each vent to ensure that all trapped air is discharged. Grouting from both ends may require a grout mixing plant at each end for long ducts.

10.3.3 Volume of grout to be discharged at vents

It is common practice to discharge only a small volume of grout at each vent prior to closing the vent. Typically this volume may be a few litres or sometimes a little more if rheological or strength tests are to be carried out on the discharged grout. The reason for this appears to be that there is an established view that venting large volumes of grout does not lead to any more air being expelled than is achieved with the venting of a few litres. However, it should be remembered that the cost of the grout will be a tiny fraction of the cost of the structure and a failure of the grout to provide full corrosion protection to the tendons may greatly reduce the operational life of the structure.

Grouting specifications should quantify the amount of grout to be discharged at each vent and the author would prefer that this quantity was equal to at least the net volume (i.e. that not occupied by tendon) of the duct between the vent and the immediately previous vent (or for the first vent from the injection point). A volume reconciliation also ought to be undertaken such that the volume of grout mixed is compared with the net volume of the duct and the quantity of grout discharged or otherwise wasted. As a possible means of enabling an immediate check to be made, some specifications require that a sufficient quantity of grout to fill the duct (and presumably including an allowance for venting, though this may not be explicitly stated) is mixed prior to the start of grouting and that this grout is held in an agitated holding tank. This may be reasonable practice for short ducts but for long ducts it can lead to the grout being held for an unnecessarily long time before injection.

10.3.4 Grouting rate

British Standard 8110 (British Standards Institution, 1985) recommends that the rate of advance of grout in the duct should be 6 to 12 m/min. The FIP (1990) suggests that the rate can vary from to 5 to 15 m/min and Woodward (1981b) recorded rates of 13 to 22 m/min in an investigation of the grouting of five prestressed concrete bridges. For flow rates of this order the pumping pressures may be dominated by the yield stresses of the grout with the plastic viscosity making a more minor contribution. As a result, the pumping pressures may not vary greatly with flow rate (Jefferis and Mangabhai, 1988) though this will depend on the nature of the grout. Grouts normally will be pumped with some form of positive displacement pump such as a Mono pump. The flow rate from such a pump cannot be controlled by throttling the flow as this would damage the pump. It can only be controlled by varying the speed of the pump (or bleeding off and recycling part of the flow, a procedure which is difficult to control). Often fixed speed pumps are used and thus the size of the pump should be related to the cross-sectional area to be grouted. This area will vary from job to job and it may not be possible to obtain a perfect match of pump and duct. Thus when designing grouting operations it is as well to ensure that the grouting rate is not a critical parameter.

10.3.5 Grouting pressure

Typically grouting pumps are required to be able to maintain a pressure of at least 1 MPa and to have a relief valve to prevent pressures exceeding 2 MPa. Some specifications may limit the pressure at the pump to a lower value. Woodward (1981b) reports an investigation of grouting where the specification required a maximum pressure of 0.7 MPa though this was frequently exceeded during grouting. It is common practice to pressurise the duct at the end of the grouting operation and specified pressures may be up to 1 MPa.

10.3.6 Modelling grout flow in a duct

For many problems in hydraulic engineering it is possible to investigate flow behaviour with scale models. To model the flow in duct it is necessary to match a number of parameters in the model and the prototype. These include the shear rate (the flow properties of grouts are shear rate dependent), the Reynolds number and the Hedstrom number.

For a Newtonian fluid in an annulus the shear rate, may be approximated as:

$$\dot{\gamma} = 6v/a \qquad (10.1)$$

The Reynolds number, Re, is:

$$\mathrm{Re} = \rho v a/\eta \tag{10.2}$$

and the Hedstrom number, He, will be of the form:

$$\mathrm{He} = \tau_0 \rho a^2/\eta^2 \tag{10.3}$$

where v is the velocity of flow in the annulus, a is the annular gap, i.e. the difference between the inner and outer radii of the annulus, ρ is the density of the fluid, η its plastic viscosity and τ_0 its yield stress. The significance of the Hedstrom number is discussed in Govier and Aziz (1972). For typical grout/duct combinations the Hedstrom number may be less than 100 and so will have relatively little effect on either the predicted pressure drop during grout injection into the duct or on the critical Reynolds number for transition from laminar to turbulent flow in the duct.

For any modelling exercise it would be necessary to use the same grout as the fluid in the model and the prototype, otherwise it would not be possible to model the time and shear history dependence of the grout. Thus ρ, η and τ_0 cannot be varied between model and prototype.

For a fixed grout rheology, consideration of the groups shows that equality of shear rate and Reynolds number (and Hedstrom number if significant) between model and prototype can be achieved only if a (the annular gap) is also held constant. Thus, in principle, it is not possible to use scale models matching the above parameters for the investigation of grout flow.

It is therefore important when undertaking grouting trials to use the fullscale duct with the appropriate tendon bundle. However, the full length of the duct need not be modelled. Critical sections may be investigated provided that a reasonable entry and exit length are modelled to ensure the correct flow profile in the critical section.

While the above groups are those typically used for pipe flow situations a number of other dimensionless groups could be considered for particular types of grout flow. For example, the Froude number (v^2/gd, where g is the acceleration of gravity and d the depth of flow) could be useful for modelling the free surface flow that occurs in descending limbs of an undulating duct.

10.4 Preparation of the grout

The fundamental parameters influencing the behaviour of grouts are the water/cement ratio, the cement type, chemistry and source, the mixer type and the mixing time and batch size, admixtures, etc. These parameters are discussed in some detail in chapters 1 and 2 and thus only issues specific to duct grouting will be considered here.

10.4.1 Cement types

The FIP (1990) recommends that ordinary Portland cement is used to prepare duct grouts though blast furnace cement may be used if it was also used in the structure, and fine-grained cement may be used in cold climates.

While ordinary Portland cement grouts will have hardened properties which are relatively consistent, their fluid properties can show considerable variation. This is not surprising as Portland cements are made from natural materials and thus some variation between works must be expected. Furthermore most standards for Portland cements focus on the hardened properties and do not include controls on the fresh behaviour and in particular on the rheology of grouts made from them. Thus cements from different works may show markedly different rheologies when prepared under identical conditions and at the same water/cement ratio. For example, vom Berg (1979) found that a 10% change in the specific surface area of a cement could increase the plastic viscosity of a grout by a factor of 5 and the yield stress by a factor of 15. The alkali content and the aluminate contents can also be very significant. An increase in either tends to reduce fluidity at constant water/cement ratio. Muszynski and Mierzwa (1990) suggest that cements suitable for duct grouting should have a total silicate/ total aluminate ratio within the following range:

$$3.5 < (C_3S + C_2S)/(C_3A + C_4AF) < 5.3 \tag{10.4}$$

Low values of the ratio should give less fluid grouts although the author has found that the ratio may not correctly rank sulphate-resisting cements which have a low C_3A content but can have a high C_4AF content.

The fluidity of the grout is not always a major issue for, as already noted, ducts can be injected with relatively stiff grouts provided that the venting arrangements are appropriate. However, the trend in current grout design seems to be towards the development of low water/cement ratio (generally less than 0.4) high fluidity grouts. The aim of this is to develop grouts which do not push air from the duct piston-like as the grout advances (see section 10.3.1) but are sufficiently fluid that any trapped air tends to rise through the grout and be discharged from the vents. The low water/cement ratio is necessary to produce a low porosity hardened material which gives best protection to the tendons. If high fluidity is to be achieved at low water/cement ratio then sulphate-resisting cements can be used to advantage as they will have low C_3A contents. A further possibility is to use an oil well cement. An advantage of these cements is that they are more tightly specified than ordinary Portland cements and the thickening time is a specified parameter (Bensted, 1989). (The thickening time is an empirical rheological parameter measured as the time to reach a specified viscosity in an oil well consistometer, American Petroleum Institute, 1990.) Thus batches of oil well cement from different works should show more consistent rheology than corresponding batches of ordinary Portland cement. Class B oil well cement, which is a sulphate-resisting cement, has been widely used in off-shore structural grouting. Class G cement also could be used. It is a sulphate-resisting cement but is slightly more tightly specified than Class B. The necessity for minimum variability in the cement stems from the fact that laboratory trials will normally precede any major or complex grouting operation. These trials will include tests on the fresh material and also on the hardened grout, for example the 28 day crushing strength. Thus the trials may have to take place at least 28 days prior to the grouting operation. Of course the cement for any trials and the main operation should be obtained from the same works. However, 28 days is long enough for the cement from a single works to show some variation and thus a tight material specification is useful.

A further advantage of using tightly specified cements is that once a successful mix design has been developed it can be translated to similarly designated cements from other works with limited modification. A grout based on ordinary Portland cement may have to be substantially modified if the source of the cement is changed.

The age of the cement also can significantly affect the rheology of a grout. Thus specifications may stipulate the maximum age for the cement and require that it is stored in a manner to minimise aeration (contact with the atmosphere).

With 'young' cement fresh from the works the powder temperature can be significantly above ambient temperature and this can cause variation in grout rheology as can long mixing times which may lead to high grout temperatures. The FIP (1990) requires that the grout temperature after mixing shall not exceed 40ºC.

Silica fume is finding increasing application in grouts as it can be used to produce low bleed grouts with very low hardened porosity. Silica fume grouts will include a superplasticizer and if well formulated can show very high fluidity and yet low bleed.

10.4.2 Mix water

Typically specifications require that the mix water should be of potable quality. The maximum permitted concentration of salts in potable waters if often about 2000 mg/litre. At this concentration the salts are unlikely to have any significant effect on the hydration of cement. However, at such concentrations there could be concern about the corrosion of prestressing strands. The FIP (1990) recommends a maximum chloride ion concentration of 500 mg/litre and this could occur in some potable waters, though it would be rare in tap water. The electrical conductivity of a water can be used to obtain a rapid indication of its total dissolved solids content. A water containing 500 mg/litre of chloride ions could have an electrical conductivity of 1300 microsiemens/cm and thus if the conductivity of a potential mix water approaches this value it may be unsuitable and chemical testing will be necessary.

10.4.3 Gas expansive admixtures

Very many different admixtures may be used to modify the properties of duct grouts and superplasticizers find particular application. These materials are now relatively well understood and cause no particular problems in duct grouting. However, one class of admixtures, the gas expansive materials, requires comment.

Gas expansive admixtures are often suggested as a means of preventing voids in ducts. However, as noted in section 2.14.3, they must be used with great care and their use by no means guarantees the elimination of voids. If a gas expansive agent is to displace voids then the duct must be vented until all voids have been displaced. However, the venting process may lead to channels being blown through the stiffening grout. Figure 10.4 shows the channelling that occurred in an 8.5 m long trial duct grouted with an expansive mix with a single vent at the crown. It can be seen that the expanding grout has not displaced the voids in the two descending limbs (see section 10.3.2) and that it has blown a channel through each ascending limb.

This should not be taken as suggesting that expansive agents are not useful in duct grouting but merely that the admixture concentration must be selected with care and that trials may be necessary to establish the optimum venting arrangements and, if appropriate, the time at which the vents should be closed. Gas expansive admixtures have a special role in some grouting operations but they should not be relied on to fill voids created by poor grouting practice.

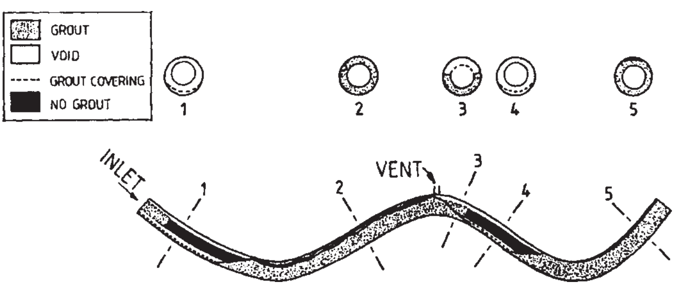

Figure 10.4 Unfilled voids and gas channelling in a duct grouted with a gas expansive mix.

10.5 Properties of the grout

The flow, bleeding and filtration properties, etc. of grouts are also discussed in chapter 2 and thus only those issues particularly germane to duct grouting are included in this chapter.

10.5.1 Penetration of interstices

If a duct were filled with a Newtonian fluid such as water then the fluid would penetrate all the interstices of the system provided there was a driving pressure and any trapped air, etc. could escape. However, cement grouts are not Newtonian fluids but have a much more complex rheology and in particular they show a yield stress, i.e. a certain stress must be applied before flow occurs. This yield value will depend on the water/ cement ratio, cement type, mixer type, admixture used, etc. A grout which has a yield stress, t_0 will penetrate a distance *L,* in a tube of radius *r,* or between parallel plates a distance *r* apart, under a pressure P, where:

$$L = Pr/2\tau_0 \tag{10.5}$$

For example, a pressure of 1000 Pa will be required to force a grout of yield stress, 50 Pa (a possible value, though the potential range is very large) a distance of 10 mm into a slot of width 1 mm. Such a pressure can be produced from a 50 mm head of grout. This head will be amply available in many sections of a duct but problems with penetration could occur at high points where the depth of grout is small or where the interstices are tight, for example at points where the tendon bundle is close to or in contact with the wall of a duct. This may occur in an undulating duct where the tendon is pulled against the wall at low points and especially at high points where the hydrostatic pressure will be a minimum.

Thus a fluid with a yield stress may not fully penetrate interstices of small dimensions. In particular, problems must be expected for penetration between tendons and, as noted above, where the tendons are pulled against the wall of the duct. Such contact points will be determined by the geometry of the duct but in general they must occur in all but entirely straight ducts. At a contact, *r*=0 and thus in principle grout cannot penetrate to the contact point. Capillary effects will help to draw grout into narrow interstices but also, possibly preferentially, help to draw pore fluid rather than bulk grout, especially if the grout has been mixed in a low shear mixer. Thus the mix water is more easily separable from the cement grains than would be the case for a high shear mixed material.

As noted in Table 2.2 high shear mixed grouts tend to have a higher yield stress than low shear mixed grouts. Thus, although high shear mixed grouts are to be preferred because of their lower bleed and better stability against segregation, they can be less satisfactory as regards penetration of interstices. However, the yield stress of cementitious systems can be substantially reduced by adding superplasticizers and they may markedly improve the penetration of grouts into small interstices.

The grouting rate can also affect the penetration of interstices. During grout injection, as the grout moves along the duct the pressure tending to force it radially into interstices or voids will be that caused by the pressure drop due to pumping plus a contribution (negative or positive) as a result of the hydrostatic head of the grout less the pressure in the void itself. Thus at the advancing face of the grout there will be no driving pressure to spread the grout radially except gravity, and in horizontal or downward inclined ducts there is no pressure to fill the entire cross-section. At points behind the advancing front the grout pressure will be that appropriate to the hydrostatic pressure, the pressure gradient and distance from the grout front. Thus the pressure will increase towards the injection point where it will reach the full injection pressure. Any voids remaining after the grout front has passed a point will tend to be filled by this driving pressure (reduced by any internal pressure within the void, for example that due to gas compression if the void is closed). There is therefore a case for maximising the grouting pressure gradient, that is using high injection rates, possibly to a value sufficient to develop turbulent flow in the grout and to lock-off a significant pressure in the duct at the end of grouting. Turbulent flow by inducing higher shear stresses at the solid surfaces may also give a better bond as well as better penetration. However, the necessary velocities would be considerably above the 5 to 15 m/min suggested by the FIP (1990).

Sufficient rates might be achieved with normal grout injection equipment but there could be problems with the allowable pressure in the duct. In oil well cementing, where conductor pipes have to be grouted to the formation and it is necessary to displace a drilling fluid, which may have a significant yield stress, from an annulus which will generally be eccentric, the achievement of high shear rates is very important. It is generally accepted that it is not possible to achieve turbulent flow in the grout but attempts are made to achieve turbulence in a spacer fluid injected between the grout and the in-situ drilling fluid.

Finally, with regard to penetration of interstices, it must be remembered that cement grouts are two-phase fluids and may not behave as homogeneous materials during penetration. Segregation of particle sizes and filtration of water to deposit solids may occur at the entry to small voids, etc.

10.5.2 Bleeding

As shown in chapter 2, in a grout mass the amount of bleed which occurs is controlled by the permeability of the grout to its own pore water, the distance such water must travel to reach a drainage surface and the stiffening time of the grout. Bleed may be very severe in vertical or inclined ducts with unsealed tendons or if the duct is not watertight (that is in situations where short drainage paths exist). In horizontal ducts it may be less significant. In general, bleed is reduced by dispersion of the cement particles as this reduces the permeability of the system. Thus high shear mixing substantially reduces bleed.

It should be noted that the use of the term bleed is unsatisfactory. In duct grouting it is loss of solid volume (surface settlement) that is important. Surface settlement and bleeding are effectively equivalent during the first stages of bleeding. However, bleed water is progressively reabsorbed as the cement hydrates. It is therefore preferable to specify a limit on surface settlement for the set grout rather than a limit on bleed water volume (for example, a typical specification might require the maximum bleed to be <2% and for all bleed water to be reabsorbed within 24 hours). An advantage of specifying surface settlement is that the measurement can be made at 24 hours (or when the grout has set) and there is no need for intermediate examinations of the sample to identify the maximum bleed water volume.

As indicated in chapter 2, the theory of consolidation developed in soil mechanics can be used to predict the settlement of grouts in a number of situations.

A very powerful aspect of the consolidation theory is that it provides a predictive tool for the investigation of the effect of drainage. In general the time to complete consolidation is proportional to the square of the drainage path length. Thus for a vertical duct the time for full consolidation is proportional to the square of the height of the column and as this is likely to be relatively large, the time will be substantial and full consolidation is unlikely to occur as it will be prevented by prior set of the grout. However, if there is a tendon in the duct and it acts as a drainage path, the consolidation will be radial with a path length of perhaps a few centimetres and thus the rate of consolidation may increase by a factor of 10 000 or more. As a result very substantial consolidation can occur in a duct with a 'leaky' tendon and the settlement may reach 10% or more of the height of the column whereas without the tendon it might be less than 0.1% (Jefferis, 1988).

The effect of duct inclination also can be investigated. For an inclined duct (with no drainage to a tendon) the bleed path will no longer be the full height of the column but the vertical path to the upper inclined surface. Thus the path length will be *d/cos?* where *d* is the diameter of the duct and ϕ is the inclination to the horizontal. The bleed water will migrate along this path to the upper surface. It may then move up this surface and if bleed is substantial it may wash cement with it to form an open path along the top of the duct. Alternatively it may accumulate and at some stage the grout mass may become unstable so that the grout slumps and the accumulated bleed water is squeezed out. Both types of behaviour have been observed by the author in the laboratory and have caused significant damage to the integrity of the grout, particularly the slumping of the grout which causes cracks and tears which are not healed by subsequent hydration of the cement.

A further aspect of consolidation theory is that if due to some local disturbance in the duct a bleed lens occurs at some intermediate point in the column, then this lens may be as great as the settlement that occurs at the surface and the theory permits such lenses to occur throughout the column height. If an intermediate lens occurs it may cause substantial damage as it may not be stable but tend to break up so that the grout above it flows down and the water within it flows upwards. In this way even a quite thin lens may damage a substantial height of the grout column.

10.5.3 Filtration

As discussed in chapter 2 filtration may be regarded as a special case of the consolidation in which the driving pressure to expel water from the grout is large compared with the strength of the material.

Filtration may occur into the prestressing strand, at leaks in the duct or at blockages in the duct. The fundamental feature is the loss of water with the deposition of a higher solids content filter cake (a cake of reduced water/cement ratio). It must be appreciated that if such water loss occurs after the completion of grout injection then a void must be developing somewhere within the duct. Thus if water is seen dripping from a duct or from the end of a strand it must be creating a void. It is therefore most important that either ducts and strands are sealed so as to be fully watertight or that after the end of the main grouting operation further grout is pumped into the duct until all leaks have been effectively sealed by local cake deposition. Thus, for example, the FIP (1990) suggests that between 10 and 20 minutes after the end of grouting the ends of the strand are freed of any sealing material and that further grout is injected using a small quantity-high pressure pump (possibly a hand pump) so as to increase the pressure to a predetermined maximum value (which is likely to be in the range of 0.4 to 1 MPa) and further grout should be injected at intervals until the predetermined pressure is sustained. This procedure ensures that a cake is deposited over any leaks and thus ensures that these potential weaknesses in the corrosion barrier are covered with a dense low water/cement ratio grout (the water/cement ratio of the cake will be lower than that of the bulk grout as can be inferred from the darker zone near the strand in Figure 2.6).

10.6 Testing fluid properties of grouts

As was discussed in chapter 2, any tests carried out on duct grouts should be selected to assess the suitability of a grout for use and in particular to discriminate between suitable and unsuitable grouts. However, when reviewing reports on grouting it sometimes seems that tests have been carried out because the test equipment was available rather than because it was appropriate. A very wide range of procedures has been used and there has been little standardisation. Individual workers have tended to use their own preferred procedures. For example, the flow funnel could be regarded as equivalent to the slump cone used for assessing the workability of concrete. However, funnels of many different dimensions have been used. It is to be hoped that the publication of CEN prEN 445 (CEN, 1992a) *Grout for prestressing tendons, Test methods* will lead to greater uniformity. However, it must be accepted that grouts are used for many different applications. For some applications the necessary grout may be effectively a semi-solid paste and for others a free flowing liquid. Few test procedures can resolve such a wide range of behaviour and thus inevitably test equipment must focus on the particular application.

When selecting any test procedure it is important to ensure that the selected procedure is a good indicator of the property of the grout which is to be assessed or is a standard procedure which enables the grout to be ranked in terms of standard properties. At the present time many of the procedures used in grout testing have been borrowed from tests for other materials and in particular many were developed for testing oil well drilling fluids and cementing systems (Rogers 1963; Chilingarian and Vorabutr, 1981). A general discussion on testing grouts and slurries is given in Jefferis (1991).

Selection of the appropriate test procedure can be of particular importance when trying to assess grout rheology.

10.6.1 Rheological testing

There is a very wide range of test procedures available for testing the flow properties of grouts. These range from simple empirical tests such as flow funnels or a flow trough via rotational viscometers such as the Fann viscometer to specialist rheological instruments suitable for use only under laboratory conditions.

10.6.2 Flow funnels

There are very many different flow funnels in common use. These funnels are often described as Marsh funnels. However, there is strictly only one Marsh funnel and this was developed for testing oil well drilling fluids and it is unsuitable for testing most structural grouts as the flow time would be very extended or the flow would stop before the required quantity had been discharged.

For any funnel the test procedure is to fill the funnel with a specified volume of grout or to a specified level and record the time for a specified volume of discharge (in general the specified discharge volume will be less than the volume poured into the funnel as some grout will adhere to the walls of the funnel and it would be difficult to assess at what point the funnel should be accepted as empty if complete discharge were required). When reporting any funnel result it is important that the volume of grout used to fill the funnel, the discharge volume, the discharge time for water and orifice diameter of the funnel are noted. Thus the CEN funnel (CEN, 1992a) has an orifice diameter of 10 mm and is required to be filled to a specified level (a volume of about 1.8 litres) and the discharge is to be 1 litre. The time for water is not included in the specification but is of the order of 6 s. The funnel specified by the FIP (1990) has an orifice diameter of 12.7 mm and a working capacity of about 1.7 litres but neither the quantity of grout to be filled into the funnel nor that to be discharged are specified.

10.6.3 The plunger test

prEN 445 (CEN, 1992a) specifies a plunger test as an alternative to the funnel test. For this test a 62 mm internal diameter tube is filled with 1.9 litres of grout and a 58.2 mm diameter torpedo-shaped plunger is then inserted into the tube and the time it takes to fall through 0.5 m is measured. The draft standard requires that the test is repeated three times and that the average of the second and third tests is recorded. The result of the first test is ignored as it is intended solely to condition the grout so as to enable more repeatable results for the second and third tests. Conditioning is a fundamental problem with grout testing. The rheology of grouts is time dependent and shear history dependent. If the grout is vigorously agitated before testing a more fluid rheology will be recorded. If tests are to be carried out at different ages then the conditioning of the grout between tests (e.g. left quiescent, gently stirred, etc.) may strongly influence the results.

10.6.4 Rotational viscometers

The Fann viscometer is a concentric cylinders viscometer which was developed for testing oil well drilling fluids. It can be used for testing cement grouts but many grouts will be so stiff that the readings will be off the scale. The instrument is designed so that the apparent and plastic viscosities, the yield stress and the gel strength (the yield stress after a rest period without shearing) can be obtained directly. However, the shear rate in the instrument is relatively high and may be much above that in many grouting operations.

The clearance between the rotating sleeve and the stationary bob is only 0.59 mm and thus with stiff grouts it is difficult to be sure that the annulus is completely filled with grout.

The Rion viscometer is also often used. This portable instrument has a slowly rotated bob which is lowered into a test cup filled with grout. The torque on the bob is read directly on a scale as an apparent viscosity. Several bobs are available so a wide range of viscosities may be measured. As the instrument has one fixed speed only an apparent viscosity can be obtained and as the shear rate is low the results may be dominated by the yield stress of the grout (see equation 2.5).

10.6.5 The Colcrete flow trough

This is an empirical test developed by the Colcrete Company (now Keller Colcrete Limited). The test equipment consists of a wide mouthed funnel with an opening of 36 mm mounted over one end of a trough of length about 825 mm, width 100 mm and depth 75 mm. To carry out a test a stopper is fitted into the mouth of the funnel which is then filled with 1 UK quart of grout (1.13 litres). The stopper is then removed and the distance the grout flows along the trough is recorded as the flow. The maximum flow to the end of the trough is 720 mm. The trough should be moistened prior to use and care is necessary to ensure that all old grout is removed so that the walls of the trough are smooth because the reading can be sensitive to the surface condition of the trough. The test is widely used and has been included in a number if specifications for duct and other grouting work.

10.7 Volume change

prEN 445 (CEN, 1992a) specifies three procedures for the measurement of volume change. The first procedure involves the measurement of the amount of bleed water expelled from a short column of grout 3 h after the start of the test. It is thus a bleed measurement at 3 h and the maximum permitted bleed is 2%. In the second test, which may be a continuation of the first, the loss in height of the grout is measured at 24 h and thus it is a measure of loss of solid volume. The maximum permitted loss is 1%. This criterion is more severe than 2% at 3 h as few grouts will expand between 3 h and 24 h unless an expansive admixture has been incorporated. The test also requires that all bleed water has been re-absorbed into the grout mass. This is a common requirement of grout specifications but the reasoning behind it is seldom stated. It would seem that the intention is to ensure that the bleed is small (the presence of free water is easily determined but the measurement of a 1% settlement is more difficult). Also, when grouting in cold weather the absorption of free water could reduce the potential for frost damage. However, if young grouts are exposed to such cold conditions there are likely to be much more severe consequences than freezing of the bleed water. In the third procedure a 100 mm diameter can is filled to a depth of 100 mm with the grout to be tested. A 'stop' plate is then placed on the grout and the displacement of this plate at 24 h is recorded. This third procedure is particularly intended for grouts containing pre-set expansive admixtures.

A feature of all three tests is that the bleed is expressed as a fraction of the original depth of the grout. However, as discussed in section 2.9, bleed is seldom proportional to depth and thus great care is necessary in interpreting and/or scaling up laboratory results.

10.8 Crushing strength

prEN 445 (CEN, 1992a) proposes two tests for measurement of the compressive strength of grouts: tests may be carried out either on the broken halves of prisms prepared in accordance with the flexural strength test of British Standard EN 196–1 or on 100 mm diameter by 80 mm high cylinders. These cylinders may be from the can test for volume change (see section 10.7) with the top (bleed surface) sawn off and both ends of the cylinder ground flat prior to the test.

In the UK it has been common practice to measure the compressive strength of structural grouts on 100 mm cubes and such practice is accepted in prEN 447 (CEN, 1992b), *Specification for common grout*.

Preparation of grout specimens is somewhat different from that for concrete. It is not appropriate to vibrate the moulds as this may cause segregation or enhance bleeding. However, vibration is very effective in releasing trapped air from grouts and possibly the only practicable way of doing it on a reasonable scale. It is therefore useful to vibrate the grout gently in a jug before pouring it into the test mould. After vibration and before moulding the grout should be lightly stirred to homogenise it. The high cement content of most structural grouts (a 0.35 water/cement ratio grout will have a cement content of about 1500 kg/m^3) means that substantial exotherms can occur as a result of the cement hydration reactions unless the samples are cured under water. Common practice is therefore to fill the moulds with grout, strike off with a straight edge and then cover with a glass plate, weight the plate down so as to seal the top of the mould and then place the mould under water. It is, of course, most important that the moulds themselves are watertight. Plastics moulds should not be used for grouts as the low thermal conductivity of plastics may seriously inhibit cooling of the grout. Similarly test moulds larger than 100 mm should be avoided as cooling may be inadequate. With calcium aluminate cements the hydration exotherm is very rapid and particular attention to cooling is necessary (see section 4.2.3).

For critical work it may be appropriate to use temperature matched curing so that the test specimens experience the same temperature regime as the grout in the structure. This can be particularly important for oil well cements where the high temperature regimes can cause significant microstructural changes in the grout.

Grout specimens can be much more brittle than concrete and under test, as failure approaches, splinters of grout may spall from the specimen. The ultimate failure may be quite explosive even in a stiff testing machine.

Typically specifications may require the cube strength of the grout to be at least 20 MPa at 7 days and 30 MPa at 28 days.

10.9 Grout reservoirs

Many grouting specifications include details of grout reservoirs to be used to ensure that any bleed water is displaced by grout. At the simplest these reservoirs consist of a grout filled container connected to the high point(s) of the duct via a pipe. For a pipe of diameter 25 mm filled with grout and freely discharging to atmosphere a gel strength of order 125 Pa will stop flow. While this may be a relatively high gel strength for a grout soon after mixing, such a value may be achieved before all bleeding is complete within the adjacent duct and thus stiffening of the grout may prevent the reservoir from functioning at the very time it is needed, i.e. as bleeding stops. However, this is not the only barrier to flow. In addition to the pressure loss due to the gel it must be remembered that if grout is to displace bleed water then this water must be displaced up into the pipe if the grout is to move down. This mutual displacement seldom occurs in small diameter pipes. Indeed the author has regularly seen voids immediately adjacent to grout reservoir connection points. If reservoirs are to be used then the connection points and associated pipework should be of as large a diameter as possible. Also the efficacy of the reservoir system should be tested prior to use.

10.10 Concluding remarks

Successful filling of voids requires careful attention to the properties of the grout and also the nature of the void to be grouted. It is of particular importance to relate the venting arrangements to the shape of the void to be grouted. In general it is not possible to pump grouts which are so thick that they will move along the duct or void piston-like. Therefore it is preferable to design grouts so that they are of high fluidity and thus have the best chance to penetrate fine interstices and do not tend to trap air within the grout mass. The ideal duct grout would seem to be a low water/ cement ratio material treated with a superplasticizer to reduce yield stress and so improve fluidity and thus the penetration of small voids and interstices. The grout should be high shear mixed to reduce bleed and may contain silica fume to improve its performance as a barrier to the ingress of air, water or other damaging chemicals and also to reduce bleed.

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