Fulton's concrete technology

Ninth edition

Fulton's concrete technology

edited by

Gill Owens

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Entrance to an office block in Nieuw Muckleneuk, Pretoria, designed by Pieter Mathews & Associates Architects.

Photograph courtesy of Daniel van der Merwe.

The PPC Batsweledi Project's new pre-heater tower at Dwaalboom, Limpopo Province.

Photograph by Jorg Jung courtesy of PPC.

Dolosse at the Port of Ngqura (Coega) near Port Elizabeth.

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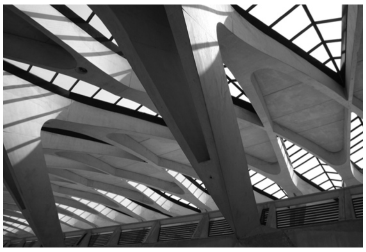

TGV (high-speed train) station at Lyon Saint Exupéry Airport designed by Santiago Calatrava.

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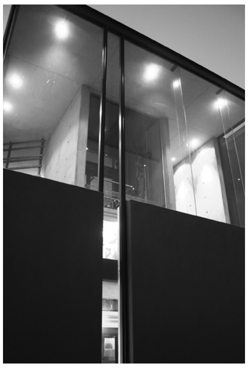
Fairlands Office Park, Gauteng, under construction.

Photograph by Jorg Jung courtesy of Grinaker-LTA/WBHO Joint Venture.



Office block in Nieuw Muckleneuk, Pretoria, designed by Pieter Mathews & Associates Architects.

Photograph courtesy of Daniel van der Merwe.

Preface

This ninth edition of *Fulton's Concrete Technology* provides up-to-date information on materials and other aspects of concrete relevant to South African conditions. Whether it finds its way onto the desks of practitioners, is used as a source document for lecturers, or is referred to as a prescribed work by students, the aim is to make a publication available which consolidates the experience and insights of experts in a single volume.

Some chapters have been revised extensively to reflect new developments and the latest research, and to ensure that the technology presented in this book remains at the highest standard. New authors have come on board and, taking cognisance of the expressed needs of readers of previous editions, five new chapters have been added: repair, reinforcement, formwork, sand-cement mixes and self-compacting concrete.

As in the previous edition, the contents are divided into five broad categories:

- Materials for concrete
- Properties of fresh concrete
- Properties of hardened concrete
- Production of concrete
- Special techniques and applications

The Cement and Concrete Institute is a marketing organisation that aims to grow concrete's market share for its members. It achieves its goals by providing education and training, technical advice and information. *Fulton's Concrete Technology* supports these objectives and strives to ensure that the processes of concrete and concreting are better understood and applied.

The C&CI is supported primarily by its Producer members:
AfriSam (South Africa) (Pty) Ltd
Lafarge Industries South Africa (Pty) Ltd
NPC-Cimpor (Pty) Ltd
Pretoria Portland Cement Company Ltd

The C&CI also enjoys support from its Associate members:
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Khuthaza Development Solutions
South African Coal Ash Association
South African Federation of Civil Engineering Contractors
Southern African Readymix Association
Terex Africa
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Chapter 1
Cementitious materials

Graham Grieve

1.1 Introduction

"Cement" is generally defined as a binder, glue or adhesive. Cement in hardened concrete and mortar forms a matrix which binds aggregate particles to form a strong, rigid composite.

Cementitious materials for building and construction consist, with very few exceptions, of portland cement on its own or blended with a cement extender. Such materials are supplied in powder form and, if mixed with water, will set and develop strength.

Portland cement is used as a basis of cementitious materials because: it is relatively cheap; setting and hardening take place at normal temperature and pressure, and under water if necessary; and concrete made with portland cement can be made to be strong and durable.

Cement extenders are materials that have cementing properties when used with portland cement. Various types are available. Those used in South Africa are derived from industrial by-products and are therefore an economical substitute for part of the portland cement but have the added advantage of providing certain technical benefits in concrete.

In South Africa, cementitious materials are available as:

- Common cements which include portland cement (PC) on its own, and factory made blends of PC and cement extender or filler.

  Common cements are intended for use in concrete but may also be used in mortar and plaster.

- Masonry cements which are formulated for use in mortar and plaster.

  Masonry cements are normally a blend of PC and finely ground limestone or hydrated lime (calcium hydroxide). Some masonry cements include an air-entraining agent.

- Cement extenders sold separately. Those for which SANS standards currently exist are:
  - Ground granulated blastfurnace slag (GGBS)
  - Fly ash (FA)
  - Condensed silica fume (CSF)
- Other metallurgical slags are also available in South Africa and consideration is being given to developing a test protocol which will help demonstrate the suitability or otherwise of such unproven materials for use in concrete.

Note on terminology

SANS 50197-1:2000 [1.1] "Cement" refers to portland cement clinker, granulated blastfurnace slag, pozzolanic materials, fly ash, burnt shale, limestone and silica fume as "main constituents" of common cements. SANS 50413-1:2004.[1.2] "Masonry Cement" refers to the "constituents specified in SANS 50197-1 but also permits the incorporation of natural mineral materials, mineral materials used for or derived from the clinker production process, hydrated and/or hydraulic building limes and inorganic pigments. SANS 1491[1.3-1.5] refers to ground granulated blastfurnace slag, fly ash and silica fume as "cement extenders."

The various parts of SANS 1491 were initially adopted in 1989 and the terminology is now familiar to practitioners in South Africa. For this reason the term cement extenders is used in this book.

The aims of this chapter are:

- To provide practical guidelines for the use of cementitious materials.
- To describe, briefly, the manufacture of portland cement and cement extenders.
- To discuss chemical and physical properties of portland cement and cement extenders.
- To provide a bibliography for the reader who requires information at a higher technical level.

This chapter is presented at three different technical levels: practical information for the practitioner; theoretical background; and a bibliography for further study. While information at the practical level should be sufficient for most purposes, the reader is encouraged to read the theoretical background in order to gain a better understanding of the subject.

The information included in this chapter is intended primarily for those involved in designing, specifying and using concrete and mortar. A detailed description of the manufacture of cementitious materials is outside the scope of the chapter.

1.2 Specifications and test methods

1.2.1 Common cements

The South African standard for "Common cements" is SANS 50197-1 *Cement. Part 1: Composition, specifications and conformity criteria for common cements*[1.1] and this is supported by SANS 50197-2:2000 *Cement. Part 2: Conformity evaluation.*[1.6]

Definitions

The standard defines the following constituents:

- Portland cement clinker, in terms of composition and how it is made
- GGBS
- Pozzolanic material

Table 1.1: The 27 products in the family of common cements

Main types	Notation of products (types of common cement)		Composition, percentage by mass(a)										
			Clinker	Blast-furnace slag	Silica fume	Pozzolana		Fly ash		Burnt shale	Limestone		Minor additional constituents
						natural	natural calcined	siliceous	calcareous				
			K	S	D(b)	P	Q	V	W	T	L	LL	
CEM I	Portland cement	CEM I	95 - 100	-	-	-	-	-	-	-	-	-	0 - 5
CEM II	Portland-slag cement	CEM II A-S	80 - 94	6 - 20	-	-	-	-	-	-	-	-	0 - 5
		CEM II B-S	65 - 79	21 - 35	-	-	-	-	-	-	-	-	0 - 5
	Portland-silica fume cement	CEM II A-D	90 - 94	-	6 - 10	-	-	-	-	-	-	-	0 - 5
	Portland-pozzolana cement	CEM II A-P	80 - 94	-	-	6 - 20	-	-	-	-	-	-	0 - 5
		CEM II B-P	65 - 79	-	-	21 - 35	-	-	-	-	-	-	0 - 5
		CEM II A-Q	80 - 94	-	-	-	6 - 20	-	-	-	-	-	0 - 5
		CEM II B-Q	65 - 79	-	-	-	21 - 35	-	-	-	-	-	0 - 5
	Portland-fly ash cement	CEM II A-V	80 - 94	-	-	-	-	6 - 20	-	-	-	-	0 - 5
		CEM II B-V	65 - 79	-	-	-	-	21 - 35	-	-	-	-	0 - 5
		CEM II A-W	80 - 94	-	-	-	-	-	6 - 20	-	-	-	0 - 5
		CEM II B-W	65 - 79	-	-	-	-	-	21 - 35	-	-	-	0 - 5
	Portland-burnt shale cement	CEM II A-T	80 - 94	-	-	-	-	-	-	6 - 20	-	-	0 - 5
		CEM II B-T	65 - 79	-	-	-	-	-	-	21 - 35	-	-	0 - 5
	Portland-limestone cement	CEM II A-L	80 - 94	-	-	-	-	-	-	-	6 - 20	-	0 - 5
		CEM II B-L	65 - 79	-	-	-	-	-	-	-	21 - 35	-	0 - 5
		CEM II A-LL	80 - 94	-	-	-	-	-	-	-	-	6 - 20	0 - 5
		CEM II B-LL	65 - 79	-	-	-	-	-	-	-	-	21 - 35	0 - 5
	Portland-composite cement(c)	CEM II A-M	80 - 94	← 6 - 20 →									0 - 5
		CEM II B-M	65 - 79	← 21 - 35 →									0 - 5
CEM III	Blastfurnace cement	CEM III A	35 - 64	36 - 65	-	-	-	-	-	-	-	-	0 - 5
		CEM III B	20 - 34	66 - 80	-	-	-	-	-	-	-	-	0 - 5
		CEM III C	5 - 19	81 - 95	-	-	-	-	-	-	-	-	0 - 5
CEM IV	Pozzolanic cement(c)	CEM IV A	65 - 89	-	← 11 - 35 →					-	-	-	0 - 5
		CEM IV B	45 - 64	-	← 36 - 55 →					-	-	-	0 - 5
CEM V	Composite cement(c)	CEM V A	40 - 64	18 - 30	-	← 18 - 30 →			-	-	-	-	0 - 5
		CEM V B	20 - 39	31 - 50	-	← 31 - 50 →			-	-	-	-	0 - 5

Notes

(a) The values in the table refer to the sum of the main and minor additional constituents.

(b) The proportion of silica fume is limited to 10%.

(c) In portland-composite cements CEM II A - M and CEM II B - M, in pozzolanic cements CEM IV A and CEM IV B, and in composite cements CEM V A and CEM V B the main constituents other than clinker shall be declared by designation of the cement.

- FA
- Burnt shale
- CSF
- Filler, which can be inert or have slight hydraulic or pozzolanic properties
- Calcium sulphate
- Additives, which are not included in the above and are added to improve the manufacturing process or properties of the cement

Composition

The standard specifies composition of cements according to the proportions of constituents, ie portland cement clinker, extenders and fillers. Table 1.1 lists the spectrum of permitted products in SANS 50197-1.

Mechanical requirements

The standard specifies strengths that are determined in accordance with SANS 50196-1 *Methods of testing cement. Part 1: Determination of strength.*[1.7] The method requires three prismatic test specimens, 40 x 40 x 160 mm, to be made for each test. The specimens are cast from a batch of plastic mortar containing, by mass, 1 part cement, 0,5 parts water and 3 parts standard sand. The specimens are cured in water at a standard temperature. Each specimen is broken in two and each half is tested in compression between platens measuring 40 x 40 mm. The average of six strength determinations represents a single result.

Required strengths are shown in Table 1.2. Note that strengths must clear an early-age "hurdle" and fall within a "window" at 28 days.

Table 1.2: Compressive strength requirements at stated ages

Strength class	Compressive strength, MPa			
	Early strength		Standard strength	
	2 days	7 days	28 days	
32,5 N	-	≥ 16,0	≥ 32,5	≤ 52,5
32,5 R	≥ 10,0	-		
42,5 N	≥ 10,0	-	≥ 42,5	≤ 62,5
42,5 R	≥ 20,0	-		
52,5 N	≥ 20,0	-	≥ 52,5	-
52,5 R	≥ 30,0	-		

Physical requirements

The standard gives requirements for initial setting time and a maximum expansion determined in accordance with the standard method.

Chemical requirements

The standard gives limits for:

- Loss on ignition
- Insoluble residue
- Sulphate content
- Chloride content
- Pozzolanicity (CEM type IV only)

Conformity criteria

Conformity of cements is required to be verified by means of a statistical quality control scheme based upon continuous inspection of manufactured cement. Assessment requires continuous sampling inspection using spot samples. The standard, SANS 50197-2:2000 *Cement. Part 2: Conformity evaluation,*[1.6]gives minimum testing frequencies and conformity criteria and procedures, for strength and physical and chemical properties.

This standard specifies:

- Factory production control
- Auto-control testing of samples
- Surveillance, assessment and acceptance of the factory production control
- Audit testing of samples
- Initial inspection of the factory and the factory production control
- Certification of conformity
- Procedures in the event of non-conformity

1.2.2 Masonry cements

SANS 50413-1:2004 *Masonry cement. Part 1: Composition, specifications and conformity criteria*[1.2] is the National standard for masonry cements, and makes use of SANS 50197-2 for specifying the method of conformity evaluation. SANS 50413-2[1.8] covers the test methods which apply to masonry cements only.

Definition

The standard defines masonry cement as "a factory made finely powdered hydraulic binder which relies essentially upon the presence of portland cement clinker to develop strength. When mixed with sand and water only and without the addition of further materials it produces a workable mortar suitable for use in rendering, plastering and masonry work."

Masonry cements are not intended for use in concrete applications.

Composition

The standard specifies that masonry cement shall consist of PC clinker, inorganic material, and where appropriate organic material, see Table 1.3.

Table 1.3: Composition of masonry cements

Type	Content, %	
	Portland cement clinker	**Organic material**
MC 5	≥ 25	≤ 1
MC 12,5	≥ 40	
MC 12,5X		
MC 22,5X		

Table 1.4: Compressive strength and air entrainment of masonry cements

Type	7-day strength, MPa	28-day strength, MPa		Air-entraining agent
MC 5	-	≥ 5	≤ 15	Required
MC 12,5	≥ 7	≥ 12,5	≤ 32,5	Required
MC 12,5X				Not permitted
MC 22,5X	≥ 10	≥ 22,5	≤ 42,5	Not permitted

Classes of masonry cements

The standard classes masonry cements on the basis of 28-day strength determined in accordance with SANS 50196-1 *Methods of testing cement. Part 1: Determination of strength.*[1.7] Table 1.4 gives strength requirements and specifies the inclusion or omission of an air-entraining agent.

Physical requirements

The standard gives limits for:

- Sieve residue
- Initial and final setting time
- Soundness expansion
- Air content and water retention of fresh mortar

Chemical requirements

The standard gives limits for:

- Sulphate content
- Chloride content

Marking

The standard gives requirements for marking on bags and documentation for the supply of cement.

Table 1.5: Effects of extenders on properties of concrete

Extender	Effects	Suitability for use in:				
		Mass concrete pours	**Shotcrete**	**Chloride exposure**	**Sulphate exposure**	**ASR conditions**
GGBS	**Fresh concrete** • May improve workability slightly. • Retards setting slightly. **Hardened concrete** • Slows development of strength. • Increases later age strength, eg at 90 days. • Refines pore structure and reduces permeability. • Increases rate of carbonation (See Chapter 9.) • Prevents or retards alkali-silica reaction (See Chapter 10.) • Binds chlorides and reduces the chloride induced corrosion of embedded steel. • Reduces rate of heat generation caused by cementing reactions.	High GGBS contents (>50%) help reduce the risk of thermal cracking	No particular benefit	GGBS is particularly suited to chloride capture and provides significant enhancement of resistance to chloride attack of reinforcement	Get specialist advice based on latest research findings from C&CI	Requires >40% GGBS content to control potential ASR for sensitive aggregate types
FA	**Fresh concrete** • Improves workability and reduces water requirement for a given slump. • Slightly retards setting. **Hardened concrete** • Slightly reduces rate of strength development. • Increases later age strength, eg at 90 days. • Reduces the rate of chloride diffusion through concrete. • Refines pore structure and reduces permeability. • Prevents or retards alkali-silica reaction (See Chapter 10.) • Improves sulphate resistance. • Reduces rate of heat generation caused by cementing reactions.	FA content of 30% or more will significantly reduce the risk of thermal cracking	Increased paste content should improve adhesion	FA content of 30% or more will enhance resistance to chloride attack of reinforcement	FA content of 30% or more should improve the sulphate resistance of concrete	Requires FA content of >20% to control potential ASR for sensitive aggregate types
CSF	**Fresh concrete** • Reduces workability. • Increases cohesiveness. • Reduces bleeding significantly. **Hardened concrete** • Increases strength. • Reduces permeability.	Not suitable	CSF is particularly suited to reducing rebound in shotcrete applications	Use of CSF has potential to significantly reduce permeability and improve resistance to chloride ingress	Use of CSF has potential to significantly reduce permeability and reduce susceptibility for sulphate ingress	Requires CSF content of >15% to control potential ASR for sensitive aggregate types

SANS 50413-2:2006 *Masonry cement. Part 2: Test methods,*[1.8] includes requirements for the following:

- Testing laboratory and test equipment
- Preparation of standard mortar
- Water retention test method
- Air content test method

1.2.3 Cement extenders

The South African standard is SANS 1491:2005 *Portland cement extenders. Part 1: Ground granulated blastfurnace slag, Part 2: Fly ash* and *Part 3: Condensed silica fume.*[1.3-1.5]

The definitions for each material covered by SANS 1491 are given in sections 1.9.2, 1.9.3 and 1.9.4. The standard also includes chemical and physical requirements, packing and marking, and inspection and methods of test.

A futher part to SANS 1491 is under development and will cover "other supplementary cementitious materials."

1.2.4 Limestone Filler

A filler, ie finely ground limestone, affects the properties of concrete as follows.

Fresh concrete:

- Prolongs the bleeding period but reduces the amount of bleed water
- Has little or no effect on water requirement

Hardened concrete:

- Reduces long-term strength development, ie beyond 28 days compared with other cements, particularly those made with cements that contain FA and/or GGBS

1.3 Properties and uses

1.3.1 Cements for concrete

Properties

Cement for concrete may be a factory made common cement, ie CEM types I to V (see Table 1.1), or a site blend of a common cement and an extender.

The effects of replacing part of the common cement with an extender are briefly summarised in Table 1.5. These effects apply to well-cured concrete (see section 1.6) and generally increase in magnitude with increasing extender percentage.

Selecting cement type

As a general rule, blended cements are likely to be less costly than "pure cements", but when used in cementitious mixes, the mixes made with blended cement tend to be more susceptible to the adverse effects of poor site curing practice. The cement type selected may be either SANS 50197-1[1.1] common cement on its own or a site blend of common cement and an extender. Users should ensure that any extenders to be added at the batching plant meet the requirements of SANS 1491 Parts 1-3.[1.3-1.5]

As a further general rule well-cured concrete made with blended cements or a site blend of cement and a cement extender will be better able to cope with the aggressive effects of normal weathering or more aggressive environmental exposures. Guidance on how to proportion concrete to most effectively cope with aggressive environments is given in Chapter 9.

1.3.2 Cements for mortars and plasters

Masonry cement is especially advantageous when used with sands that are poorly graded (single-sized) or lack fine material. Such sands tend to produce harsh mixes if used with common cements, although the addition of an appropriate quantity of building lime to the mix made with common cement will also give a more friendly mix.

The selection of a good quality sand is the first prerequisite for making a good mortar or plaster mix; such sands should meet the relevant requirements of SANS 1090.[1.9] While any common cement complying with SANS 50197-1 may be used (with or without lime) to produce mortar or plaster mixes, cements with higher extender contents will be more susceptible to negative effects resulting from poor site curing practices.

Masonry cements complying with the requirements of MC 12,5X and MC 22,5X in SANS 50413-1[1.2] are also supplied in some areas in South Africa. These cements should only be used for masonry applications, and the mix proportions should be adjusted in accordance with the manufacturer's recommendations. It should be noted that the mix proportions of mortar or plaster mixes made with masonry cements will normally have to be richer than an equivalent mix made with a common cement.

As indicated in Table 1.4, MC 12,5X and MC 22,5X cement have no air entrainment. Futhermore, these cements may contain little or no hydrated lime in the blend. Such cements will not have significantly beneficial effects on mix quality where poor quality sands are used.

1.4 Storage and handling

In South Africa, cements are made subject to a compulsory standard which makes it illegal to supply cement that does not meet the requirements of the relevant national standard. South African cements may therefore be accepted as conforming with the relevant standard specifications on leaving the factory. Any material containing portland cement may however deteriorate if it is exposed to water either in liquid or vapour form. Cements should therefore be handled and stored in such a way that exposure to water or air is minimised, and the length of time between cement manufacture and cement use is as short as possible.

Portland cement, cement extenders and blends of these materials are, with few exceptions, available in paper or woven polypropylene sacks and in bulk.

1.4.1 Bagged cement

Manufacturers make bagged cement available in a number of ways to suit customer and transport requirements.

In addition to being supplied as loose bags, cement can also be purchased in units of 40 bags (2 tonnes) per unit. Each unit comprises eight layers of five bags each. The units are loaded onto the transport vehicles by forklift at the factory or depot, but have to be unloaded manually at the destination. This is termed "unitised" or "U" cement.

When the units are packed onto returnable pallets, they may also be unloaded at the destination by forklift which reduces unloading times. This is termed "unitised, palletised" or "UP" cement.

When the units are wrapped in plastic sheets, or enclosed in a plastic hood, this is termed "unitised, palletised, covered," or "UPC" cement. The advantage of UPC cement is that it can be stored outside as long as the plastic cover remains intact.

Storage

The cement store should be weatherproof and solidly constructed. It should be provided with a damp-proof floor, which should be covered with a heavy-duty plastic sheet. The bags of cement should be closely stacked in order to reduce air circulating between them to a minimum. They should not be packed against outside walls. The arrangement for stacking should be such that the cement can be used in the order in which it is received. Doors should be opened as infrequently as possible and windows (if any) should be kept closed.

Stacks should not exceed 12 bags in height. For estimating storage space, assume that a cubic metre of space is required to store 20 bags. Allow for a floor loading of 25 kN/m^2. It should be noted that the bag-to-bag friction of polypropylene bags is lower than is the case for paper bags and therefore the "rules" for unrestrained stacking height must be reduced. However, where polypropylene bags are stacked in unitised stacks, the normal stacking height restrictions would apply.

Palletised cement should be stacked not more than two pallets high.

Handling

Cement bags (whether made from paper or polypropylene) cannot withstand rough treatment. Rules for handling bags of cement include:

- Floors and walls of storage sheds and floors of truck bodies must be free from sharp protrusions such as splinters, nails and bolts.
- Cement bags must not be dropped from a height.
- Cement bags must always be properly stacked, and not stored in a disorderly heap.
- Bags must be carried, and not dragged over rough surfaces.
- Scaffolding, trestles or other building equipment must not be placed on top of the cement.
- Workers wearing boots should not walk over stored cement.
- When a bag has been damaged, the cement should be transferred as soon as possible into two empty cement bags. Do not use sugar or fertiliser bags.

1.4.2 Bulk cement

Bulk cement is stored in silos which may be permanent (in precasting factories or readymix works) or temporary (on construction sites).

The silos, which generally have a capacity of between 12 and 50 tonnes, are filled by special bulk transporters operating from the factory or depot. The silo discharges into a cement mass-batcher. Apart from savings on paper sacks, bulk handling of cement offers the following advantages:

- Compact storage
- The silo is weatherproof
- The cement is automatically used in the order in which it is received, reducing the risk of deterioration during site storage
- Elimination of wastage due to broken bags, pilfering and spillage
- Reduction in handling costs
- Charging of mixer to optimum capacity if cement is batched by mass. (Batching the materials on the basis of one 50-kg bag of cement often results in under- or overloading the mixer.)

1.5 Setting and hardening of cement

At normal temperatures, concrete stiffens gradually after being placed, and sets or becomes unworkable after about three to four hours. For a more detailed description of the process, see section 1.11.

1.5.1 False set and flash set

On rare occasions a batch of cement may exhibit a sudden stiffening within a few minutes of adding the mixing water. This condition may be due either to false set, or, more rarely, to flash set.

False set is characterised by a negligible evolution of heat by the cement paste; when the stiffened concrete is re-mixed without addition of any water it regains its plasticity, and may then be placed in the usual manner without any untoward effect. The condition is caused to a certain degree by overheating during grinding. It does not occur often in South Africa because the clinker is cooled to some extent after leaving the kiln.

With flash set the cement paste heats up excessively; reworking does not restore the original workability. This type of set is very rarely encountered in South Africa.

1.5.2 Brief-mix set

This occurs when mixing is not continued for long enough. The phenomenon is attributed to cement grains becoming stuck together by gel which is in the process of forming on cement grains.[1.10] It is most likely to occur when mixes are very rich, ie high cement content and low water:cement ratio (w:c) or mixing is ineffective, eg by hand. The w:c is defined in section 1.7.

1.6 Curing

Cementing compounds in cement paste can form only in water-filled space. This is true irrespective of whether the reaction involved is hydration, as in the case of portland cement and GGBS, or pozzolanic, as in the case of FA and CSF. These reactions are also influenced by temperature: reaction rates increase with increasing temperature and become zero at very low temperatures.

Curing is described as the maintenance of appropriate moisture and temperature conditions to permit the continuation of the hydration or pozzolanic reaction. Curing is important to ensure strength development in concrete and essential for the development of impermeability of the near-surface concrete – an important factor in the potential durability of a structure or element. Practical aspects of curing are dealt with in Chapter 12.

1.7 Water:cement ratio

At any given degree of completion of the hydration or pozzolanic reaction, strength and permeability of paste made with a given combination of portland cement and extender are dependent on the water:cement ratio (w:c) of the paste. (The term "cement" is understood to include all cementitious materials: portland cement and extender/s.) Strength decreases and permeability increases with increasing w:c. Strength and impermeability of properly compacted hardened concrete are determined largely by those qualities of the hardened cement paste. Thus it is possible to modify the strength and impermeability of concrete by manipulating w:c. Indeed, mix design for strength is based on w:c.

W:Cs typically used for concretes and mortars are:

Conventional concrete	0,45 - 0,80
High-performance concrete	0,25 - 0,40
Sand-cement floor screeds	0,55 - 0,60
Bedding mortar for masonry; cement plaster	1,00 - 1,15

1.8 The cement industry as a contributor of greenhouse gas emissions

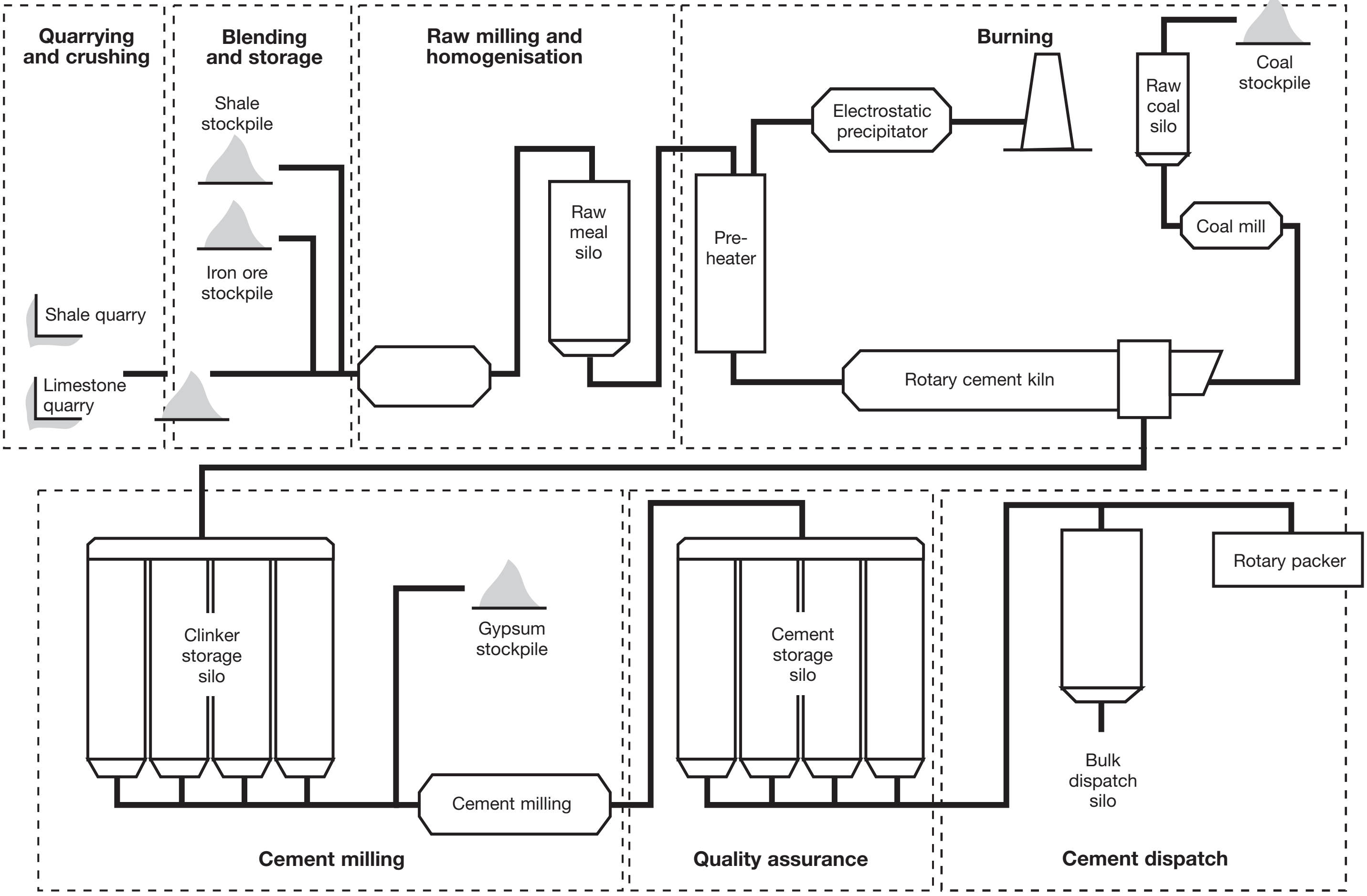
In recent years the international cement industry has come under scrutiny as a significant contributor of CO_2 to the atmosphere, both through the dissociation of calcium carbonate and through the combustion of hydrocarbon fuels in the kiln. The Working Group Cement under the World Business Council for Sustainable Development has committed its members – which include major multinational cement companies – to reducing emissions both through improved production efficiencies and by dilution of clinker content in the products supplied to the market through the judicious use of cement extenders. The commitments made by the working group have impacted on South Africa with the result that there has been a trend away from "pure" cements and a growth in the use of blended cements, with a resultant reduction in specific CO_2 emission. This was reported in the ACMP publication, *Achieving a sustainable South African cement industry.* (See under Further Reading.)

1.9 Manufacture and composition

1.9.1 Portland cement

The principal raw materials used in the manufacture of portland cement are oxides of lime, silica, aluminium and iron.

Lime, ie calcium oxide CaO, is not found in nature, but is produced by heating calcium carbonate (limestone or calcite): $CaCO_3 \rightarrow CaO + CO_2$



Based on a figure in SABS Bulletin, Dec. 1998

Figure 1.1: Manufacture of portland cement

Table 1.6: Composition of portland cement clinker

Oxide	% by mass in cement
CaO	63 - 69
SiO_2	19 - 24
Al_2O_3	4 - 7
Fe_2O_3	1 - 6
MgO	0,5 - 3,6
Na_2O + 0,658 K_2O	0,2 - 0,8

Silica, ie silicon dioxide SiO_2, is found in many natural rocks and minerals including clays. Alumina, ie aluminium oxide Al_2O_3, is found in clay minerals. Iron, ie ferric oxide, Fe_2O_3, occurs in most minerals and especially in clays. It is therefore convenient for the cement manufacturer to obtain silica, alumina and ferric oxide from clay or shale.

These four oxides may be divided into two categories according to their main functions. Lime and silica form compounds that will hydrate to form cementing compounds; alumina and ferric oxide act as fluxing agents in the process of clinker burning.

The cement manufacturing process is shown diagrammatically in Figure 1.1.

Table 1.7: Compound composition of South African CEM I cements

Compound	Formula	Abbreviation	% by mass in cement
Tricalcium silicate	$3CaO.SiO_2$	C_3S	60 - 73
Dicalcium silicate	$2CaO.SiO_2$	C_2S	8 - 30
Tricalcium aluminate	$3CaO.Al_2O_3$	C_3A	5 - 12
Tetracalcium aluminoferrite	$4CaO.Al_2O_3.Fe_2O_3$	C_4AF	8 - 16
Magnesia	MgO	M	1,9 - 3,2
Gypsum	Raw material	-	4,4 - 6,7
Free lime	CaO	-	0,2 - 2,5

In the kiln, calcium carbonate is first converted to calcium oxide at temperatures between 800ºC and 1 000ºC. The materials then flow to a hotter part of the kiln where, at temperatures of about 1 400ºC to 1 450ºC, the blend of four oxides is converted to cement clinker. The limestone or calcite and the clay or shale used for the manufacture of cement do not consist only of the four oxides required. They also contain some magnesia and alkalis which become part of the clinker. The ash from the coal or other fuels used in the kiln is also taken up into the clinker.

The composition, in terms of oxides, of South African portland cement clinker is shown in Table 1.6.

Four main compounds are present in cement clinker: tricalcium silicate, dicalcium silicate, tricalcium aluminate and tetracalcium aluminoferrite. Gypsum is added to the clinker at the milling stage in order to retard the rate of hydration of the cement. The compound composition of CEM I cement is shown in Table 1.7.

In addition to the compounds shown in the table, small amounts of alkalis, in the form of sodium and potassium oxides, are also present in the cement.

Tricalcium silicate is the compound in portland cement that distinguishes portland cement from other cements made with lime and silica. In the manufacture of PC, the aim is to produce a clinker which contains a large proportion of tricalcium silicate and only a negligible amount of free lime. This is possible only if the raw materials, ie oxides, are finely ground and combined in the correct proportions, and the kiln is operated at the correct temperature.[1.11]

1.9.2 Ground granulated blastfurnace slag (GGBS)

GGBS is a byproduct of the blastfurnace process which is used to produce iron. Blastfurnace slag consists mainly of silica and alumina, derived from the iron ore, and lime or dolomitic lime added as a fluxing agent. If this slag is cooled slowly, it becomes crystalline and insufficiently reactive to function effectively as a cement extender. If the slag is cooled rapidly, it assumes a glassy, and thus reactive, state. The manufacture of GGBS consists of quenching (ie granulating) the hot slag with water, and then milling it to a fine powder.

The chemical composition, in terms of oxides, of South African GGBS is shown in Table 1.8.

If Table 1.8 is compared with the composition of blastfurnace slags from other parts of the world, it can be seen that the magnesia (MgO) content of South African slag is relatively high. The reason for this difference is the type of limestone used for fluxing: dolomitic limestone in South Africa and ordinary limestone elsewhere. Research,[1.12, 1.13] supported by decades of experience in the use of South African GGBS, has shown that the high MgO content is not detrimental.

Table 1.8: Chemical composition of South African GGBS

Oxide	% by mass in GGBS
SiO_2	34 - 40
CaO	32 - 37
Al_2O_3	11 - 16
MgO	10 - 13
FeO	0,3 - 0,6
MnO	0,7 - 1,2
K_2O	0,8 - 1,3
S	1,0 - 1,7
TiO_2	0,7 - 1,4

More recently the major iron producing company, ArcelorMittal, has constructed some of its more modern smelters as Corex plants. The slag produced by such plants is reported to be more reactive than that produced in the conventional blastfurnaces mentioned above. In 2008 only one such plant, ie that situated in Saldanha Bay, was operating in South Africa. The slag produced by this smelter was mainly being used as a cement extender in the Western Cape. The typical chemical composition of this Corex slag falls with the ranges shown in Table 1.8.

1.9.3 Fly ash (FA)

FA is extracted by electrostatic precipitators or bag filters from the flue gases of furnaces fired with pulverised coal. After extraction, FA is classified into fine and coarse fractions. The fine fraction (about 10% retained on the 45-µm sieve) is used as an extender and the coarse fraction is used in other applications. Some cement manufacturers intergrind unclassified ash (which includes both coarse and fine fractions) when making "blended" cements.

The chemical composition of South African FA is shown in Table 1.9. Because the CaO content is less than 10%, South African FA is type V (see Table 1.1).

Table 1.9: Chemical composition of South African FA (ex Matla, Lethabo and Kendal)

Oxide	% by mass
SiO_2	48 - 55
Al_2O_3	28 - 34
CaO	4 - 7
Fe_2O_3	2 - 4
MgO	1 - 2
$Na_2O + 0{,}658\ K_2O$	1 - 2

Table 1.10: Chemical composition of CSF

Oxide	% by mass
SiO_2	92 - 96
Al_2O_3	1,0 - 1,5
Fe_2O_3	1,0 – 1,6
CaO	0,3 - 0,6
MgO	0,6 – 0,8
K_2O	1,2 – 2,0
H_2O	0,4 - 0,8

1.9.4 Condensed silica fume (CSF)

CSF is a byproduct of the ferrosilicon smelting process. CSF is the condensed vapour from the process.

The chemical composition of CSF is shown in Table 1.10. In addition to the oxides shown in the table, CSF contains a small amount (about 1,4%) of carbon.

CSF is extremely fine. The specific surface is about 20 000 m^2/kg and average particle size is 0,15 μm. Because of this fineness, particles are strongly attracted to each other and cannot slide or roll over each other to achieve a dense packing. CSF therefore has a very low bulk density (about 200 kg/m^3) and consequently is difficult to handle and transport. In order to make the material more manageable, CSF is "densified" by means of electrostatic forces that cause particles to agglomerate into small pellets about 0,5 mm in diameter. In the densified state, bulk density is about 600 kg/m^3.

Comparing the compositions of portland cement and cement extenders

The principal oxides in portland cement are lime, silica and alumina. These oxides are found also in cement extenders. Figure 1.2 compares the proportions of lime, silica and alumina in portland cement, GGBS, FA and CSF. In general, reactivity is a function of lime content. Thus reactivity increases towards the right-hand side of Figure 1.2.

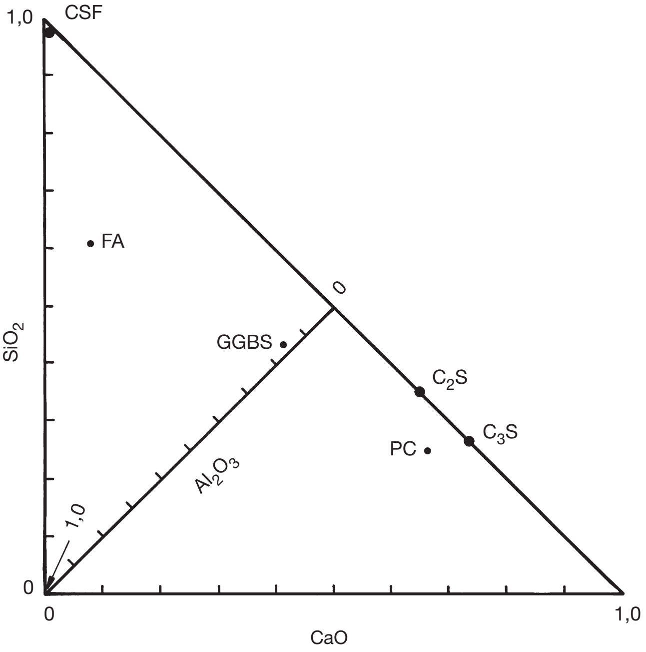


Figure 1.2: Proportions of lime, silica and alumina in portland cement, di- and tricalcium silicate, GGBS, FA and CSF

1.9.5 Limestone

The SANS standards for both Common and Masonry cements permit the incorporation of limestone with specific properties as a component of certain cement types. However, the limestone used in this way does not participate in any of the hydration reactions and it is thought that the limestone acts as a fine filler and reaction nucleation point.

1.10 Cementing reactions and reaction products

Portland cement, GGBS, FA and CSF are all obtained in fine powder form. When appropriately blended and mixed with water, reactions take place to produce cementing compounds. The term hydration is used to describe reactions in which water is chemically combined.

In this section, cementing reactions are discussed in simplified terms.

The following abbreviations are used:

C = CaO, calcium oxide
S = SiO_2, silica
A = Al_2O_3, alumina
F = Fe_2O_3, ferric oxide
H = H_2O, water

Therefore

C_3S = tricalcium silicate
C_2S = dicalcium silicate
C_3A = tricalcium aluminate
C_4AF = tetracalcium aluminoferrite
CH = $Ca(OH)_2$, calcium hydroxide

1.10.1 Portland cement

The hydration of portland cement is described by the following reactions:[1.14]

$2C_3S + 6H \rightarrow C_3S_2H_3 + 3CH$ .... (1)
$2C_2S + 4H \rightarrow C_3S_2H_3 + CH$ .... (2)
$C_3A + CH + 12H \rightarrow C_4AH_{13}$ .... (3)
$C_4AF + 4CH + 22H \rightarrow C_4AH_{13} + C_4FH_{13}$ .... (4)

In these reactions, CH for reactions (3) and (4) is provided by reactions (1) and (2). Reaction (3) has intermediate stages involving sulphate from the gypsum but these are not shown.

Reactions (1) to (4) are all exothermic. The quantity of heat evolved during complete hydration of portland cement is about 500 kJ/kg.[1.15]

Reaction products have the following characteristics: $C_3S_2H_3$, Calcium silicate hydrate, is in the form of very fine needles and plates, and contributes most of the strength of the hardened cement paste (HCP). CH, calcium hydroxide, is in the form of relatively large crystals which do not contribute to the strength of the HCP. Because CH is readily soluble, any pore water in the paste is a saturated solution of CH. C_4AH_{13} and C_4FH_{13} do not contribute significantly to the strength of the HCP.

Assuming average composition of portland cement and using atomic masses, reactions (1) to (4) can be used to calculate that, for complete hydration, 1 000 g of portland cement will combine chemically with about 280 g of water to produce 615 g of $C_3S_2H_3$ and 255 g of CH.

1.10.2 Ground granulated blastfurnace slag

Provided that GGBS is alkali activated, it reacts with and in water to form cementing compounds. The reactions of GGBS when activated by portland cement are complex and not fully understood. Czernin,[1.16] states that, on the whole, the hydration products of GGBS are similar to those of portland cement except that no CH is liberated. Mantel[1.17] reports that, according to Bakker, surplus acid components are set free during hydration and these react with CH to form additional calcium silicate hydrate and calcium aluminate products.

The above may be summarised, in simplified reaction form:

$3\,(C + S) + 3H \rightarrow C_3S_2H_3 + S$ ....(5)

$(a \times S) + (b \times CH) + \text{Other} \rightarrow C_3S_2H_3 + \text{CA products}$ .....(6)

In reaction (5), (C + S) denotes GGBS. In reaction (6), a and b are numbers with values required for equilibrium; S is derived from reaction (5); CH is derived from reactions (1) and (2) above.

1.10.3 Fly ash and condensed silica fume

FA and CSF are pozzolans. They form cementing compounds in water by reacting with calcium hydroxide and do not hydrate – ie react with water – as do PC and GGBS. The cementing compounds formed by pozzolans are similar to those formed by the hydration of portland cement: the principal compound is $C_3S_2H_3$.[1.18]

A simplified reaction is :

$2S + 3CH \rightarrow C_3S_2H_3$ ....(7)

The CH on the left-hand side of reaction (7) is derived in practice from the hydration of PC, ie reactions (1) and (2) above.

1.10.4 Limestone

Limestone used as a component in a cement blend does not participate in any of the hydration reactions and it is thought that the limestone acts as a fine filler and reaction nucleation point.

1.11 Setting and hardening of portland cement

Provided that the ratio of water to cement is not too low, freshly mixed paste is plastic and workable. With time, the paste becomes stiff and unworkable and gradually changes to a rigid solid which gains strength with time.

These changes in state are caused by hydration of the cement. The rate of hydration is however not uniform. As soon as cement is mixed with water, a rapid reaction begins. Cement particles start to dissolve, tricalcium silicate is hydrated to form gel and release calcium hydroxide, and the mixing water becomes a supersaturated solution of calcium hydroxide.

The mixing water leaches alkalis from the cement grains while the aluminates pass into solution, react with dissolved gypsum, and precipitate as insoluble calcium sulphoaluminate (ettringite).[1.19] Ettringite, it is believed, attaches itself to the aluminate and thus slows down the access of water to the aluminate and so prevents the too-rapid setting of the cement.[1.19] With time, further reactions eliminate this retarding effect.

Because reactions, and the resultant changes in physical state, do not proceed at a uniform rate, it is convenient to divide the process into stages:[1.20]

During the *dormant* stage, the paste remains plastic and workable.

During the *setting* stage, the paste is stiff and unworkable.

During the *hardening* stage, the paste is a rigid solid which gains strength with time.

The transition between dormant and setting stages is known as the *initial set.* The transition between setting and hardening stages is known as the *final set.*

1.12 Physical aspects of cement paste

In the following, cement paste made with portland cement only will be discussed first. Hardened cement paste (HCP) made with blends of portland cement and a cement extender will then be dealt with.

1.12.1 Portland cement paste

Fresh paste

Fresh paste consists of cement particles dispersed throughout water-filled space.* The average distance between adjacent cement particles depends on the fineness of the cement and the ratio of water to cement in the paste. At the w:c ratios normally used, cement particles are not in contact but are separated by films of water. In fresh paste, therefore, water forms a continuous phase.

In the fresh state, cement particles, because of their greater density, tend to settle and water tends to migrate upwards. This movement of water is called bleeding, and a layer of almost clear water on the top of the paste can result from it.

The transition between fresh paste and hardened paste is discussed in section 1.11.

Hardened paste – bulk characteristics

The structure of hardened cement paste cured at normal temperatures has the following characteristics:[1.23]

It consists predominantly of a gel made up of particles colloidal in size and probably of platy structure similar to that of some types of clay. Embedded in the gel are comparatively large crystals of calcium hydroxide and residues of unhydrated cement. The gel has a large surface area, of the order of 200 000 m^2/kg – about three orders of magnitude greater than the cement particles from which it is derived.

The gel, even of the densest possible and completely hydrated cement paste, is characteristically about two thirds solid and one third pores, but has a permeability considerably lower than that of most natural rocks. The low permeability of the highly porous gel is attributable to the smallness of the gel pores. A reasonable estimate of the average distance between the solid surfaces of the gel particles is roughly 2 nm, or about five times the diameter of a water molecule. In a hardened cement paste there are not only gel pores but capillaries which, though some two orders of magnitude larger than the gel pores,[1.22] are still submicroscopic. These capillaries are the residue of the original water-filled spaces in the fresh paste. Initially such capillaries form a continuous interconnected system, but as hydration proceeds they tend to become blocked by the formation of cement gel. There is evidence that in mature pastes of original w:c ratio below about 0,7, the capillaries tend to lose their continuity and take the form of capillary cavities.

In order to quantify the microstructure of cement paste, it is necessary to know the ratio of hydrated volume to original volume of cement, and the porosity of the gel.

References 1.23 and 1.24 give formulae for calculating the volumetric ratios of cement paste at normal temperatures. These ratios are specific to each cement and are functions of compound composition and particle relative density. In reference 1.25, Powers found that a unit of volume of cement hydrates to a volume of 2,23 and that the porosity of gel is 0,33. Addis,[1.26] basing calculations on the average composition of five South African portland cements made without additions, found these values to be 2,34 and 0,34 respectively. Because the aim of this discussion is to describe the microstructure in approximate terms, and because the basis of such calculations is so inexact, a simplified approach seems to be justified. Such an approach is taken by Czernin[1.27] who states that at a w:c ratio of 0,4, the overall volume of hydrate exactly fills the space originally occupied by cement and water. This means that unit volume of cement hydrates to a volume of about 2,25.

The following calculations are based on constant values:

- A unit of volume of cement hydrates to a volume of 2,25
- The mass of water which combines chemically with a unit of mass of cement is 0,29
- Gel porosity is 0,34
- Specific volume of cement is 0,32 $m\ell/g$
- Specific volume of water is 1,0 $m\ell/g$

Microstructure of HPC will be investigated for w:c equal to 0,4, greater than 0,4 and less than 0,4. (Strictly speaking, the amount of water used to calculate w:c should be that left in the paste at the completion of the bleeding process: bleed-water is lost from the paste. The true w:c is therefore less than w:c at the time of mixing. For the sake of simplicity, the difference between these two values of w:c is ignored in this investigation.)

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* Particles do not disperse uniformly but tend to form flocs in which spacing is relatively close, thus leaving water-filled spaces devoid of particles.[1.21] This phenomenon has an important effect on the workability of pastes and concretes. Plasticising admixtures play a role in dispersing these flocs and improve the effectiveness of distribution of cement particles through the mix. See Chapter 5.

W:C equal to 0,4

As discussed above, if all the cement in such a paste hydrates it will exactly fill the space available. Capillary porosity is therefore zero. Phases before the start of hydration and at complete hydration are shown graphically in Figure 1.3.

If, for example, hydration is 80% complete, 20% of the original cement will he unhydrated and the volume of capillary pores will be equal to 20% of the original volume of water; capillary porosity of the paste will be:

$$\frac{0{,}2 \times 1{,}25}{2{,}25} = 0{,}11 \text{ or } 11\%$$

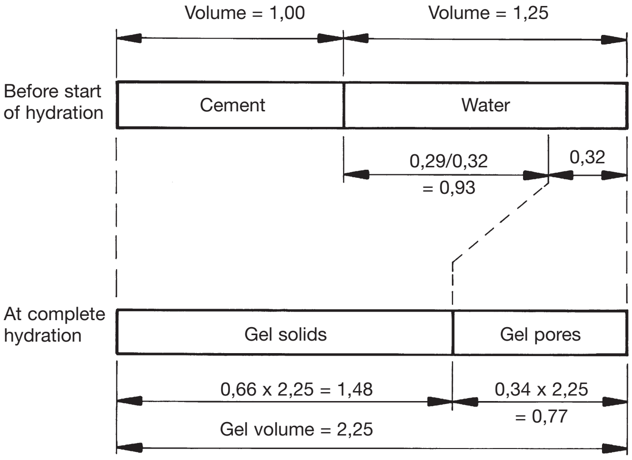

Figure 1.3: Absolute volumes of phases of cement paste for w:c = 0,4 by mass

W:C greater than 0,4

Such pastes, after all the cement has hydrated, are characterised by the presence of capillary pores. Phases before the start of hydration and at complete hydration are shown graphically in Figure 1.4 Capillary porosity (P_c) at full hydration depends on water:cement ratio:

$$P_c = \frac{\left(\frac{W}{C}\right) - 0{,}4}{\left(\frac{W}{C}\right) + 0{,}319}$$

P_c as a function of w:c ratio, is shown graphically in Figure 1.5.

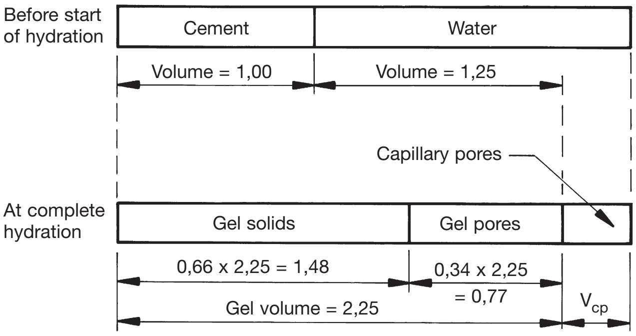

Figure 1.4: Absolute volumes of phases of cement paste for w:c > 0,4 by mass

W:C less than 0,4

Such pastes, when further hydration is constrained by lack of space, contain unhydrated cement. Phases before the start of hydration, and when no further hydration is possible, are shown graphically in Figure 1.6. The volume fraction of unhydratable cement (V_{uc}) in this case is a function of water:cement ratio:

$$V_{uc} = \frac{0{,}319 - 0{,}798 \left(\frac{W}{C}\right)}{0{,}319 + \left(\frac{W}{C}\right)}$$

V_{uc} as a function of water:cement ratio, is shown graphically in Figure 1.5.

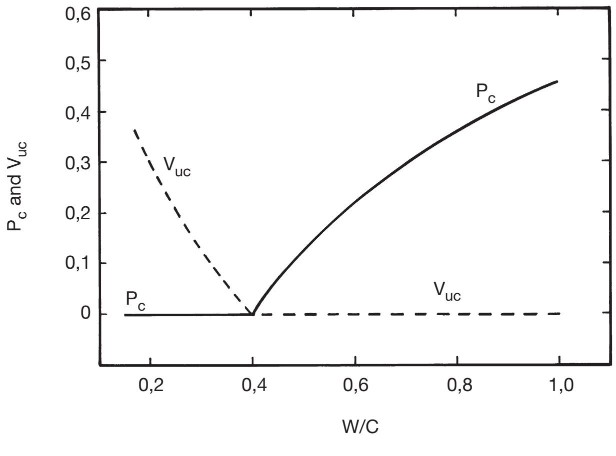

Figure 1.5: Capillary porosity (P_c) and volume fraction of unhydratable cement (V_{uc}) versus w:c

Hardened paste – paste-aggregate interface[1.28]

In freshly compacted concrete, water films form around aggregate particles. This effect is more pronounced around coarse aggregate particles where film thickness may be increased by the accumulation of bleed-water. The presence of a water film increases water:cement ratio of the paste close to the aggregate surface compared with the bulk paste. In the hardened state the interface zone is characterised by high capillary porosity, relatively large calcium hydroxide crystals, ettringite crystals and very little calcium silicate hydrate. Paste-aggregate interfaces are therefore considerably weaker than the bulk hardened cement paste.

Self-desiccation of cement paste

If a paste sample is sealed immediately after mixing, it is possible for hydration to be stifled by self-desiccation if the w:c ratio is less than a critical value.

This critical value may be calculated as follows.

From Figure 1.7

Let unit volume of cement be 1ℓ.

Mass of chemically bound water (see *Hardened paste – bulk characteristics*)

$$= 0{,}29 \times \frac{1{,}00}{0{,}32} \text{ kg}$$

$$= 0{,}906 \text{ kg}$$

Volume of water in (saturated) gel pores
$= 0{,}770\ \ell = 2{,}25 \times 0{,}34\ \ell$

Because of adsorption pressures, the gel-pore water is "compressed" by 10 % and therefore has a specific volume of 0,9 $m\ell/g$.[1.30]

The mass of water in gel pores is therefore $= \frac{0{,}770}{0{,}9} = 0{,}855$ kg

Total mass of water chemically bound and in gel pores therefore = 0,906 + 0,855 kg = 1,761 kg.

Consider the case where the paste sample is sealed after mixing, ie no water is allowed to escape or is added. At a critical w:c ratio, there will be sufficient water to hydrate the cement and fill the gel pores but the capillary pores will be empty of water. If the w:c ratio is higher than this critical ratio, capillary pores will be partially filled. If the w:c ratio is lower than this critical ratio, there will be insufficient water in the system to hydrate all of the cement; hydration will be stopped by self-desiccation of the paste.

Water contents at the critical w:c are shown in Figure 1.7.
At the critical w:c ratio, mass of mixing water = mass of chemically bound water plus mass of water in gel pores = 1,761 kg

But mass of cement $= \frac{1{,}00}{0{,}32} = 3{,}125$ kg

Therefore critical w:c $= \frac{1{,}761}{3{,}125} = 0{,}56$

If w:c < 0,56, water must be fed into the hydrating paste to prevent self-desiccation and the consequent stifling of hydration. If w:c > 0,56, sealed pastes contain sufficient water for full hydration.

The phenomenon of self-desiccation has practical implications in the curing of concrete.

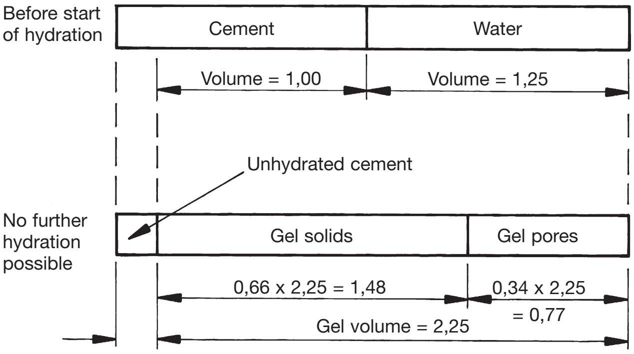

Figure 1.6: Absolute volumes of phases of cement paste for W/C < 0,4 by mass

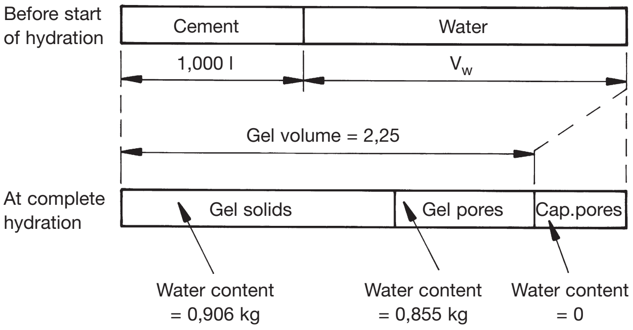

Figure 1.7: Water content of gel solids, gel pores and capillary pores where w:c is critical for self-desiccation

1.12.2 HCP made with portland cement and cement extenders

Compared with the work done by Powers and his colleagues[1.22, 1.24, 1.25, 1.29] on the nature of HCP made with portland cement only, little research has been done on pastes made with blends of PC and extenders. The reaction products of such blends are however so similar to those of portland cement, that it seems reasonable to expect physical characteristics of both types of pastes to be similar. Extenders can however significantly affect the homogeneity of the bulk paste, the paste-aggregate interface and the phenomenon of self-desiccation. These aspects are discussed below.

Homogeneity of the bulk paste

In fresh paste, well-dispersed fine particles that are less reactive than cement particles (ie which do not decompose in water as quickly as similarly fine cement particles) provide numerous nucleation sites for the precipitation of cement hydration products.[1.29] This phenomenon is known as the "fine-filler effect" and the result is a more homogeneous microstructure of the hardened cement paste with consequent improvement of strength and impermeability.

Fine particles in GGBS, FA, CSF and ground limestone may act as fine fillers. Because of its extreme average fineness, CSF is probably the most effective fine filler.

Paste-aggregate interface

The presence of well-dispersed fine particles in fresh paste serves to segment channels of water flow and so reduce bleeding. This has the result of reducing film thickness of water on the undersides of coarse aggregate particles. Fine particles present in the water of the interface zone also create a fine-filler effect in this zone. (See *Homogeneity of the bulk paste* above.) The result is an improvement of the strength and impermeability of the paste-aggregate interface.[1.30]

Self-desiccation

Pozzolanic extenders, eg FA and CSF, do not hydrate. Instead they react with calcium hydroxide (which is hydrated calcium oxide) to form cementing compounds. In a paste made from a blend of portland cement and pozzolan, only the portland cement consumes water. The critical w:c for self-desiccation of such a paste is therefore less than that of a portland cement paste. Taking a 70/30 blend of portland cement and fly ash as an example, the critical ratio for self-desiccation would be 0,39 compared with 0,56 for portland cement only.

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Chapter 2
Other cements

Bryan Perrie

2.1 Introduction

This chapter deals with modified portland cements, other hydraulic cements and chemical cements. The aim is to cover several of these cements that have been and are being used around the world, although only a few of them may be available in South Africa.

The use, applications, characteristics, performance, and methods of manufacture of each cement are discussed as appropriate. Suggested further reading is included where relevant.

The list of cements covered is by no means comprehensive, but includes the majority of cements used in civil and building applications, whether for new construction or as a repair material.

2.2 Modified portland cements

2.2.1 Air-entraining cement

This type of cement is produced by intergrinding an air-entrainer into the cement. It is made and sold mainly in the USA.[2.1]

The improved workability and durability of air-entrained portland cement concrete are well known. Air-entraining cement may have advantages for the concrete products industry, where the conditions of production are rigidly controlled and remain relatively unchanged. However, in ordinary concreting operations, where the grading and nature of the aggregate, the concrete mix proportions, the mixer type and placing temperatures vary from job to job, it seems preferable to adjust the amount of air entrainment to the particular conditions.[2.2]

Air-entrained cements are also used in road construction in regions where the hardened concrete may be exposed to freezing temperatures.[2.3]

2.2.2 Anti-bacterial cement

Cement-based products such as concrete, mortar and plasters are often subject to attack by micro-organisms such as moulds, bacteria, yeast, etc. Anti-bacterial cement contains small quantities (0,005 to 1,0% by mass of the cement) of an organic fungicide. The effect on various organisms of one such fungicide, tributyl tin acetate, interground into cement, is discussed in references 2.4 and 2.5.

2.2.3 Coloured cement

This is usually portland cement to which pigments have been added. Strong pigments can be added to ordinary portland cement in quantities of up to 10% by mass. For lighter colours, white portland cement has to be used as the base and the resulting cement is consequently more expensive. The pigment is best added by the manufacturer during the process of grinding the cement clinker.[2.6]

2.2.4 Expansive cement

Expansive cements are used in shrinkage-compensating concretes. These concretes find application in concrete structures, especially floors and slabs, where drying-shrinkage cracking is undesirable. When properly restrained by reinforcement or other means, shrinkage-compensating concretes will expand by an amount equal to the expected drying shrinkage. Because of the restraint, a compressive stress is induced in the concrete and subsequent drying will reduce this stress rather than cause tensile stresses and cracking to develop.

According to Field,[2.7] expansive-cement concretes are divided into two categories depending on the amount of prestress they develop:

- In *shrinkage-compensating* concrete, expansion, if restrained, induces compressive stresses which approximately offset any tensile stresses in the concrete induced by drying. The compressive stresses induced in such concrete are in the region of 0,17 to 0,3 MPa.
- In *self-stressing* concrete, expansion, if restrained, induces compressive stresses high enough to result in significant compressive stresses in the concrete after drying shrinkage and creep have occurred. These stresses would be about 0,7 to 7 MPa.

Recommended practice for the use of these cements is covered by ACI 223-98.[2.8] Malisch[2.9] states: "To ensure adequate expansions and satisfactory results when expansive cements are used, good quality control and on-site supervision are necessary ... Moist curing is doubly important, because without it, failure to achieve desired expansions as well as desired strength will result."

The physical characteristics of cured shrinkage-compensating concrete are usually of the same order as for other types of concrete but some concern has been expressed regarding resistance to sulphate attack.

In the USA and Russia, expansive cements are produced as an entity, whereas in Japan they are produced by adding expansive admixtures to ordinary portland cement.[2.10]

In America three types of expansive cements are used namely Types K, M and S:

Type K is a mixture of portland cement, anhydrous tetracalcium trialuminate sulphate (C_4A_3S), calcium sulphate ($CaSO_4$), and lime (CaO). The C_4A_3S is a constituent of a separately burned clinker that is interground with portland cement or alternatively it may be formed simultaneously with the portland cement clinker compounds during burning.

Type M is an interground or blended mixture of portland cement, calcium-aluminate cement (CA and $C_{12}A_7$) and calcium sulphate.

Type S is a portland cement containing a high tricalcium aluminate content (C_3A) and a greater quantity of calcium sulphate than that usually found in portland cement.

In Japan another type of expansive cement, not based on sulphate and owing its expansive properties to the hydration of free lime, is used.[2.10, 2.11]

2.2.5 Extra-rapid-hardening cement

There are numerous potential uses for cement that achieves significant strength in a matter of hours. These include the repair of concrete roads and bridge deck patching and resurfacing, where traffic can be allowed to use the treated surfaces soon after renovation work has been completed. Other possible appplications include sprayed concrete (shotcrete), rapid placement of concrete under water, rapid manufacture of components and cement-fibre composites.[2.12]

Extra-rapid-hardening cement is obtained by intergrinding calcium chloride with rapid-hardening portland cement. The normal addition of calcium chloride is 2% (of commercial 70% $CaCl_2$) by mass of the rapid-hardening cement. In addition to accelerating the strength gain, the addition of the calcium chloride imparts quick-setting properties, and extra-rapid-hardening cement concrete should be placed and fully compacted within 20 minutes of mixing.[2.13]

Because of the acceleration of the process, heat is evolved more quickly and the cement is more suitable than ordinary portland cement for use in cold weather.[2.14]

Orchard[2.14] claims that only a small amount of initial corrosion of the reinforcement occurs but that the effect does not appear to be progressive or to decrease bond strength. Osborne and Smith[2.12] however recommend that extra-rapid-hardening cement should not be used in either reinforced or prestressed concrete. This view is shared by Monfore and Verbeck.[2.15]

2.2.6 Hydrophobic cement

The tendency of cement to deteriorate and form lumps during storage due to the absorption of moisture is well known. Certain substances, if ground with ordinary portland cement during manufacture, have the property of forming a water-repellant film around each cement grain. According to Nurse,[2.16] stearic acid, aleic acid, lauric acid and pentachlorophenol can be used for this purpose. The most effective is aleic acid which should be added at about 0,3% by mass of cement.

The merits claimed for the treatment include:

- The addition acts as a grinding aid in manufacture, thus producing a finer cement for the same power consumption.
- The water-repellant film formed around each grain markedly reduces the rate of deterioration during storage or during transport under unfavourable conditions. The film is broken down when aggregate is added and the materials are mixed with water.
- During mixing, the additive is said to plasticise the cement without excessive air entrainment, thus permitting a reduction in the water content of the concrete.
- The hardened concrete is said to have reduced permeability and water absorption.

2.2.7 Low-heat cement

To avoid the problem of thermal cracking in large masses of concrete due to contraction caused by the dissipation of heat, it is necessary to dissipate the heat as it is formed, or, if this cannot be done quickly enough, to reduce or delay the generation of heat. Low-heat portland cement is formulated to both reduce and delay the generation of heat.

The delay in the generation of heat gives a longer time for the dissipation of the heat and also helps to reduce cracking by giving time for creep and plastic yield of the concrete to relieve the stresses caused by thermal expansion and the subsequent contraction.

The slow evolution of heat in low-heat portland cement is the result of increasing the proportion of C_2S, reducing the proportion of C_3S and restricting the amount of C_3A. This is achieved by restricting the amount of calcium and increasing the silicates present in the raw materials. Although the C_3A generates heat at a higher rate it may not contribute as much heat as the silicates due to its lower proportion.[2.17]

A reduction in temperature will retard the rate of hydration and so further restrict the rate of heat generation. A feature of low-heat cement is therefore a low rate of gain of strength.

2.2.8 Oilwell cement

This is also known as well-drilling or deep-drilling cement. Cement is used in the drilling of oil wells to fill the space between the steel lining tube and wall of the well, and to grout up porous strata to prevent water or gas from gaining access to the oil-bearing strata. Oil wells are drilled to great depths at which temperatures and pressures are high.

Oilwell cement must be able to withstand being pumped for up to three hours when subjected to pressures up to 150 MPa and temperatures up to 200ºC. In addition it must gain strength quickly after setting.[2.18]

This type of cement differs in composition from ordinary portland cement in that the tricalcium aluminate compound (C_3A) content is low, it is more coarsely ground and may contain special additives to produce a low-viscosity, slow-setting slurry.[2.19]

The most widely used standards for oilwell cements are the recent ISO Standards, ISO 10426-1 and 2.[2.20, 2.21] These standards cover eight classes and three grades of oilwell cements.

2.2.9 Regulated-set cement (jet cement)

According to Uchikawa and Kohno,[2.22] applications for this cement are:

- Urgent repairs where construction and working time is limited
- Construction at low temperatures
- Precast manufacture for quick demoulding and rapid turnover
- Binding materials for foundry moulds
- Shotcreting of tunnel linings
- Binding materials for the solidification and treatment of heavy toxic metals
- Grouting materials

The cement is manufactured either by producing a clinker containing 1 to 30% by mass of a calcium haloaluminate having the formula $11CaO.7A1_2O_3.CaX_2(C_{11}A_7CaX_2)$, in which X is a halogen, or by grinding together portland cement clinker and a clinker containing a calcium haloaluminate. The halogen may be chlorine, fluorine, bromine or iodine although fluorine is preferable. The sulphate content of these cements is 1 to 12% expressed as SO_3 and added as calcium sulphate. The clinker is formed by heating a raw meal rich in alumina in the presence of CaF_2 at 1 275 to 1 400ºC or in the presence of $CaC1_2$ at 1 380 to 1 510ºC. The set can be regulated in the range 2 to 30 minutes by use of a set retarder such as a solution of citric acid in water.[2.23]

Uchikawa and Kohno[2.24] claim the following characteristics for regulated-set cement:

- Setting time can be controlled. Setting time at temperatures above 10ºC is very short but can be controlled by adding the required amount of retarder (citric acid).

- Ultra-rapid hardening and super-high early strength, eg 10 MPa after two hours, are possible. Regulated-set cement is called a "one hour" cement as the fluor-aluminate strength development is essentially complete in the first hour.

- The cement shows stable strength development extending over a long time and has high ultimate strength. Calcium silicate hydrate is formed by the hydration of alite. There is no loss of strength with age by the conversion of hexagonal calcium aluminate hydrate to the cubic form as observed in aluminous cements (high-alumina cements).

- The development of strength at low temperatures is excellent. At 5ºC mortars can reach 9 MPa at six hours and 30 MPa at one day.

- Drying shrinkage is small because expansive materials such as ettringite are formed in the early stages of hydration so that drying shrinkage at later ages is negligible.

- Water permeability is low. The water permeability coefficient is about an order of magnitude lower than ordinary portland cement at the same stage.

2.2.10 Trief cement

Trief cement is the same as portland blastfurnace cement except that the granulated blastfurnace slag is ground wet and separately from the cement. It is claimed that early-age strengths equal to those of ordinary portland cement are obtained with a lower heat of hydration.

The granulated blastfurnace slag is ground in a ball mill at a water content of about 30% and is mixed, just prior to use, with the portland cement in the ratio of seven parts blastfurnace slurry to three parts of cement. Trief-cement concrete is made and placed in the usual way.[2.25]

2.2.11 Ultra-rapid-hardening cement

These cements are used in applications similar to those of extra-rapid-hardening (see section 2.2.5).

Ultra-rapid-hardening cement is produced by grinding portland cement extremely fine and increasing gypsum content in order to control setting. Finenesses between 500 and 900 m^2/kg have been used.[2.26, 2.27, 2.28] An advantage over extra-rapid-hardening cement is that ultra-rapid-hardening cement can be used in both prestressed and reinforced concrete.

Orchard,[2.29] Bensted[2.30] and Kollek[2.31] comment on the effect of fineness on heat of hydration, long-term strength gain, workability, etc, and indicate 24-hour strengths four to seven times higher than ordinary portland cement and 28-day strengths 50 to 100% higher.

Both Orchard[2.29] and Bensted[2.30] warn of high alkali levels (ie Na_2O and K_2O) with this cement.

2.2.12 Waterproof cement

This consists of ordinary portland cement to which certain waterproofing substances have been added during grinding. These substances include calcium stearate, aluminium stearate, gypsum treated with tannic acid and certain non-saponifiable oils. It is open to question as to whether these cements produce a less permeable concrete.[2.32]

2.2.13 White cement

White cement is essentially portland cement that has been manufactured using carefully selected raw materials and special processes to avoid contamination. The grey colour of normal portland cements is caused primarily by the iron oxide content so this needs to be kept as low as possible for white cement. White cement is used primarily for decorative purposes. Standards for white cement exist in a number of countries.

White calcium aluminate cements are also produced.

2.3 Other hydraulic cements

2.3.1 Calcium aluminate cements

The term high-alumina cement came into use when this type of cement was introduced in the UK after World War I to distinguish it from portland cements which contain much less alumina. Many other aluminous cements, with alumina contents between 50 and 90%, have been developed mainly for refractory applications.

All these types of cements form calcium aluminate hydrates and are best referred to as calcium aluminate cements (CACs). They are manufactured by fusing or melting a mixture of aluminous and calcareous materials (generally bauxite and limestone) and grinding the resulting material to the required fineness.

CACs have many important and useful purposes including:

- Rapid strength development even at low temperatures
- High temperature resistance/refractory performance
- Resistance to a wide range of aggressive chemicals

CAC is also widely used in combination with other minerals and additives including portland cement and calcium sulphate. The properties of such combinations include rapid setting and controlled expansion/shrinkage compensation. Unfortunately, under the influence of heat and moisture, the metastable (hexagonal CAH_{10}) calcium aluminate hydrates transform to the stable (cubic C_3AH_6) hydrates with the accompanying formation of aluminium hydroxide and the liberation of water:

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

This process is known as "conversion". The rate of conversion depends on the temperature to which the concrete is exposed. At normal temperatures conversion may take many years, but at a temperature of 40ºC it may occur within a few months and at 70ºC it may be completed in a matter of hours. Whether it occurs early or late in the life of the concrete, conversion results in a loss of strength, increased porosity and reduced resistance to chemical attack. In particular, resistance to sulphate attack is considerably reduced. HAC concrete is also susceptible to attack by alkalis, which can cause the complete disintegration of converted HAC concrete of high original water:cement ratio.

Oberholster[2.33] issued the following warnings regarding the use of CAC

- CAC should not be used for load-bearing structures where hot and moist conditions occur simultaneously.
- CAC should not be used for prestressed or reinforced concrete because of the danger of corrosion of the reinforcement. This is because HAC releases little or no $Ca(OH)_2$ on hydration to protect the steel.
- CAC should not be used for mass concrete unless the heat development can be controlled.
- CAC should not be used in contact with caustic alkalis or materials that can supply these alkalis.
- The water:cement ratio should be kept low.

The use of CAC for structural purposes has been prohibited by codes of practice in many countries including South Africa.

Lea[2.34] states "Over 80 years of use of CAC concrete have shown that well-made good-quality concrete has a very good track record of field performance. However, it may also be surmised that CAC concrete is more sensitive to misuse than portland cement, due to the temporary high early strengths which may be developed even at high water:cement ratios. Therefore correct use of CAC concrete must be on the basis of converted strengths for structural design and strict quality control of the mix design. To ensure good long-term strength and durability a water:cement ratio below 0,4 and therefore a cement content above 400 kg/m^3 are recommended."

Applications and uses

Rapid hardening

The average time of initial set is two to four hours, the final set occurring about 30 minutes after the initial set. Thereafter the cement gains strength very rapidly and high strengths (30 MPa) are obtainable within 24 hours. All material is therefore suitable for emergency work, eg between tides or between traffic peaks.

Use at low temperatures

Because of the high rate of heat generation, CAC may be used for concreting at low temperatures. It has, for example, considerable advantages in the repair of cold-room and freezer floors. However, the generation of heat may result in evaporation of the mix water required for the formation of the hydrates. Desiccation may be prevented by:

- Limiting the thickness of concrete placed to a maximum of 450 mm.
- Avoiding the mixing or placing of the concrete at temperatures above 30ºC.
- Stripping shuttering as early as possible (generally after four hours) and spraying liberally or soaking with water for 48 hours.

Industrial floors

The following properties make CAC concrete suitable for use in industrial floors and pavements:

- Impact and abrasion resistance
- Chemical and acid resistance
- Temperature tolerance to cold and heat

Pipes and pipe linings

CACs have a history of successful use as linings and coatings for sewage pipes, other wastewater applications and where soft water attack of conventional concrete occurs.

Mines and tunnels

CAC is used in significant quantities in mining and tunnelling for mine packing and rock bolting.

Proprietary products

CACs are widely used in the building and civil engineering industry to formulate a wide range of products including floor levelling compounds, grouts, repair products and mortars. In such cases they are usually part of a complex formulation.

Refractory applications

CAC has excellent refractory properties. When mixed with refractory aggregates, concretes or mortars suitable for use at temperatures of up to 1 800ºC may be obtained. However, a temperature greater than 700ºC is necessary for ceramic bonding in order to make the cement a refractory material.

2.3.2 Geopolymers

Geopolymer chemistry, when associated with portland cement, yields cementitious materials with remarkable properties, such as ultra-rapid usable strength gain, natural sulphate resistance and freeze-thaw resistance. Modern blended geopolymer cements can also be expected to offer long-term durability.[2.35] Most references to this application of mineralogy belong in the category of patent literature. The applications of this new family of mineral binders have been tested in pilot plants, in Europe and in the USA. A good many have reached the industrial stage in the USA and France.[2.36]

Geopolymers consist basically of silico aluminates activated by alkalis.

2.3.3 Hydraulic lime

Hydraulic lime is used in mortars and plasters.

Hydraulic limes are obtained from the burning of limestones that contain a proportion of clay. At the burning temperature of 1 000 to 1 200ºC the reaction products include C_2S and less basic silicates, C_2AS, C_4AF and calcium aluminates. The hydraulic properties can be attributed to the dicalcium silicate and in some degree to the aluminates. There exists an almost continuous series of limes varying from fat limes with a content of alumina and silica below 1 to 2% to eminently hydraulic limes with up to 50% of these constituents. As the content of alumina and silica increases, the rapidity of slaking and the evolution of heat, so characteristic of fat limes, decrease until with the eminently hydraulic limes no appreciable reaction occurs unless they are finely ground.

Fat limes when used as mortars harden slowly only by absorption of carbon dioxide from the air with formation of calcium carbonate. They will not set under water. Hydraulic limes, because of the alumina and silica compounds they contain, harden slowly without carbon dioxide and can be used under water.

2.3.4 Natural cement

Natural cement is manufactured from naturally occurring "cement rocks" that have compositions similar to the artificial mix from which portland cement is manufactured, ie argillaceous limestone. The temperature of calcination is only as high as is necessary to drive off carbon dioxide and water but not to cause sintering. Natural cement is lower in calcium oxide than ordinary portland cements and has to be ground to a very high fineness. It requires an extremely long period for full strength development and for this reason its use in construction is limited.[2.37]

2.3.5 Strontium and barium cements

These are cements in which the calcium has either been completely or partially replaced by either strontium or barium. Barium cement is extremely resistant to sea water; strontium cement can withstand higher temperatures than ordinary cements.

2.3.6 Supersulphated cement (SSC)

SSC is made by grinding a mixture of 80 to 85% granulated slag, 10 to 15% anhydrite or hard-burned gypsum, and approximately 5% portland cement. The product is ground more finely than normal portland cement.[2.38]

Orchard states that SSC can be used in all cases where portland cement is suitable and all the normal concreting rules apply. The mix proportions should be a little richer with SSC, and lean mixes should not be used.[2.39] However, there is some doubt that concrete made with SSC will adequately protect included steel against corrosion.

When ground very finely, SSC shows excellent initial and final strength values, especially high binding strength. In spite of the rapid rate of hardening, which can equal that of portland cement, the heat of reaction is low.[2.40]

SSC has a comparatively high resistance to chemical attack and Orchard[2.41] reports that rich concrete mixes with the cement have given satisfactory service in weak solutions of mineral acids of pH 3,5 and upwards.

Not all blastfurnace slags are suitable for the production of SSC. The alumina content must be at least 15% and a high lime content is desirable.

SSC shows a tendency to surface powdering. This is due to carbon dioxide decomposing the hydration products, especially calcium sulphoaluminate, during early ages. This can be overcome by inhibiting the access of carbon dioxide to the concrete surface at such ages.

Steam curing of SSC is not recommended.

2.4 Chemical cements

2.4.1 Aluminium oxychloride cement

This is used to produce concretes having refractory properties up to 1 500ºC. However during heating, hydrogen chloride, which is corrosive to metals, is released.

2.4.2 Magnesium oxychloride cement (Sorel cement)

This cement is formed by mixing magnesium oxide powder with a solution of magnesium chloride. It has high early strengths and is suitable for use with all aggregates. It is used primarily for flooring because of its elastic properties; disadvantages include dimensional instability and water solubility (releasing corrosive solutions).

2.4.3 Magnesium oxysulphate cement

This is essentially a variant of magnesium oxychloride cement and improves the water resistance. It has been used in the manufacture of low-density insulating panels.

2.4.4 Magnesium phosphate cement or magnesia-ammonium phosphate cement

This cement is formed by reacting magnesium oxide with soluble phosphates. It has high early strength, good water and freeze-thaw resistance and is dimensionally stable. It is used in concretes and mortars for road repairs.

2.4.5 Sulphur

Sulphur concrete consists of elemental sulphur (which is in the molten form while the concrete is in the plastic state), stone and fine aggregates, and contains neither water nor portland cement. Its high compressive strength at early ages (of the order of 35 MPa at eight hours) makes it ideal for small precast units for outdoor use; its good chemical durability makes it an excellent material in industrial plants. However, its low melting point at 119ºC, its vulnerability to combustion and the production of toxic gases, its corrosive effect on reinforcing steel under wet or humid conditions and its brittleness make it unfit for most structural uses. The available published data suggest that concretes made with unmodified sulphur may be stable under thermal cycling but their resistance to repeated cycles of freezing and thawing is open to question. Concretes made with modified sulphur may or may not perform better.[2.42]

2.4.6 Zinc oxychloride cement

This is produced by reacting together zinc oxide and zinc chloride. The hardened cement paste is extremely hard and is not attacked by boiling water or acids, unlike magnesium oxychloride.

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Chapter 3

Aggregates for concrete

Graham Grieve

3.1 Introduction

Aggregates are used in concrete as diluents of cement paste in order to make the concrete dimensionally more stable. Because the cost of aggregates is generally well below that of cementitious materials, aggregates are also used to reduce the cost of concrete. Aggregates in concrete were originally considered to be inert but it is now known that the properties of aggregates can influence the physical properties of hardened concrete. And using aggregate cost as the only criterion for the selection of aggregates may result in the production of more expensive concrete with undesirable properties in the hardened state.

Aggregates, for most of the concrete produced in South Africa, are derived ultimately from solid rock, which is crushed or has been broken down by natural processes. Other, less significant, sources of aggregates are waste products such as slag and crushed building rubble; synthetic compounds such as polystyrene; and heat-treated minerals such as vermiculite and perlite. (The latter two categories are used for low-density concrete.) It is notable that much of the recent published information on research into aggregates has been focused on the use of recycled concrete and building rubble.

This chapter is intended to provide the specifier and user of concrete with a basic understanding of important properties of aggregates as a basis for their selection and use.

The body of this chapter deals with aggregates derived from rock. Important chemical and physical properties of aggregates and their influence on the properties of concrete are dealt with in some detail. The South African specification and sampling and testing are discussed. The geological origin of aggregates is described and aggregates used in various parts of South Africa are discussed. The effect of crushing techniques on aggregate properties is dealt with briefly and a list for further reading is included. Some aggregates derived from waste materials are dealt with in an appendix. (Information on low-density aggregates is given in Chapter 19.)

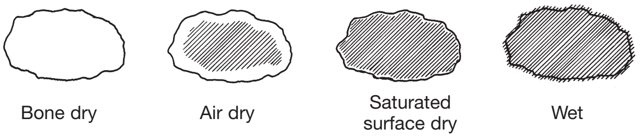

Figure 3.1: Moisture states of aggregates

3.2 Properties of concrete aggregates and their influence on the properties of concrete

This section deals with absorptivity and porosity; dimensional properties; grading; organic materials in aggregates; particle density; particle shape and surface texture; soundness; strength; deleterious chemicals in aggregates; and deleterious rocks and minerals. (Alkali reactivity is dealt with in Chapter 10.)

3.2.1 Absorptivity and porosity

No aggregate particles are entirely free of internal pores or voids. Porosity has an influence on the particle density of the aggregate, while the extent to which pores or voids can be filled with liquid determines the absorption of the aggregate. Figure 3.1 shows the various degrees of saturation of these internal pores and of the surface. Stockpiled aggregates could be in any of the conditions shown in Figure 3.1.

Water absorption of concrete aggregates is determined in accordance with SANS 5843.[3.1] SANS 1083:2006[3.2] makes no reference to water absorption of aggregates. *Commentary on SABS 1083:1994*[3.3] advises caution in applying rigorous standards for water absorption of sands because the test method has poor reproducibility. In the previous edition of SANS 1083, 1% was deemed to be a "safe" value. Higher absorption may adversely affect workability and durability of concrete.

Typically the absorption of Southern African quarried aggregates is below 0,5%. But some aggregates, eg basalt considered for use in the Lesotho Highlands Water Project, had absorption significantly higher than this. Precautions are called for in batching and mixing concretes made with such aggregates, particularly when the moisture content of the aggregates is less than that of a saturated surface-dry state because the subsequent absorption of mixing water will lead to slump loss and cause difficulties in the placing of concrete. *Commentary on SABS 1083:1994*[3.3] notes that the use of water-reducing admixtures may significantly increase water absorption of "moderate absorption" (2 to 4%) aggregates.

Porosity is believed to be a property that influences compressive strength and the resistance of an aggregate to aggressive environments. Porous aggregates should not be used in freeze-thaw conditions. However, there have been no reports of problems ascribed to aggregate porosity in South Africa. Nevertheless, porosity should be considered where impermeability or high resistance to chemical attack are important requirements.

Commentary on SABS 1083:1994[3.3] discusses the soundness and durability of aggregates as measured by SANS 5839 *Soundness of aggregates (magnesium sulfate method)*.[3.4] In a recent review of this method, C&CI found the description of the calculation to be ambiguous. Depending on the interpretation of this part of the method, this ambiguity could lead to considerable differences in the result reported. Although a recommendation was made to the SABS Technical Committee to consider replacing this method with the British Standard BS 812:121:1989, *Testing aggregates. Part 121. Method for determination of soundness*[3.5] this was not done when the SANS standard was revised in 2006. It is therefore suggested that the BS method be used for guidance when calculating the result of a soundness test on an aggregate sample.

3.2.2 Dimensional properties

One of the fundamental reasons for including aggregates in concrete is to improve the dimensional stability of the hardened concrete. The dimensional stability of concrete is influenced by aggregate properties such as drying shrinkage, wetting expansion, thermal volume change, elastic stiffness, and creep. Each of these properties is discussed below.

Drying shrinkage

Most commercially available aggregates are far more dimensionally stable under changing moisture conditions than hardened cement paste. The reduction in drying shrinkage brought about by the introduction of sand and stone with normal (ie low) shrinkage potential is illustrated in Table 3.1.

Table 3.1: Typical drying shrinkages

Material	Shrinkage, microstrain
Hardened cement paste	2 500 - 3 000
Mortar	600 - 1 200
Concrete	300 - 800

If a coarse aggregate with high shrinkage potential is used in concrete then the shrinkage of the concrete could fall in the range for mortar. Similarly if the sand used in a mortar has high shrinkage potential then the shrinkage of the mortar could fall in the range for paste.

Excessive drying shrinkage of concrete, made with aggregates of high shrinkage, has been shown to be a cause of deterioration of reinforced concrete in some parts of Southern Africa.[3.6] Particularly notable amongst such aggregates are some of the Karoo sediments. Where the excessive shrinkage is restrained by reinforcement, shrinkage cracks develop in the concrete. These cracks, depending on their width, may permit access of moisture to the steel. In structures exposed to the weather, corrosion of the steel and the consequent expansion lead to further cracking and spalling of the concrete. The difference between the free shrinkage in unreinforced zones and the restrained shrinkage in reinforced zones of a structural member may lead to serious deformation, eg sagging of slabs, and consequent cracking of the structure. Fortunately, with dissemination of information on this problem, use of such aggregates with high shrinkage in concrete in South Africa tends to be the exception rather than the rule.

Rocks and minerals that can cause deterioration of concrete because of excessive volume change are: nontronite; montmorillonite; kaolinite; some illites; some deuterically-altered basalt and dolerite; and some argillaceous limestone. Montmorillonite clay is far more dangerous in this respect than kaolinite.

Commentary on SABS 1083:1994[3.3] gives details of the three SABS test methods for measuring shrinkage. (See also section 8.5.6.) It should however be noted that Badenhorst[3.7] has raised serious concerns about the relevance of the SANS 6085 concrete shrinkage test method and has recommended the development of a concrete shrinkage test capable of giving a 28-day result which could be related to design values and performance of concrete in structures.

Wetting expansion

Wetting expansion is the converse of drying shrinkage. If concrete that has undergone shrinkage through loss of moisture is wetted, some of the shrinkage will be recovered. Repeated wetting and drying of concrete leads to microcracking of the concrete.

Typically between 50 and 70% of the drying shrinkage is recovered on rewetting. Where aggregates with high shrinkage potential have been used, the recovery could be as high as 90% of the original shrinkage. In one case C&CI recorded a recovery after a number of wetting and drying cycles of more than 100%, ie an expansion, for a concrete made with a metallurgical slag.

Elastic modulus

Elastic modulus (also known as Young's modulus), which is stress divided by strain, is used to quantify elastic stiffness.

The addition of aggregate to cement paste has the effect of increasing the elastic modulus: the mixture – concrete – has an elastic modulus between those of paste and aggregate. Figure 3.2 illustrates typical stress-strain behaviour of aggregate, concrete and cement paste.[3.8]

While aggregate and paste exhibit a fairly linear stress-strain relationship, concrete does not. The non-linear behaviour of concrete is probably due to the influence of the interfacial zone between paste and aggregate. This zone is characterised by high porosity and the presence of portlandite ($Ca(OH)_2$) crystals, which result in lower strength and stiffness.

Davis and Alexander[3.9, 3.10] undertook studies on several South African aggregates, and more recent values have been published by Holcim.[3.11] They evaluated the elastic moduli of these aggregates and also studied the elastic moduli of concrete made with them. They showed that aggregate can have a significant effect on the elastic modulus of concrete in which it is used. They proposed a formula which, using values which they provided for a range of South African aggregates, gives a close estimate of the elastic modulus of concrete made with a specific aggregate. (See also Chapter 8.)

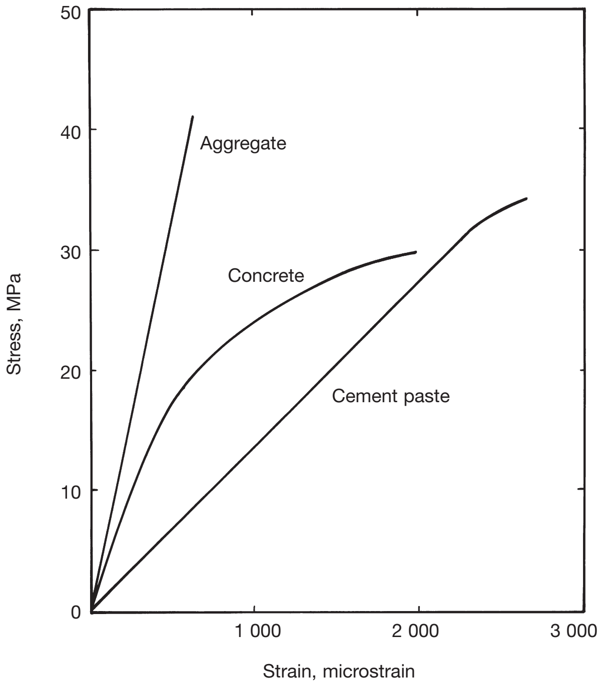

Figure 3.2: Stress-strain relations for cement paste, aggregate and concrete

Creep

Creep is defined as the increase in strain or deformation under a constant stress. The main contributor to creep in concrete is the hardened cement paste. As aggregates show very little creep under load, the effect of introducing aggregates to a cement paste to make concrete is to reduce creep deformation by diluting the paste and by restraining the tendency of the paste to creep. An increase in the aggregate:cement ratio of concrete will therefore decrease the creep potential of the concrete.

Aggregate stiffness also has an influence on the creep of concrete, the stiffer the aggregate (ie the higher the elastic modulus), the lower the creep.

Davis and Alexander[3.9, 3.10] reported on the effect on concrete creep of various South African aggregates and showed the very considerable range in creep results obtained when aggregates from different sources were used. They suggest modifications to the BS 8110[3.12] method of estimating creep from a formula. The proposed modification is a factor by which the calculated creep value is multiplied; the factor is specific to aggregate from a particular source and is derived from their experimental data.

Thermal movements

The volume of concrete expands and contracts with increasing and decreasing temperature. The linear coefficient of thermal expansion (CTE), which is used to quantify this behaviour, depends on the mix proportions and constituents of the concrete. This is because the CTEs of hardened cement paste and aggregates can be in a wide range and can differ markedly from each other. Table 3.2 below shows the range of typical values of CTE for concrete, hardened cement paste and aggregates.

Table 3.2: Typical coefficients of thermal expansion of concrete and its constituents within the range of normal ambient temperatures

Material	Coefficient of thermal expansion, microstrain/K
Concrete	5 - 18[3.10]
Aggregate	4 - 13[3.10]
Hardened cement paste	circa 20

Davis and Alexander[3.10] carried out tests on some South African aggregates and provided data on the coefficients of both aggregates and concrete made with those aggregates. Their coefficients of thermal expansion of the concretes are summarised in Table 3.3.

Table 3.3: Coefficients of thermal expansion of concrete made with aggregates of various rock types within the range of normal ambient temperatures[3.10]

Material	Coefficient of thermal expansion, microstrain/K
Andesite	7,4
Dolerite	6,0 - 8,1
Dolomite	7,5 - 9,2
Felsite	9,2
Granite	6,4 - 9,7
Greywacke	10,9
Quartzite	9,4 - 12,2
Tillite	6,6

3.2.3 Grading, fineness modulus and dust and clay content

Grading

The grading of an aggregate refers to the proportions that pass through sieves with square openings of various sizes, and is determined by passing a representative sample through a series of standard sieves. Concrete aggregates are divided into two categories: fine aggregate or sand, and coarse aggregate or stone. The sieve opening size separating coarse from fine has been chosen as 4,75 mm. It should be noted that most standard specifications for concrete aggregates will permit some material retained on the 4,75-mm sieve in a sand, and will similarly allow the presence of some material passing the 4,75-mm sieve in a stone.

The standard SABS sieve sizes (selected when South Africa metricated in 1960), in millimetres, are: 75, 37,5, 19, 9,5, 4,75, 2,36, 1,18, 0,6, 0,3, 0,15 and 0,075.[3.13] In addition, 53, 26,5, 13,2 and 6,7 are frequently used intermediate sizes.

A grading analysis of the aggregates can be used to provide parameters to be used to:

- Determine the proportions of the materials in a trial concrete mix.
- Ensure that materials delivered to site have consistent properties by means of repeated tests of material from one source.

The grading of sand and stone has a major influence on the workability, cohesiveness and bleeding properties of concrete, but particle shape and surface texture must also be recognised as important features of aggregates since these have a significant effect on the water requirement of the concrete mix.

Sand grading

While the grading of sand has a relatively minor effect on concrete water requirement, it can have a major influence on the workability, cohesiveness and bleeding of fresh concrete. In general terms, it is important that the particle-size distribution of a concrete sand is not too narrow as this may lead to harshness and excessive bleeding in the fresh concrete.

Despite extensive investigations carried out over many years, it has not been possible to determine from basic principles an optimum aggregate grading that will ensure suitability for any particular case. Suitable gradings are thus generally arrived at empirically by trial of the materials in a concrete mix. However, empirically derived grading envelopes are generally remarkably similar whether the sand is used in pumpable concrete, in mixes intended for use with sliding formwork or high-quality off-shutter finishes. Experience gained over many years has shown that the proportions of particles of the sand passing the 75-, 150- and 300-μm sieves have the greatest effect on the properties of the fresh mix. The contribution of the other fractions is less important.

Other than influencing the coarseness and adding body to the sand, the particles retained on the 300-μm sieve have little effect on the properties of fresh concrete.

The minus-150-μm and minus-300-μm fractions play a major role in determining the cohesiveness and workability of a fresh concrete mix. Sands deficient in these sizes tend to produce concrete that is harsh and inclined to segregate and bleed. On the other hand an over-abundance of these fractions may cause stickiness of the fresh concrete, in some cases to such an extent that handling difficulties are experienced. These sizes are also responsible in some instances for the entrapment of excessive quantities of air and can thus reduce strength, but this problem is not normally encountered where there is sufficient minus-75-μm material.

Consideration should also be given to the total fines content – fine sand particles plus cementitious material – of a concrete mix. A sand which is well suited to average structural grades of concrete may cause concrete to be sticky when the cementitious content is high, as would be the case for strength grades above say 40 MPa.

Excessive quantities of minus-75-μm material may increase the water requirement of the mix with a consequent increase in drying shrinkage, particularly if natural sands are used. The water requirement and drying shrinkage of concrete are therefore useful indicators of the suitability of a sand.

SANS 1083-2006[3.2] specifies maxima for the dust (minus-75-μm) content of natural or crusher sands. This fraction could be fine sand, but may also contain appreciable quantities of silt or clay. Silt and clay are more likely to be found in natural sands than in crusher sands where the bulk of the dust is likely to be rock flour. *Commentary on SABS 1083:1994*[3.3] provides additional detail intended to help with interpretation of the SANS 1083 limits, particularly as far as the SABS test methods for particle size determination for the minus-75-μm fraction[3.13] and deleterious clay content[3.14] are concerned.

It is important however to ensure that sufficient dust, consisting of sound material, is used in a concrete mix to ensure the desired properties in both the fresh and hardened state. South African practice is to take advantage of the fines occurring naturally in concrete sands, although the dust content of the aggregates used should be monitored to ensure that it is consistent. The minus-75-μm fraction assists in controlling bleeding in fresh concrete, and consequently reduces the adverse effects this might have on compressive strength, bond, permeability and potential durability. The optimum proportion of this fraction depends, as mentioned before, on the total fines content of the proposed mix and the composition of the particular dust fraction.

Generally fine sands with high dust contents will be suitable for lean concrete mixes, while sands with low dust contents will be suitable for rich mixes.

Table 3.4: Grading of sands for conventional concrete

Sieve size, mm	Cumulative percentage passing sieve			
	Limits: SABS 1083:1994		**Recommended by C&CI**	
4,75	90 - 100		90 - 100	
2,36	-		75 - 100	
0,18	-		60 - 90	
0,60	-		40 - 60	
0,30	-		20 - 40	
0,15	5 - 25		10 - 20	
0,075	Natural sand, max. 5(1)(2)	Crusher sand, max. 10(1)(2)	Natural sand 5 - 10	Crusher sand 5 - 20

(1) The limit for a blend of natural and crusher sand should be between 5 and 10%, and proportional to the percentages of each type of sand in the blend.

(2) Limits may be doubled, provided that:
- The sand complies with the requirement for the methylene blue adsorption value.[3.13]
- The clay content (material of particle size smaller than 5 μm) of the sand does not exceed 2%.

Many sands have gradings that fall outside envelopes recommended in Table 3.4. It may not be economically feasible to reject such sands and therefore some means need to be found to make them more suitable for the particular application in concrete. It is often possible to do this by blending of fine and coarse sands, so that the resultant grading is closer to the optimum.

The grading of a blend may be calculated or determined graphically from the gradings of the sands used in the blend.

Note that where sands have different particle densities, the calculation should be based on the absolute volumes of the sands. (See also section 11. 5 Example 2.)

SANS 1083:2006[3.2] specifies different levels of dust and the minus-150-μm fractions for natural and crushed sands. In calculating the grading of a blend of natural and crushed sands as above, SANS 1083 indicates that the permissible level of dust content for the blend is calculated as the arithmetically weighted value.

The effect of variation in sand grading is far more pronounced than that of variations in stone grading. For this reason a maximum variance of fineness modulus (FM) of 0,2 is often specified for concrete sand. It should be stressed that when sand gradings stray outside these limits, a minor adjustment to the proportions of sand and stone is usually all that is required to restore the workability of the fresh mix.

The influence of sand grading is less with rich than with lean mixes; the influence increases as workability increases or cement content decreases.

Stone grading

The grading of stone has a less influence on the properties of the fresh concrete than that of the sand, but haphazard grading is not advisable for concrete intended for important applications as this is certain to impact negatively on the uniformity of the concrete. For this reason, and because of variability caused by segregation, all-in gravel and crusher-run stone should be used with great caution. It is advisable to screen such materials into separate fractions and such fractions should be batched separately into the mixer in appropriate ratios.

The grading of stone is more easily controlled than that of sand because stone may be purchased in nominally single sizes which can then be combined at the mixer in any desired proportions. A proportion of over-size particles in the various fractions of stone can be expected as a result of screen wear, and a percentage of under-size particles is also generally present.

The presence of some over-sized particles is generally unimportant while an excess of under-sized particles may result in an increase in the water requirement of the concrete and could also result in aggregate interference with coarse particles of the sand. Work done in the C&CI laboratories showed the influence of stone grading on the properties of fresh concrete to be insignificant. The graph in Figure 3.3 shows the extremes of the range of gradings used for trial mixes in the laboratory; satisfactory quality concrete was

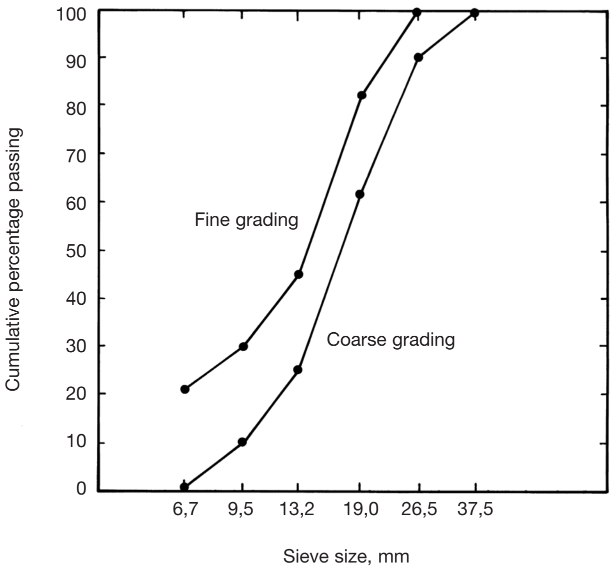

Figure 3.3: Range of stone gradings used to investigate effect of stone grading on concrete properties

produced with stone from each end of this spectrum, and the effect of this change on water demand was insignificant.

Insistence on narrow limits of grading may result in the rejection of locally available materials and consequent increases in costs. With thorough testing, it may be shown that many aggregates outside such limits do not need to be rejected.

When stone of up to 26,5-mm size is required, it is usually preferable to use a nominally single-sized stone. So-called single-sized stone however commonly contains a small fraction of over-sized material and a larger quantity of under-sized material. Provided that the overlap of gradings of the sand and stone does not result in particle interference, this distribution of sizes may improve the flow characteristics of the fresh concrete. (Particle interference occurs between different sizes when the distance between the larger particles is not sufficient to allow movement of the smaller particles.)

Experience has shown that nominally single-sized 19-mm or 26,5-mm stone normally combines well with the coarse crusher sands that are freely available in many parts of South Africa. Provided that the slump is kept reasonably low, the combination of single-sized stone and fine natural sand also produces concrete which is acceptable for most purposes. (The design of mixes using very fine sands is described in section 11.3.6).

It should be noted that the most commonly specified coarse aggregate size is 19 mm. When a quarry/crusher operation makes 19-mm stone, other sizes are also produced. Unless these other sizes can also be sold to cover costs, this "waste" will increase the cost of producing the 19-mm stone. Contractors should be aware that where circumstances and specifications permit, the total aggregate cost of the concrete might be reduced if other, less popular, sizes are used.

The use of single-sized stone, ie gap-graded concrete, as recommended above, applies to much concrete work. But for mixes of high workability having slumps of 100 mm and over, it is often preferable to use a graded stone; this can commonly be achieved by using a blend of more than one size of stone in the mix. Highly workable mixes are much more susceptible to segregation than mixes of medium or low slump and segregation may be reduced by the use of a graded stone.

A problem associated with the use of continuously graded aggregate is that critical proportions of similar sized particles especially in the 2,36 to 9,5-mm range can cause particle interference.[3.15] This may be a relatively minor nuisance when the aggregates concerned are rounded river pebbles, but when crushed materials are used the concrete is often intractable and unresponsive to vibration.

The relative merits of gap-graded and continuously graded stone in concrete have been much debated.[3.16 - 3.20] The technical advantages of each are listed below but, in comparing merits, it must be borne in mind that economic considerations may outweigh technical benefits. For instance, graded stone may be cheaper than single-sized stone but the inconvenience of storing several different sizes of stone and the extra problems of batch control may encourage the use of only one size of stone.

Advantages of gap-graded stone
- Less likelihood of particle interference
- Greater sensitivity of consistence to changes in water content (This makes for more accurate control of mixing water which in turn ensures more consistent concrete compressive strength results.)
- Greater responsiveness to vibration of stiff mixes

Advantages of continuously graded stone
- Less segregation of wetter mixes
- Less sensitive to slight changes in water content (This is an advantage where uniform workability is important.)
- Improved pumpability especially at higher pressures
- Improved flexural strength due to the increased surface area of graded stone

The maximum size of stone used in concrete has a considerable effect on the water content required for a given consistence. The larger the maximum size of stone, the less the water required and consequently the more economical the mix in terms of the cost of cementitious binder for a given strength. For instance, a mix made with 37,5-mm stone will require approximately 15 ℓ/m^3 less water than a corresponding mix made with 19-mm stone. (See Table 11.3 for the effect of stone size on water requirement.) It is therefore desirable to choose the largest possible stone size, subject to practical considerations. For example, nominal stone size should normally be not larger than about one fifth of the least dimension of concrete section and about 5 mm less than the minimum clear distance between reinforcing bars. For high-quality reinforced concrete in an aggressive environment, the use of a maximum stone size 5-mm less than the cover to steel should also be considered. It should also be borne in mind that the smaller the stone size, the easier the concrete will be to handle and place and the less likely it will be to segregate.

When single-sized 37,5-mm stone is used, it should be blended with 19-, 13,2- or 9,5-mm stone. While a wide range of blends may be used, generally speaking the greater the difference in size of the aggregates, the lower should be the proportion of the smaller size. Some technologists combine sizes on a maximum density basis while others believe that best results are obtained by following a smooth grading curve.

Stone up to 250 mm in size has been used in the building of dams and for other massive structures but 150 mm is generally regarded as being the practical upper limit.[3.21] When wear and tear on the plant used in producing such concrete is taken into account, many contractors in South Africa are of the opinion that 100- or 75-mm stone is the most economical maximum size. As the maximum size increases so a larger number of intermediate sizes may be introduced with increased opportunity for using either gap- or continuously graded material.

The American Concrete Institute[3.21] states that a wide range of proportions of materials from different size fractions may be used to make suitable blends. The distribution of intermediate sizes is rarely critical but the need for gap grading is greater if the particle shape of the stone is poor. It is often possible to use a blend of sizes that will more fully utilise the crusher output. Alternatively, unused or under-utilised stone sizes may be used to make crusher sand.

Fineness modulus

This dimensionless parameter is a measure of the average particle size. Fineness modulus (FM) is determined in accordance with SANS 201;[3.13] it is an empirical factor obtained by adding the total percentages, of the original mass of the test sample, of material retained on each of the standard sieves, excluding the 75-µm sieve, and dividing the sum by 100. An example of the calculation of FM of a sand sample is given in Table 3.5.

Table 3.5: Determination of the fineness modulus of sand: example (Mass of test sample = 567 g.)

sieve size, µm	Retained on specific sieve		Cumulative % of total mass	
	Mass, g	% of total mass	Retained on sieve	Passing sieve
4 750	0	0	0	100
2 360	52	9,2	9,2	91,8
1 180	156	27,5	36,7	63,3
600	109	19,2	55,9	44,1
300	153	27,0	82,9	17,1
150	65	11,5	94,4	5,6
Total			279,1	
FM = 279 / 100 = 2,79				

It is important to note that in the calculation of FM only standard sieves are considered; intermediate sizes, ie 6,7-, 13,2- and 26,5-mm sieves, are ignored.

While FM is most commonly used when referring to characteristics of sands, a similar calculation can be done for stone.

A weakness of FM as an index is that a single FM value can apply to a wide range of different sand gradings.

FM is used to categorise sand or, less frequently, stone, as coarse, medium or fine. A very fine sand is one with an FM below 1,0, and fine sands would be those with an FM up to 2,0. Medium sands have an FM ranging between 2,0 and 2,9; sand with an FM above 2,9 can be regarded as coarse. FM of sand is a parameter used in the proportioning of concrete mixes (see Chapter 11).

Dust and clay content

SABS 1083:1976 (now superseded) defined the material passing the 75-µm sieve as dust. Limits were placed in the specification on the dust content of fine and coarse concrete aggregates. It is believed that this was done to ensure that the concrete did not become contaminated with clay minerals, particularly the active smectite clays, which could lead to undesirably high drying shrinkage in the concrete

It should be noted that while use is often made of particular particle sizes in the defining of what is regarded as clay, true clay (eg kaolinite, bentonite, etc) is mineralogically distinguishable from other materials (eg iron oxide, fine quartz, etc) which might also, coincidentally, be of similar particle size. Particle size, therefore, while being convenient, is not a meaningful indicator of the presence or absence of clay. Nor does the definition distinguish between the different types of clays, so that highly active clays of the smectite type are categorised along with less active types such as kaolinite.

SANS 1083:2006[3.2] has dealt more rationally with this subject. A test method for the determination of the proportion and nature of the material smaller than 5 µm (ie clay) was introduced[3.22] when SABS 1083:1994 was published. At the same time another test method was introduced to measure the level of activity of the "clay" fraction using the methylene blue indicator test.[3.14] These tests can and should be used where it is desirable to include additional aggregate fines without unduly increasing the concrete water requirement or drying shrinkage.

Work at C&CI established that the compressive strength of concrete increases as the dust content of the fine aggregate increases. (It is believed that the mechanism involved here is the "fine filler" effect, which is described in section 1.12.2.) This is true irrespective of the clay content of the dust, but it is recommended that caution be exercised when working with materials that contain high proportions of active clays. An extreme case of this phenomenon was observed when a dolomite crusher sand was used in a concrete mix made with 19-mm stone. Although the sand had a dust content well in excess of 20%, the mix proved to be workable and had a water requirement of 190 ℓ/m^3 for a slump of 75 mm.

The 28-day compressive strength results were found to be some 10 to 15% higher than expected. Drying shrinkage as measured using the method in SANS 5836 [3.23] was 0,085% compared with 0,090% for the reef quartzite control despite the very high dust content.

While few sands with such high dust contents would normally be considered for use in concrete, it seems that the principle of improvement in the quality of concrete with an increase in dust content can be applied fairly generally. C&CI laboratory tests showed similar trends with the use of processed decomposed granite sand, and andesite, granite, norite and reef quartzite crusher sands.

It is believed that, for normal grades of concrete, a high dust content (within reason) is better than a low one. For acceptable control of bleeding, the dust content (ie material passing the 75-μm sieve) should not be much below 5%; 10% would probably be better under most circumstances. This statement rather contradicts the requirements in SANS 1083 as a maximum dust content is specified for sands derived from natural sources although the standard does allow for this to be increased provided the active clay content is not too high.

3.2.4 Organic material in aggregates

Organic material, if present in natural sand, can result in problems with the use of such sands in concrete or mortar. The main problem appears to be one of retardation of setting, although discolouration of concrete or mortar is also possible.

SANS 5832[3.24] Organic impurities in fine aggregates is an indicator test for the presence of organic material; the test does not show whether the organic material present is deleterious or not. The test involves mixing a sample of the sand with a sodium hydroxide solution; a colour change in the solution indicates the presence of organic material. In the past it was necessary to prepare a control solution against which the colour of the suspect sample was compared. SABS has found a stained glass sample that has the desired control colour, thus simplifying the procedure. It should be noted that a sand with an unacceptably high organic content can still be accepted provided it is shown there is no detrimental effect on compressive strength (see SANS 5834[3.25] Soluble deleterious materials in fine aggregates).

SANS 5832 has drawbacks and presents some difficulties in the interpretation of results because:

- Certain organic substances in sand show colours darker than the standard, suggesting that the sand should be rejected whereas in fact these substances may have little or no effect on the strength or durability of concrete. An example of this is sand from the Richards Bay area containing substances said to be derived from gum leaves. A number of samples of this sand tested by C&CI failed the standard test for organic impurities but the sand proved in practice to be satisfactory in concrete.
- The presence of sugar cannot be detected by the standard test.

SANS 5833[3.26] *Detection of sugar in fine aggregates* is a test intended to identify the presence of sugar as a contaminant of a sand because the presence of sugar can have a significant effect on the setting time of concrete and mortar. The test is designed to detect a wide range of different sugars. It should be noted that sugar, when exposed in damp conditions, will tend to ferment and convert to alcohol in a relatively short time. The alcohol will not be detected in this test thus rendering the test useless in determining the cause of some retardation problems.

Commentary on SABS 1083:1994[3.3] covers these issues in some detail.

3.2.5 Particle density

Knowledge of the particle density of each constituent of a concrete mix enables the concrete technologist to calculate the absolute or solid volumes of the materials from their masses and so to calculate the yield of the mix.

Particle density influences the choice of an aggregate for concrete where the concrete density is an important factor, for example in gravity dams, radiation screening and sound and thermal insulation.

Concrete used for radiation screening is commonly made with aggregates such as barytes, magnetite or haematite which, amongst other properties, have high relative densities. (See also Chapter 18.)

Where the aggregate particle relative density is outside the normal range of between 2,5 and 3,0, difficulty might be experienced with mixing, transporting, placing, compacting and finishing concrete made with such aggregate. Segregation is also more likely to be a problem.

3.2.6 Particle shape and surface texture

Aggregate particle shape is probably the most important factor that determines the performance of the aggregate in fresh concrete. The shape of the coarse and particularly the fine aggregate particles has a strong influence on the water requirement of concrete and mortar mixes. The closer the particle shape approaches a spherical or cubical shape, the lower the water requirement is likely to be; conversely, the flakier or more angular or elongated the particles are, the higher the water requirement is likely to be. *Commentary on SABS 1083:1994*[3.3] covers this issue in some detail.

Similarly, as the roughness of the surface texture increases so the water requirement is likely to increase, due to the greater surface area requiring wetting, and the likelihood of the increased friction and mechanical interlock between particles. The effect of surface texture on water requirement is less than that of particle shape.

Particle shape can be quantified (after a fashion) in a number of ways. Flakiness, usually of coarse aggregate, can be determined using SANS 5847,[3.27] although it is not a requirement for coarse aggregate for use in concrete applications in SANS 1083.

The voids content of an aggregate may also be used as an indication of particle shape: the better the particle shape, the lower the voids content. (However, grading also influences voids content: graded aggregates have a lower voids content than single-sized aggregates of the same particle shape.)

The voids content of either coarse or fine aggregate can be determined using SANS 5845[3.28] *Bulk densities and voids content of aggregates.* If voids content is to be used as an index of particle shape, the test is done on the size fraction that represents the greatest proportion of the aggregate. If voids content is to be used as an indication of the packing capacity of the aggregate then the whole of the sample should be tested. In the 1976 edition of SABS 1083 the suggested upper limit for voids content of a single-sized coarse aggregate was 48%; no information on this subject is provided in SANS 1083:2006[3.2] but the matter is discussed in *Commentary on SABS 1083:1994*[3.3].

Fine aggregate has considerably more influence than coarse aggregate on the water requirement of concrete. Tests carried out on several South African sands[3.29] gave an average void content of 36% with a range from 28 to 44%. Changing from one extreme of this range of 16 percentage points in voids content to the other was reported to affect the water requirement of concrete to the extent of about 70 ℓ/m^3. Tests on nominally single-sized 19-mm stone gave an average voids content of 46% with a range from 34 to 50%. Changing from one extreme of this range of 16 percentage points in voids content to the other was reported to affect the water demand by only about 30 ℓ/m^3. It can be seen that, while the range was the same for both sand and stone, the effect of variation in the void content of the sand was more than twice that of the effect of the stone.

Because the particle shape has a significant effect on the water requirement, measurement of water requirement can be used as an indirect measure of particle shape. SANS 5835[3.30] *Water demand of fine aggregates for concrete* details a calculation for determining water demand from the particle size distribution of sand. C&CI has shown this method to be unreliable for the range of sands available in Gauteng. It must be remembered that the particle shape, while being the most significant factor in determining the water requirement of a mix, is not the only factor. Particle surface texture, fines content and mineralogy are other factors that affect water requirement.

It is worth noting that it is possible to exercise considerable control over the shape of aggregates by adjustments to the production process. It is true that for many rocks such as shale, slate and schist, which are characteristically laminated or show marked cleavage planes, the inherent excessive flakiness of the rock cannot be overcome, but it is equally true that with a very wide range of rocks, flakiness of the aggregate is primarily due to poor aggregate production techniques.

In a summary of the literature dealing with the factors governing the shape of crushed rock, Shergold[3.31] states that the conditions that favour the production of good cubical aggregate are:

- A low reduction ratio, especially in the final stages of crushing
- Removal, by scalping, of chippings and fines formed in primary crushing
- Choke feeding
- Closed circuit feeding
- Use of corrugated crushing surfaces

Authorities agree that the most cubical and the soundest particles are produced by the smallest possible reduction ratio, and this would seem to be the most important of the above conditions. An exception to this rule is believed to be the category of impact crushers which produce aggregates of good shape even with high reduction ratios.

3.2.7 Soundness

In appendix C-11 to the 1976 edition of SABS 1083, it was pointed out that the sulphate soundness test, now SANS 5839,[3.32] is not highly selective but provides a rough indication of the durability of an aggregate when it is subjected to aggressive environmental conditions. There is no evidence available to suggest that the disruption in the test can be directly correlated with performance of an aggregate when subjected to these conditions. For this reason it is important that when an aggregate is to be evaluated on the basis of this test, it should be done by comparison with an aggregate of similar mineralogical composition and geological history and which has proved satisfactory in service. C&CI considers SANS 5839[3.32] to be incorrect in its description of the method of calculating the results. C&CI has proposed that SABS consider cancelling this test method and replacing it with the BS 812:121[3.33] test method for soundness, as the result is not subject to the mathematical manipulation which is seen as a weakness of the ASTM Method.[3.34]

ASTM C33[3.34] seems to be the only specification that puts limits on the amount of breakdown permitted for aggregates subjected to the soundness test. Limits are 10% maximum loss for fine aggregates subject to the sodium sulphate test and 15% where magnesium sulphate is used. It should be noted that material failing this test might still be accepted for use in concrete provided that concrete made with the aggregate is able to comply with the other durability assessments such as the freeze-thaw durability test. An evaluation of existing performance records of structures made with similar materials may also be done.

In South Africa, the organisation with most experience of this test was SA Transport Services who used it to determine the suitability of aggregate for railway ballast. An experienced practitioner[3.35] examined the "corpses" of coarse aggregate particles after completion of the test and found these observations gave a good indication of the durability of that material, when considered together with the test results. ASTM C 88-05[3.36] describes a process of doing a qualitative examination of particles of coarse aggregate exhibiting distress when exposed to the test solution, ie which split, crumble, crack or flake, and this information is required as part of the test report.

At least six methods of assessing soundness of aggregates were used for investigating possible sources of basalt aggregates for the Lesotho Highlands Water Project. One of these was a wetting and drying test performed on rock core sections. Another test was an ethylene glycol immersion test which was found to be extremely severe; it was noted that an aggregate that had performed satisfactorily in service for decades might be regarded as unsuitable for use in concrete on the basis of the ethylene glycol test result. It is believed that the environment inside concrete tends to protect what might be regarded as inferior material, so that its performance in concrete is acceptable.

3.2.8 Strength

The unconfined compressive strength of specimens of South African natural stone from which acceptable aggregates are produced has been reported to range from 70 to about 540 MPa,[3.9] ie well in excess of the range of strengths of most concrete used for construction.

Within a wide range there appears to be no correlation between the compressive strength of the aggregate and the flexural or compressive strength of the concrete.[3.37,3.38] We may therefore conclude that the compressive strength of the aggregate, where this is considerably higher than that of the hardened cement paste, is not a significant property of the material.

The compressive strength test, as carried out on individual rock specimens, has poor reproducibilty, and is difficult to do because compressive strength specimens must be cut from the parent rock. The more common methods of determining the strength of aggregates are the aggregate crushing value (ACV) test (SANS 5841),[3.39] and the 10% fines aggregate crushing test (10% FACT) (SANS 5842).[3.40] Both these tests measure the same property of rock, ie the crushing strength of the broken rock; it should be noted that this cannot necessarily be equated to hardness or abrasion resistance. For ACV values in the range 14 to 30% and 10% FACT values in the range 100 to 300 kN there is a fairly good correlation between the results obtained on the two tests:[3.41]

$$\text{ACV} = 38 - 0{,}08 \times (10\%\ \text{FACT})$$
$$\text{or } 10\%\ \text{FACT} = 12{,}5 \times (38 - (\text{ACV}))$$

In each case the ACV is expressed as a percentage and the 10% FACT in kN.

The ACV and 10% FACT values are useful indices of overall quality, but the usefulness is confined to a general assessment of quality and comparison with acceptance limits specified in SANS 1083:2006. The test values are of little use when comparing aggregates in relation to their performance in concrete.

As stated above, there appears to be little correlation between the compressive strength of the coarse aggregate and that of the concrete. It should however be noted that for high-strength concrete (compressive strength in excess of say 60 MPa) it is commonly recommended that higher-strength materials (with 10% FACT in excess of about 170 kN)[3.9] are selected. (See also Chapter 17.)

However, in the case of high-strength concrete, Alexander[3.42] reports that where the density of the interfacial transition zone (ITZ) is improved, the aggregate strength can have a significant effect on concrete strength. He also points out that particle surface texture has a strong influence over the density of the ITZ and thus can also enhance the concrete compressive strength.

3.2.9 Water-soluble salt and other deleterious chemicals in aggregates

It is desirable that the aggregate should, as far as possible, be chemically inert, but many natural aggregates contain substances that are deleterious in concrete. Substances considered to be chemically deleterious may be broadly classified into five groups:

1. Substances soluble in water, which may be leached out of the aggregate thereby weakening it or promoting efflorescence in the concrete, eg common table salt.
2. Soluble substances, or substances which become soluble in the cement matrix, which may interfere with the normal hydration of the cement, eg humic acid.
3. Substances that react with the cement destroying its properties, eg sodium sulphate.
4. Substances that may react with the alkali constituents of the cement, eg opal. (See Chapter 10.)
5. Substances that may cause corrosion of the reinforcing steel, eg common table salt.

It should also be noted that while the aggregate itself may be inert, encrustations or coatings on it may be deleterious. Natural coatings may be composed of silt, clay, gypsum, impure carbonates of lime and magnesia, opaline silica, manganese oxide, iron oxide, and mixtures of these materials.

Organic substances classed under (2) above are occasionally present in concrete sands but do not commonly give rise to problems. The presence of sugar has been the source of some concrete problems that have occurred more particularly in the Natal sugar cane areas and in Swaziland. This is covered in section 3.2.4.

3.2.10 Rocks and minerals that may be deleterious in aggregates

It is normally not possible to stipulate precisely what proportion of any contaminating substance can be regarded as injurious in concrete. Among the numerous factors which should be considered are: nature of substance; its concentration; its form; particle size and distribution; conditions of exposure of concrete; and the structural and aesthetic importance of the structure. It should be noted that a contaminant which is entirely enclosed or finely dispersed within the matrix of aggregate particles is much less likely to cause a problem than where the contaminant can freely interact with water or cement paste; for this reason, caution should be exercised when interpreting test results where these are derived from finely milled aggregate samples.

The following notes have been compiled as a guide only, and judgement must be exercised in accepting or rejecting an aggregate in any specific instance.

Ores

Ores containing sulphides, for example pyrite, zinc blende (sphalerite), lead glance (galena), etc, may be deleterious to concrete. Others, like sparry iron ore, sometimes contain sulphides and are suspect on that account.

In particular, the following are known to be deleterious:

Copper pyrite (chalcopyrite)

This combines readily with moisture to form sulphates.

Iron pyrites

Reef quartzite aggregates contain iron pyrites in quantities from 0 to 0,5% by mass; small lots may contain as much as 3%. This is not normally detrimental in coarse aggregates but in fine aggregate, where oxidation to sulphates could have occurred, the sulphate content could be critical. Although it is difficult to set limits, between 0,25 and 0,40% should be considered the maximum permitted. Soils and groundwater under and in the proximity of gold mine slimes dams have been reported to have high sulphate levels resulting from the oxidation of pyrites.

Marcasite

This is particularly deleterious in warm humid areas.

Sulphate minerals

These may cause decomposition of the concrete through reaction with aluminates and calcium hydroxide in the cement.
In this class are:

- Alunite, $K_2Al_6(OH)_{12}(SO_4)_4$
- Epsomite, $MgSO_4.7H_2O$
- Gypsum, $CaSO_4.2H_2O$ (This mineral is restricted by the USA Bureau of Reclamation to 0,25% by mass of coarse aggregates.)

Aggregates deleterious because of physical characteristics

In general, all soft, friable, flat, elongated or laminated aggregates with marked cleavage planes have a deleterious effect on concrete (see Table 3.6).

Table 3.6: Materials with undesirable physical characteristics

Material	Remarks
Coal and lignite	Coal can reduce the strength of concrete considerably - particularly in soft varieties. ASTM limits the amount of material which will float in a liquid, having a relative density of 2,0, to 0,5% in the case of sand, and 1,0% in the case of coarse aggregate.
Chalk	Soft chalk may be the cause of cracking of concrete.
Slate and schist	Cleavage planes reduce strength.

The presence of shell particles in sand, in reasonable quantities, is not regarded as detrimental.

Commentary on SABS 1083:1994[3.3] gives additional details of limits of shell content and the effect that inclusion of shell has on concrete properties.

ASTM specifies a maximum permissible limit of 5% for soft particles by mass of coarse aggregate. There is no requirement in respect of sand.

Note: In general, any coatings or encrustations likely to destroy bond, and in particular any soluble chemical salts, are undesirable.

Some failures of concrete have been caused by using:

- Aggregates having coefficients of thermal expansion very different from that of cement hydrate.
- Coarse aggregates having thermal properties very different from those of the sand.

The coefficient of thermal expansion of hardened cement is between 11 and 16 microstrain/K for temperatures within the normal atmospheric range. Rocks having an arithmetical average coefficient of expansion in all crystallographic directions greatly differing from this value may be deleterious in concrete.

Cases have been reported of deterioration due to this cause. The aggregates used in two of the reported cases were a dolomitic marble with a coefficient of thermal expansion of 4 microstrain/K and calcite crystals with a coefficient of 4,3 microstrain/K.

Miscellaneous impurities

Materials which, for a variety of reasons, are regarded as undesirable in aggregates are shown in Table 3.7.

It should be noted that, in the weathered condition, the distinction between biotite and muscovite is not always clear.

Table 3.7: Miscellaneous impurities

Material	Remarks
Olivine	Highly ferriferous fayalite olivine may decompose to black chlorophaeite with an increase in volume.
Hornblende	Certain highly ferriferous hornblendes break down in the presence of moisture especially if saline, to throw out reddish limonitic material which forms undesirable coatings on the aggregate. This mineral has been found in certain metamorphosed dolerite.
Biotite, haughtonite mica	These can break down rapidly in the presence of moisture to white muscovite mica.
Humus and organic material	These are limited by colorimetric test.

3.3 Specification for aggregates

SANS 1083:2006 *Aggregates from natural sources*[3.2] is commendable in that it is a practical document in which an attempt has been made to avoid unnecessary or restrictive requirements. The specification permits the use of materials falling outside the suggested limits provided sufficient testing has been undertaken to show the suitability of a particular material for a specific purpose. To this end, the specification provides additional tests which may be necessary for further evaluation. However, with the ever-increasing demand on, and depletion of, high-quality aggregates, revision of the present standard could be considered as it may be in the interests of economy and quality to broaden some of the existing limits. Technological advances over the past few decades, coupled with experiences of satisfactory performance, can assist in providing guidelines to extending the present limits. The reader is again referred to *Commentary on SABS 1083:1994*[3.3] for additional information.

The principal features of the 2006 specification are reproduced here by courtesy of the South African Bureau of Standards.

3.3.1 Definitions

Aggregate

Sand or stone formed by the natural disintegration of rock, or produced by the mechanical crushing or milling of rock.

Fine aggregate

Sand of particle size such that at least 90% passes a sieve having square apertures of nominal size 4,75 mm and is retained on a sieve having square apertures of nominal size 75 µm.

Coarse aggregate

Stone of particle size such that it is retained on a sieve having square apertures of nominal size 4 750-µm.

3.3.2 Requirements for fine aggregate

Grading

Not less than 90% shall pass a 4 750-µm sieve and between 5 and 25% shall pass a 150-µm sieve.

Dust content

The material passing a 75-µm sieve shall not exceed 5% (10%) by mass. (The figure in parenthesis is the permissible limit when the aggregate is derived from the mechanical crushing or milling of rock.) Exceptions are permitted provided that additional tests (see below) are done to show that deleterious quantities of expansive clay are not included.

Fineness modulus

The FM of fine aggregate shall fall within the range 1,2 to 3,5. Where FM is specified by the purchaser, the actual value shall not differ from the specified value by more than 0,2.

Chloride content

The chloride content of fine aggregate expressed as per cent by mass of Cl^- shall not exceed the following limits:

Sand for prestressed concrete	0,01
Sand for normal reinforced concrete	0,03
Sand for non-reinforced concrete	0,08

Soluble deleterious impurities

This requirement is applicable only to fine aggregate derived from the natural disintegration of rock, and is then only mandatory if tests for organic impurities and/or sugar indicate that this is necessary. The test method requires that a mortar made with the sand being evaluated shall develop compressive strength of not less than 85% of that of a mortar made with the same sand after thorough washing.

Table 3.8: Grading of stone for concrete, percentage of material passing sieve of stated size[3.2]

Nominal aperture size of sieve, mm	Nominal size of aggregate, mm							
	75,0	**53,0**	**37,5**	**26,5**	**19,0**	**13,2**	**9,5**	**6,7**
75,0	100	100						
53,0	0 - 50	85 - 100	100					
37,5	0 - 25	0 - 50	85 - 100	100				
26,5	0 - 5	0 - 25	0 - 50	85 - 100	100			
19,0		0 - 5	0 - 25	0 - 50	85 - 100	100		
13,2			0 - 5	0 - 25	0 - 50	85 - 100	100	
9,5				0 - 5	0 - 25	0 - 55	85 - 100	100
6,7					0 - 5	0 - 25	0 - 55	85-100
4,75						0 - 5	0-25	0-55
2,36							0-5	0-25
1,18								0-5
Dust content	Not to exceed 2,0%							

Table 3.9: SANS test methods for aggregates

Number	Title
195:2006	Sampling of aggregates
197:2006	Preparation of test samples of aggregates
201:2008	Sieve analysis, fines content and dust content of aggregates
202:2006	Chloride content of aggregates
5831:2006	Presence of chlorides in aggregates
5832:2006	Organic impurities in fine aggregates (limit test)
5833:2006	Detection of sugar in fine aggregates
5834:2006	Soluble deleterious impurities in fine aggregates (limit test)
5835:2006	Estimation of the effect of fine aggregate on the water requirement of concrete
5836:2007	Effect of fine and coarse aggregate on the shrinkage and expansion of cement:aggregate mixes (mortar prism method)
5837:2002	Low density materials content of aggregates
5838:2006	Sand equivalent value of fine aggregates
5839:2002	Soundness of aggregates (magnesium sulfate method)
5840:2002	Shell content of fine aggregates
5841:1994	Aggregate crushing value of coarse aggregates
5842:2006	FACT value (10% fines aggregate crushing value) of coarse aggregates
5843:2002	Water absorption of aggregates
5844:2006	Particle and relative densities of aggregates
5845:2006	Bulk densities and voids content of aggregates
5847:2002	Flakiness index of coarse aggregates
5848:2002	Polished-stone value of aggregates
5849:2006	Total water soluble salts content of fines in aggregates
5850-1:1998	Sulfates content of fines in aggregates Part 1: Water soluble sulfates in fines in aggregates
5850-2:2002	Sulfates content of fines in aggregates Part 2: Acid-soluble sulfates in fines in aggregates
5855:1976	Free water content of aggregates
5856:2006	Bulking of fine aggregates
6241:2006	Particle size distribution of material of diameter smaller than 75 µm in fine aggregate (hydrometer method)
6242:2002	Acid insolubility of aggregates
6243:2002	Deleterious clay content of the fines in aggregate (methylene blue adsorption indicator test)
6244:2006	Particles of diameter not exceeding 20 µm and not exceeding 5 µm and smaller, respectively, in fine aggregate (pipette method)
6245:2006	Potential reactivity of aggregates with alkalis (accelerated mortar prism method)

3.3.3 Requirements for coarse aggregate

Grading

The grading of nominal single-size aggregate shall comply with the requirements as set out in Table 3.8.

10% FACT value

The minus-13,2-mm-plus-9,5-mm fraction shall be tested for the 10% fines aggregate crushing value (ie the load required to produce 10% of fines). The loads in kN shall not be less than the following:

Stone for concrete subject to abrasion:	Dry value: 110
Stone for concrete not subject to abrasion:	Dry value: 70

Alternatively, if the test is acceptable to the purchaser, the dry aggregate crushing value (ACV) shall not exceed 29%.

As mentioned earlier, for ACV values in the range 14 to 30% and 10% FACT values in the range 100 to 300 kN, there is a fairly good correlation between the results obtained on the two tests:[3.41]

$$\text{ACV} = 38 - 0{,}08 \times (10\%\ \text{FACT})$$
$$\text{or } 10\%\ \text{FACT} = 12{,}5 \times (38 - (\text{ACV}))$$

In each case the ACV is expressed as a percentage and the 10% FACT in kN.

Flakiness index

Flakiness index is an addition in the revised specification for coarse aggregate SANS 1083:2006. The value is a maximum of 35%.

3.4 Sampling and testing

It is extremely important to ensure that the aggregate samples taken for testing purposes are representative of the bulk of the material considered for use during that period. The test results may be meaningless if this is not done.

SANS Method 195:2006[3.43] describes a sampling method intended to give a reasonable probability that the sample thus obtained is representative of the lot sampled. In most cases the size of the sample produced following this or other similar methods will be larger than required for testing purposes.

SANS Method 197:2006[3.44] describes a process to reduce the size of the sample obtained above to a size suitable for testing, namely the test sample. Table 1 of this method gives guidelines for the sample size for the full range of aggregate tests.

Table 3.9 lists the existing SANS test methods for aggregates.

It should be noted that to ensure reliable test results, it is extremely important that the test house used to undertake aggregate testing should be competent to do the required tests. One way for a test house to demonstrate its level of competence is for it to achieve SANAS accreditation against the SANS 17025 standard[3.45] as one of the requirements for accreditation is to participate in inter-laboratory precision evaluations.

3.5 Origin of aggregates

The mineralogical, physical and chemical properties of rocks are determined largely by the events of their geological history. To some degree also, the physical characteristics of aggregates obtained by crushing are influenced by the processes and methods used in breaking down the rock to a size suitable for use in concrete making.

Knowledge of the ways in which rocks are formed, and of the various natural processes whereby their original characteristics are altered, may lead to a better understanding of those intrinsic properties which determine the suitability of a rock as a source of concrete aggregate. Rocks are commonly classified as igneous, sedimentary or metamorphic, according to the method of their formation.

3.5.1 Igneous rocks

Igneous rocks are those formed by the solidification of molten masses, and the rate and condition of cooling determine many of their characteristics. Extruded volcanic rocks, ie those ejected on the earth's surface, are cooled very rapidly, so that crystallisation of the component minerals is generally only partly effected, and the resulting rock consists of a mixture of crystalline ingredients and glassy matter. Cooling and consolidation may proceed so quickly that there is no time for crystals to form and the whole rock is glassy or vitreous.

On the other hand, rocks of the intruded igneous type, ie those formed from molten rock intruded into overlying rock masses to form sills, dykes, laccoliths, etc, cool slowly, sometimes under great pressure, so that several compounds of the original rock mass become thoroughly crystallised.

There are pronounced differences in character between the volcanic masses and the plutonic rocks (ie those that solidified extremely slowly deep below the earth's surface) but between the two types every transition may be found, and all the varieties shade imperceptibly into each other to form a continuous series.

Thus the texture of volcanic rocks is generally glassy or compact, the latter term being applied to rocks in which the crystals are too small to be seen with the naked eye.

Plutonic rocks generally have no glass constituent or groundmass, some being very finely crystalline, others exceedingly coarsely grained, but all types being characterised by complete crystallisation.

The structure of igneous rocks is usually massive.

All molten rock masses contain steam and vapours. While the mass is still molten these escape readily, but as the mass cools and becomes increasingly viscous the bubbles of steam or gas are trapped, becoming spherical cavities which may become flattened and elongated with the lava flow. In this way lavas sometimes solidify into rocks characterised by such cavities, which are termed vesicles. In some cases the vesicles are so numerous that the rock has the appearance of frothy glass, while in others the cavities are less abundant and the structure is termed vesicular.

Igneous rocks are the primary rocks and from these all others are derived. Their mineralogical composition is therefore of particular importance, not only with regard to their own intrinsic characteristics, but also to the properties of sedimentary or metamorphic rocks which are formed directly or indirectly by the disintegration or chemical decomposition of such primary material.

The mineral constituents of igneous rocks have been conveniently classified as follows:[3.46]

- Feldspathic silicates: feldspars and feldspathoids
- Ferromagnesian silicates: pyroxene, amphibole, biotite, olivine
- Free silica: quartz
- Accessory minerals: magnetite, ilmenite, haematite, apatite, rutile, zircon, etc

In igneous rock, with the exception of the ultra-basic types, one or more of the feldspathic silicates are found associated with one or more of the ferromagnesian silicates, with smaller quantities of accessory minerals. Quartz may or may not be present.

Acid igneous rocks contain >65% SiO_2 , intermediate rocks have between 55 and 65% SiO_2, and basic igneous rocks contain <55% SiO_2.

3.5.2 Sedimentary rocks

Sedimentary rocks are derived from the chemical or mechanical breakdown of older rocks. The fragments resulting from such disintegration accumulate in deposits, for the most part under water, and the particles may be cemented together by the deposition of other siliceous, calcareous, argillaceous or ferruginous materials to form a dense mass.

Such cementing material may be derived by precipitation from the supernatant waters, or may be accumulated, as in the case of the calcareous minerals, largely through organic agencies.

Chemical precipitates are crystalline, but by far the greater proportion of sedimentary rocks are made up of fragments of all sizes of earlier rocks. As in the case of sand or gravel beds, the fragments may lie loosely together, or may be firmly bound by strong cementing materials, and the rock may possess all intermediate degrees of compactness.

The fragments of the older rocks may be sharply angular, but as they are often transported by wind or by water for considerable distances, attrition may cause them to become smoothly rounded in their passage to the final point of deposition.

The individual particles making up the body of the rock may be large, as in the case of conglomerate, or very small, as in the case of mudstone, shale and limestone.

The particles, being derived from the disintegration of older rocks, are generally simpler in chemical composition and less liable to break down under natural conditions than those of igneous or metamorphic rocks. The durability of the rock itself therefore depends to a great extent on the nature of the cementing materials.

As a result of their mode of deposition, the structure of sedimentary rocks is almost invariably stratified, and is essentially different from that of igneous rocks. However, glacial deposits are not stratified and in this respect differ from other rocks of this class.

Because of geological and climatological changes, radical differences often occur in the nature of the material being deposited, and in consequence marked differences in the characteristics of sedimentary rocks are to be found between successive layers. For this reason limestone is frequently inter-bedded with shale, sandstone with siltstone, and quartzite with dolomite.[3.46]

3.5.3 Metamorphic rocks

Metamorphic rocks are those resulting from the alteration in place of pre-existing sedimentary or igneous rocks. Such alteration is brought about by the application of high temperatures or pressures to the rock mass, very often in the presence of percolating waters or vapours, whereby the constitution and characteristics of the original rock are partly or completely changed.

It has been observed that the metamorphism of sedimentary rocks frequently results in an improvement in hardness, durability, toughness and similar desirable properties. Igneous rocks, on the other hand, show little improvement on these properties when metamorphosed.

A large group of rocks in this class is characterised by a foliated structure, which is not to be confused with the stratification of sedimentary rocks nor with the flow banding of some lavas, but which is the result of the segregation of one or more of the constituent minerals and their re-orientation in the form of parallel plates.

Metamorphic rocks differ greatly both in structure and in texture. While all foliated rocks are crystalline, some show only very faint traces of foliation or of crystallinity. Moreover, between the metamorphic rock and its parent, all degrees of transition are found, and on that account classification may become extremely difficult.

Where the rock has been formed under high temperature and pressure, an equigranular texture and a massive internal structure result and lend great strength and toughness to the rock. Such rocks include hornfels and quartzite.[3.47]

3.5.4 Geological history

While each class of rock exhibits certain properties which influence its behaviour in concrete, its original characteristics may be modified by the events of its geological history.

Thus, cycles of freezing and thawing, thermal expansion and contraction, wetting and drying, mechanical erosion and attrition, oxidation and hydration, and the action of percolating waters may break down the original rock structure entirely. Earth stresses may lead to jointing and fractures, exposing fresh faces to attack, opening fresh channels for percolating waters, and new paths for destructive root growth.

The disruptive effects of thermal and moisture movements and the action of frost are well known, but the destructive and constructive work of percolating water is perhaps not so well recognised. The leaching out of certain constituents of the rock mass and their replacement by other compounds precipitated from percolating water is only one of many alteration mechanisms. Sand and sandstone for example have frequently been altered to quartzite by the agency of waters carrying silica in solution.[3.48]

Perhaps the most destructive of all processes is that involving the hydration of some of the mineral constituents of the rock. For instance, the spalling of granite faces is commonly more attributable to the expansion which accompanies the hydration of feldspars than to thermal movement. In this action, the feldspar is altered to clay with an accompanying release of silica which may be left in place or deposited elsewhere.

It should also be pointed out that rain water is not pure water, but always contains certain quantities of oxygen and carbonic acid derived from the atmosphere. Other inorganic materials which arise as a result of industrial, mining or agricultural activities may also contaminate ground water. Ground water may also contain, in addition, any of a wide variety of organic and inorganic compounds taken into solution during its passage through the earth's crust. The action of percolating waters may therefore be not only that of solution but may involve chemical reactions quite apart from oxidation or hydration.

The weathering effects of water on rock materials occur over a geological time scale and it is therefore likely that despite the fact that many man-made ground water contaminants are very aggressive, their effects on rock bodies have yet to become evident.

Weinert carried out a study on weathering effects on road-building aggregates and produced a map of Southern Africa with contours of climatic "N" values showing areas of equal potential weathering effect.[3.49] (N, or the Weinert number, is an index calculated from mean rainfall and mean evaporation for various areas.) Weinert indicated that for N <5, rocks are subject to chemical decomposition which leads to the breakdown of most minerals other than quartz to clay minerals. For N >5, the weathering was more likely to be physical disintegration. Weinert's chart can therefore be used to indicate the areas with high potential for the presence of clay minerals in the in-situ materials.

In some cases the initial characteristics of the rock may be so profoundly modified by agencies of weathering and alteration that it is not possible to draw conclusions regarding its suitability as an aggregate simply from the identification of its geological type. In such cases, in the absence of any history of successful service, empirical tests and petrographic examination are the only effective methods of predicting its probable performance in concrete.

3.6 South African aggregates

South African aggregates in general perform satisfactorily in concrete but in some areas there is a dearth of material that meets the normal requirements for concrete aggregates. One of the most effective means of alleviating this problem is to accept the fact that, for many applications, the use of out-of-specification materials may well serve to produce concrete of adequate quality, although this should be done with caution where a high fines content might be accompanied by a high clay content. It is a matter of great economic importance to the construction industry to make the best use of aggregates in those areas where high-quality materials are not available. To achieve this, sound engineering judgement and skilful mix design may often be required to get the best use out of a poor-quality aggregate. In making decisions about the selection criteria for the aggregates for a particular project, consideration should also be given to the service life expectancy for the proposed structure/s; clearly it would be more acceptable to use out-of-specification materials for a project with a limited service life requirement than for a project where the service life is specified as 50 or 100 years. The following section is divided into two main parts, the first being a description of the most commonly used coarse aggregate or stone, and the second describing the nature and characteristics of fine aggregate or sands commonly used in the manufacture of concrete.

3.6.1 Coarse aggregate

Aggregates used in South Africa for the manufacture of concrete may be considered under the following headings, listed aproximately in decreasing order of consumption:

- Quartzite and sandstone
- Basic igneous rocks (dolerite, gabbro and norite, basalt, andesite)
- Granite
- Greywacke
- Tillite
- Miscellaneous (dolomite and felsite)

This order of listing reflects the availability of different rock types in the vicinity of areas of high consumption rather than the suitability of the rock for use in concrete.

Quartzite and sandstone

Sandstone is one of the most common types of sedimentary rock and represents accumulations of mineral and rock fragments transported and deposited by water and wind. While quartz grains are undoubtedly the most important constituents of sandstone, it is not unusual for the quartz grains to be accompanied by highly variable amounts of feldspars, clays, rock fragments and other materials which may be abundant in the source area of the sediment. It will suffice here to note that sandstone may be classified as impure depending on the content of non-quartz materials.

A sedimentary accumulation of mineral and rock fragments becomes lithified to form sandstone by a combination of factors, the chief of which are compaction by superincumbent load and the availability of cementing materials among which silica, iron oxide and carbonates are most common. The degree of compaction and cementation largely determines the physical nature of the sandstone in that these features control properties such as friability, porosity and crushing strength.

The transition from sandstone to quartzite is gradual with intermediate stages being recognised. Sandstone becomes quartzite in the first instance by processes of recrystallisation of the original separate detrital particles resulting in the formation of an interlocking granular mosaic. Deep burial or tectonic forces affect recrystallisation. In rocks so affected, friability and porosity generally decrease while crushing strength increases and the rock tends to acquire a glassy appearance. Similar phenomena may result from cementation of the grains, especially by silica.

Quartzite of the Witwatersrand Supergroup

A substantial proportion of the quartzite used historically for the manufacture of concrete in the interior of South Africa was derived from the waste dumps resulting from the exploitation of the gold-bearing strata of the Witwatersrand Supergroup. The quartzite from this Supergroup is composed largely of quartz particles of different sizes together with interstitial micas, the whole being firmly cemented by secondary silica. While some recrystallisation effects are in evidence this feature is not marked.

The Witwatersrand Supergroup consists of an alternating succession of shale, siltstone, quartzite and conglomerate together with interbedded lavas and is cut by numerous sills and dykes of essentially basic rock of various ages. While the waste dumps of the Witwatersrand and Orange Free State gold mines contain a high proportion of quartzite reflecting the general association of the goldbearing strata with this rock type, the dumps contain in addition all the associated rock types in amounts depending on their abundance in the areas of mining activity. For instance, samples from a dump at Daggafontein (situated near Springs, east of Johannesburg) yielded only about 30% of quartzite, the remainder being shale, siltstone and fine-grained basic igneous rock. The presence of these materials in this case has little effect on the general concrete-making properties but does have an adverse influence on the workability of concrete mixes. This is due to the poor particle shape of the shale aggregates, which tend to fracture during crushing into elongated and splintery pieces.

The presence of a high content of shale, siltstone and lava in the rock from these dumps does not necessarily imply high shrinkage. Mortar shrinkage tests carried out on specimens made with these materials have generally given results of the same order as those obtained from tests conducted on quartzite. On rare occasions, portions of Witwatersrand waste dumps have been contaminated to some extent by deleterious materials consisting of decomposed basic igneous rocks which are responsible for excessive shrinkage when used as aggregate. Fortunately these materials are not abundant.

Pyrite is frequently found in the Witwatersrand quartzite but the amount of contamination is very seldom sufficient to jeopardise the structural suitability of concrete made with such coarse aggregate. On the other hand there have been occasions when Witwatersrand quartzite has been rejected for aesthetic reasons owing to the likelihood that oxidation of pyrite would cause unsightly stains and spalls on exposed-aggregate surfaces. Oxidation of exposed pyrite has occurred in these rock dumps and the resultant sulphuric acid tends to leech downwards to the bottom of the dump and into the ground below. Caution should be exercised when using material from the lower areas of such dumps for producing concrete aggregate to ensure that the sulphate levels of the concrete aggregate are within acceptable levels, if necessary by washing with water during the crushing process.

The quartzite of the Witwatersrand Supergroup has provided a valuable source of aggregates, which for many years has been accepted in Johannesburg and the Reef as the major concrete-making material. In general, quartzite has yielded high quality and reliable aggregates, as has the Witwatersrand quartzite from the Orange Free State gold mines. While on the east Rand some resources are still available for exploitation, the west Rand materials have been more considerably depleted.

Reports of suspected alkali reactivity related to the use of the Witwatersrand quartzite have been noted.[3.50] The most widely reported of these was the repair to one of the portal frames supporting the overhead section of the M1 motorway in Johannesburg.[3.51]

Hippo Quarries published a booklet on the levels of potential alkali reactivity of aggregates produced by their quarries; Witwatersrand quartzite is among the more reactive materials listed.[3.52] Much of this information relating to currently available materials is provided in a Materials Handbook published by Holcim South Africa[3.53]; only one of the materials listed in this publication is reported as being potentially reactive and this is not a quartzite. However, readers of this document are cautioned that all Witwatersrand Supergroup Quartzites should be regarded as being potentially reactive.

Quartzite from the Table Mountain Group

The second most important source of quartzite aggregate used in South Africa is the Table Mountain Group (TMG) of the Cape Supergroup which is exposed along the southern and eastern seaboards of the country. The rock types comprising this Group include both sandstone and quartzite and it is perhaps unfortunate that the term Table Mountain Sandstone has become deeply ingrained in geological literature dealing with this group of strata. Recrystallisation has effectively converted much of the sandstone to quartzite, as has silica cementation. The latter phenomenon has in places occurred selectively so that glassy quartzitic strata may alternate with more friable layers as in the Natal Group. Gradations between sandstone and quartzite are frequent in rocks of the TMG. There has been a report of crushed aggregate from the TMG being slowly alkali reactive.

Pinkish coloured quartzite from this Group supplies about 60% of the coarse aggregate requirements in Durban. In Port Elizabeth, hard compact greyish-white TMG quartzite provides the city with very nearly all of its coarse aggregate and a large proportion of its fine aggregate. This material has been shown to have a moderate degree of potential alkali reactivity.

Most of the TMG quartzite aggregate materials compare very favourably with the Witwatersrand quartzite. In some instances,[3.52] TMG quartzite in concrete has given strength and elastic modulus results slightly superior to those of the Witwatersrand quartzite. More recent studies indicate that the differences between TMG and Witwatersrand quartzite are small relative to the differences between these and other types of rock.[3.9,3.10]

Pretoria Group quartzite

Pink, red or buff-coloured quartzite from the Pretoria Group of the Transvaal Sequence quarried in and around Pretoria accounts for an estimated 40% of the coarse aggregate used in that city.

The 10% FACT values have in general been fractionally lower than those of Witwatersrand quartzite but have been more than adequate for conventional structural grade concrete. Elastic modulus tests carried out on concrete made with this aggregate have given good results consistent with those obtained for Witwatersrand quartzite. This source has been shown to produce aggregate with a very low level of potential alkali reactivity.[3.51]

Karoo sediments

Sandstone and quartzitic sandstone of the Karoo Supergroup are widespread in the interior of South Africa. The quartzitic types seldom show recrystallisation effects and their quartzitic nature results chiefly from varying degrees of cementation due to compaction. Clays of various types are common in the sediments and are believed by some to be the cause of excessive drying shrinkage.

Quartzitic sandstone of the Beaufort Group is exploited at East London for use as concrete aggregate but the stone is in general somewhat inferior for this purpose to the quartzite of other formations. A major quarry of the area yields both a bluish grey and a brown aggregate. The former is of better quality and is in demand for more important concrete applications.

Fine-grained Ecca Group sandstone is quarried in parts of KwaZulu-Natal, notably in the midlands. This aggregate exhibits shrinkage characteristics that are often slightly higher than those of the quartzite described under the previous headings but it is nevertheless acceptable for most grades of concrete work.

Basic igneous rocks

The Southern African crust has been repeatedly invaded by vast quantities of basic igneous rocks, the most important mineral components of which are feldspars and pyroxenes. Considerable textural variation is evident among basic igneous rocks and this feature is related to their eruptive history. Effusive basic rocks are referred to as basalt and are characteristically fine grained. Rocks resulting from slow cooling of basalt magma at depth develop as coarser grained materials to which the name gabbro or norite is applicable. Rocks having a cooling history between the latter and basalt are represented by dyke and sill-like intrusions of intermediate grain size commonly referred to as dolerite or diabase.

Dolerite

The volcanic activity that marked the close of the sedimentation of the Karoo Sequence resulted in the intrusion of vast numbers of dolerite sills and dykes into the Karoo beds. Dolerite dykes and sills are widespread throughout southern Africa but those associated with the Karoo are generally of greater interest to concrete manufacturers since they are more likely to be dimensionally stable than many of the aggregates derived from the sedimentary and basaltic rocks.

Dolerite is dark in colour and usually medium- or fine-grained in texture. Its two principal minerals are plagioclase and pyroxene which normally occur in the rock in roughly equal volumes. Other minerals frequently present in dolerite are olivine or quartz, while small quantities of magnetite, ilmenite, apatite and sometimes a little biotite may also be present.

Unweathered dolerite is a high-quality concrete aggregate although the use of dolerite coarse aggregate together with dolerite crusher sand commonly causes some degree of harshness and segregation in fresh concrete. Bleeding is often pronounced and is aggravated by the high particle density of the rock. Considerable improvement in concrete made with dolerite may be obtained by paying attention to the following points.

- Since bleed capacity is primarily dependent on the water content of the mix, the use of well-shaped (ie cubical or "chunky") aggregate to reduce water requirement will greatly assist in curbing bleeding. The particle shape of the crusher sand is particularly important as well-shaped particles considerably improve the placeability and flow characteristics of the fresh concrete mix.
- The presence of an adequate quantity of extreme fines in dolerite crusher sand is desirable, and approximately 8 to 12% of material passing the 75-µm sieve is generally to be recommended.
- A suitable addition of fine natural sand will usually bring about a significant improvement in workability and is often better than the use of dolerite crusher sand on its own no matter how well this is graded.
- When the harshness of dolerite mixes cannot be adequately controlled by the methods outlined above it is advisable to make use of an air-entraining admixture. Dolerite crusher sand frequently inhibits the entrainment of air and a higher-than-normal dosage of admixture may be required. It should however be noted that air entrainment might have an adverse effect on the compressive strength of the concrete and, where necessary, steps should be taken to counteract this by decreasing the water:binder ratio of the concrete mix.

In contrast to the satisfactory performance of most unweathered dolerite in concrete, dolerite that has undergone deuteric alteration is undesirable due to the presence of deleterious secondary minerals including clays which may contribute to excessive water requirement and drying shrinkage. Fines produced during crushing of even slightly weathered dolerite contain a high percentage of secondary minerals and it may be necessary to remove these materials by washing.

Areas of deuteric alteration are not uncommon in large dolerite bodies and it is not always possible to distinguish positively between undesirable and satisfactory dolerite in the field. Petrographic study may be of assistance in identification of undesirable material as is the chemically combined water content of the rock where values of about 0,5% point to an advanced state of alteration.

Dolerite is often considered to be a most difficult and expensive rock to crush but this reputation may not be entirely justified. Most of the wear and tear on crushing plant can be attributed to:

- Abrasive wear on liners, which is probably a function of the hardness of the mineral constituents and compressive strength of the rock. As quartz is the hardest of the common rock-forming minerals, liner abrasion is generally considered to be mainly a function of the quartz content of the aggregates.
- Wear on the bearings which is probably a function of the hardness or compressive strength of the rock. The quartz content of dolerite is usually of the order 0 to 5% and dolerite's abrasive characteristics may thus be expected to be less than those of quartzite and granite which have quartz contents of about 95% and 20 to 30% respectively. A large quarry producing both dolerite and quartzite aggregates reported that the jaw life of crushers processing dolerite is approximately twice as long as that of jaws crushing quartzite. The compressive strength of dolerite is high and this will tend to reduce the life of bearings of crushing equipment but it should be noted that some quartzite also has a high compressive strength, often equal to and sometimes exceeding that of dolerite.

Dolerite subjected to the sodium sulphate weathering test, ASTM C 88,[3.55] generally performs extremely well. Occasionally, however, significant reductions in mass have occurred and in these cases examination has revealed random surface pitting of the rock. The cavities are usually about 1 mm or less in diameter but sometimes reach a size of about 2 mm. This surface pitting is due to disintegration of the pyroxene exposed at the surface of the rock.

Orr[3.56] studied the occurrence and distribution of deuterically altered "rapid weathering dolerites" in South Africa and concluded that ethylene glycol could be used as a simple and quick indicator test to identify these potentially problematic concrete-making aggregates. Such dolerite is not believed likely to pose any real danger to concrete provided the particles are well protected within the matrix of dense concrete. This aspect is discussed further in the discussion on the properties of basalt.

Gabbro and norite

These occur to the north of Pretoria. Coarse-grained basic igneous rocks associated with the Bushveld Complex outcrop occur in an east-west arc several hundred kilometres in length. The basic rocks that comprise this belt are composed essentially of feldspar and pyroxene and, depending on the relative proportions and nature of these minerals, the rocks are designated as gabbro, norite, anorthosite, etc. Common usage by concrete manufacturers has entrenched the term norite for all these rock types although they are also referred to as "black granite" in the polished stone industries.

Provided that these Bushveld Complex rocks are in an unweathered condition, they provide acceptable concrete aggregate and they are extensively used in the Pretoria and Rustenburg areas. In the late 1980s a quarry started operating to the north-west of Johannesburg, producing norite material.

Norite aggregate behaves very similarly to dolerite in that its above-average particle density tends to increase segregation and bleeding of the mix.

Basalt

Karoo Eruptives

The volcanic activity responsible for the dolerite intrusions into the beds of the Karoo Supergroup also produced enormous volumes of basaltic lava. The Drakensberg Basalt, for instance, accumulated in a series of flows varying in thickness from less than a metre up to about 50 m.[3.57] In some places a succession of eruptions produced total thicknesses amounting to well over 1 200 m as, for example, those exposed on sections of the Drakensberg escarpment. Similar basalt appears along the Lebombo range in KwaZulu-Natal and Swaziland, on the Springbok Flats of the central Transvaal and in the Kaokoveld in Namibia. The basalt is usually dark grey or a greenish- or bluish-grey, but brown, chocolate or purplish hues are also found. Because of the widespread distribution of this rock, it is occasionally considered as possible aggregate where other materials are not available.

The chemistry and mineralogy of the basalt are very similar to those of dolerite but the rocks show marked differences in other respects. Dolerite commonly has a uniform crystalline texture due to relatively slow cooling at depth. Basalt, on the other hand, often contains a mixture of different-sized crystals which, on average, are generally smaller than those of dolerite.

Many basalt flows, unlike dolerite, are amygdaloidal with the gas cavities formed during the original lava eruption infilled by various secondary materials such as silica, calcite or zeolites. These secondary materials are believed in some instances to be responsible for a moderate level of potential alkali reactivity, although this was not recorded in tests on some of the basalt flows identified as possible sources of material for use in the Lesotho Highlands Water Project.[3.58]

Volcanic glass is often a component of basalt. The presence of glass does not indicate a problem other than from the possibility of alkali reactivity. However, partly due to the porous nature of the basalt, much of the glass and many of the ferromagnesian minerals present may have been converted to secondary clays among which montmorillonite and other smectite group minerals are common. These secondary minerals contribute to the poor performance of basalt in concrete, particularly with regard to the effect on the dimensional stability of concrete. Thorough testing of basalt aggregate is essential before its use in concrete may be permitted even if it appears fresh in hand specimens. It should also be pointed out that different samples of basalt taken from the same vicinity can show wide variations in their properties. (See section 3.2.7 for discussion of the ethylene glycol test.)

Ventersdorp Eruptives

Basaltic and andesite lavas comprise the major part of the Ventersdorp Supergroup which covers large areas of the North West Province and Northern Cape Province. These lavas are exploited to the south of Johannesburg for use as concrete aggregates. The Eikenhof Quarry, situated close to Johannesburg, has supplied large quantities of andesite rock both as coarse and fine concrete aggregate. Further deposits have been quarried at Vaal Quarry, Vereeniging, and VaalHarts Quarry near Hartswater, and both of these operations have supplied andesite for use in concrete. The latter source has been recorded as resulting in higher than average concrete shrinkage. None of these andesites appears to be significantly alkali reactive.

The Ventersdorp Volcanics consist mainly of fine- to medium-grained greyish-green andesitic lavas which in places are highly amygdaloidal. Also present are pyroclastics of various types but essentially of andesitic composition.

A significant feature of the rock is its ability, when crushed to fine sizes and used as sand in a concrete mix, to provide an increase in the strength of concrete. It is believed that this is due to the higher crushing strength, coarser surface texture and lower degree of pre-fracture of the andesite when compared with other rocks such as quartzite.[3.59,3.60] The extent of the premium in strength depends on the mix proportions selected and on the size fractions of the fines.

Granite

Under this heading are included crystalline igneous and metamorphic rocks, of differing grain size, consisting essentially of quartz and feldspar together with differing amounts of mica, amphibole and iron oxides. Such rocks are widespread in South Africa.

Weathering effects in granite rocks are manifest mainly in the replacement of the feldspar components by clay minerals, chiefly illite and / or kaolinite. There are indications that the micaceous minerals developed in weathered granite may cause some instability in concrete and use of weathered granite may in some cases be inadvisable, especially if concrete stiffness is of concern.

That granite, in spite of its widespread occurrence, is not more extensively used for concrete in South Africa is due primarily to the fact that other rock types equally suitable for this purpose are often more conveniently situated nearer to points of building activity. Depletion of these readily accessible reserves of rock will probably lead to an increase in the demand for granite as a concrete aggregate. On the Witwatersrand, for instance, the supply of quartzite is diminishing as mines close down and their rock dumps are consumed. To replace the quartzite, engineers are turning more to the andesite south of Johannesburg and to the granite and dolomite to the north of the city.

There is a tendency in areas close to Cape Town towards the use of granite in preference to greywacke where users are concerned about the possibility of alkali aggregate reaction. Sources of this granite are near Mamre and Malmesbury. As some granites are known to exhibit moderate levels of potential alkali aggregate reaction, decisions on selection of granite aggregates from unknown sources should be made only after satisfactory information on the aggregate is obtained.

Granite may sometimes impart a low elastic modulus to concrete probably because of the slightly weathered nature of many deposits, and paste-aggregate interfacial effects. Tests conducted on concrete made with granite from the South Coast of KwaZulu-Natal, from Zululand and from Midrand north of Johannesburg, have given elastic modulus results somewhat lower than average, the modulus ranging from normal down to as much as 20% below average. However, it should be noted that there are many quartzites which, when used in concrete, impart elastic moduli very similar to the lower end of the range of values reported for granite concretes.[3.9]

Kaplan[3.61] has given examples of granite concrete from the UK with elastic moduli well below those of gravel and limestone aggregates. Groupings of rock types under this heading are, however, broad and not all granite concretes will necessarily show low moduli of elasticity.

Greywacke

This rock, which is more commonly known in the Western Cape as Malmesbury Shale, is a greywacke that developed by thermal metamorphism of the initially argillaceous rocks of the Malmesbury Group by the intrusion of the plutons of the Cape Granite and as such its distribution is confined to the vicinity of the granite bodies. The greywacke is generally a fine-grained, "glassy" rock consisting of a mosaic of quartz, feldspar, mica and iron oxides and, sometimes, alumino-silicates among which cordierite is often prominent. From a consideration of the origin of the greywacke it is to be expected that a gradual transition may be expected from crystalline rock suitable for use in concrete to less desirable shaly materials, in a direction at right angles to the granite contact.

Characteristically the greywacke does not readily crush to a good cubical or "chunky"particle shape; consequently the quality of the stone produced varies from poor to fair. But by using appropriate crushing techniques, a considerable improvement in particle shape of the aggregate can be effected.

Fine rock flour generated by the crushing of greywacke may be used to improve the quality of concrete made with Cape Flats sand which is commonly lacking in the finer fractions. The extremely fine rock flour blends well with the dune sand and reduces bleeding of the fresh mix besides improving the surface finish and durability of the hardened concrete. For lean mixes the rock flour may be substituted for up to about 8% of Cape Flats sand with beneficial results. For richer mixes the proportion should be limited to about 4%. Greywacke was the first aggregate to be recognised as exhibiting alkali reactivity in South Africa, with a number of bridges and the N2 freeway road pavement in the Cape Flats area being affected. It is now recognised as one of the more reactive types of aggregate in South Africa.[3.52]

Tillite

The Dwyka Formation, forming the lowermost sediments of the Karoo Supergroup, consists essentially of tillite and shale, with the tillite predominating. The tillite generally comprises a heterogeneous greyish-blue, brown or greenish rock composed of a mixture of various pebbles and boulders cemented by a fine-grained matrix consisting essentially of quartz, feldspars and clays. The tillite represents a vast deposit of glacial debris and among the pebbles and boulders are found an immense variety of rocks derived from pre-Karoo formations. Included in the variety are quartzite, shale, granite, limestone, amygdaloidal lavas, etc.

From the description it will be clear that no general rule can be given for the behaviour of tillite as a concrete aggregate and that the performance of the rock in concrete depends upon the composition of the rock at the locality in which the material is quarried. C&CI experience of the rock as a concrete aggregate has largely been limited to tests conducted on samples from the Durban area although some samples from the southern and northern coasts of KwaZulu-

Natal and from Zululand have also been tested. In Durban the rock has been accepted as a generally satisfactory concrete material, vast quantities having been quarried and sold as a coarse aggregate.

It has been estimated that about 40% of the concrete in Durban is made with tillite, the remainder being made mainly with Natal Group quartzite.

Miscellaneous aggregates

Dolomite

Massive thicknesses of calcareous rocks known as the Chuniespoort Group of the Transvaal Sequence and the Campbell Group of the Griqualand West Sequence are distributed over large areas of the interior of South Africa. These rocks are mainly dolomitic limestone consisting essentially of calcite and dolomite.

The amount of dolomite used as aggregate for concrete work has grown in recent years. The rock plays an important role in prolonging the life of concrete exposed to attack by acids.[3.62] These chemicals attack the cementitious matrix surrounding particles of aggregate in concrete, causing rapid deterioration. If the particles consist of the commonly used siliceous aggregates, the full force of the chemical is concentrated on destroying the hydrated cementitious paste. As the aggregate particles fall away fresh surfaces are exposed to the attack. Dolomite and limestone are themselves soluble in acids. So, when calcareous aggregates are used, the attack is spread over a very much larger area and more of the acid is neutralised, thus reducing the rate of deterioration significantly. The acid-neutralising properties make calcareous aggregates suitable for the manufacture of concrete sewer pipes and have assisted pipe manufacturers in providing an economical means of prolonging the service life of concrete pipes exposed to the problem of sewer corrosion.

The concrete-making properties of calcareous rocks are greatly influenced by their non-carbonate constituents. Chert is characteristically present in the dolomite in large irregular masses or in the form of layers. Being a siliceous material and insoluble in acid, chert must be regarded as an impurity in calcareous aggregates intended for use in corrosion-resisting concrete.

The total quantity of chert, clay and other acid-insoluble residues that may be present in calcareous aggregates is therefore a matter that may be of importance to the engineer and particularly to the manufacturer of concrete pipes. The point is covered in SANS 677:2003 Concrete non-pressure pipes.[3.63]

Dolomite aggregates are notable for the effect they have on the water requirement of concrete. When the material is crushed with appropriate attention to achieving a good particle shape, particularly in the crusher sand, concrete water requirements as low as 175 ℓ/m^3 have been recorded for concrete made with 19-mm stone and having a slump of 75 mm. The high dust contents (minus-75-μm material) resulting from the efforts to improve the shape of the crusher sands appear to have a beneficial influence on the compressive strength of concrete mixes and have been shown not to adversely affect shrinkage, even at dust contents around 30%.

Dolomite concretes also usually exhibit high elastic moduli and low creep and shrinkage.[3.9, 3.10]

It should be noted that dolomite is an aggregate that tends to polish under traffic action with the potential to reduce skid resistance of pavement surfaces. This is not a problem with the use of dolomite coarse aggregate, but care should he taken where skid resistance is of concern to ensure that not more than 60% of dolomite or limestone crusher sand is used in road pavement concrete; the balance of the sand should have a high silica content.

Felsite

This igneous rock is a lava equivalent of granite and comprises a dense, very fine-grained pink to chocolate-brown rock. Its use as a concrete aggregate is confined mainly to the region between Pretoria and Groblersdal and, more frequently, to the areas around Bronkhorstspruit, Witbank and Middelburg. In general, felsite is a tough and durable rock acceptable for use as a concrete aggregate but, characteristically, the shape of the crushed materials is flaky and elongated. Consequently the shape of the aggregate, both coarse and fine, produced by many of the crushing plants in the areas mentioned above may be poor, unless steps have been taken to ensure improved particle shape through the use of appropriate crushing techniques.

A feature of concrete made with felsite stone is the lack of adhesion at early ages between the stone and the matrix. This characteristic, which is probably due to smooth rock surfaces resulting from the fine-grained nature of many felsites, is of no significance in normal structural concrete as the bond soon improves. However, in concrete road construction where joints are usually sawn within the first 24 hours, the early lack of bond is detrimental in that pieces of coarse aggregate may be torn out along the line of the joint during the sawing process – a phenomenon called ravelling.

3.6.2 Fine aggregate

In this section, the nature and characteristics of sands commonly used for concrete manufacture are described, and details are given of the various types of sand used for this purpose in South Africa.

Sand or fine aggregate used in the manufacture of concrete may be described as a non-cohesive granular material consisting mainly of particles that pass the 4,75-mm sieve and are retained on the 0,075-mm sieve. Particles having grain sizes below this range are generally classified as silt (75 to 2 μm) or clay (< 2 μm) and are collectively referred to as "fines." SANS 1083 refers to the minus-75 μm material as "dust" and specifies a hydrometer test method to determine clay content which is defined as minus-5 μm material. The SANS definition of fine aggregate is given in section 3.3.1.

The weathering of rocks produces sands, silts and clays and there are surprisingly few dominant minerals present in most sands. Most commonly, sands are dominated by quartz with feldspar. Mica and iron oxide occur as less abundant phases. Quartz is a hard, chemically stable mineral and as such constitutes the most desirable component of concrete sand. Feldspars are less inert and frequently show signs of decomposition resulting in the formation of clay minerals. Some sands contain small quantities of minerals of high particle density such as garnet, zircon, rutile and ilmenite which do not affect the behaviour of sand in concrete.

The criteria to be considered in selecting sands for concrete are covered in section 3.2.

The characteristics of a sand significantly influence the amount of water required to make a given volume of concrete workable. If the size and type of stone and type of cement used in the concrete, and consistence of the concrete, are standardised, the water requirement for unit volume of concrete is a useful index of the concrete-making properties of the sand.

In order to compare different sands, the concept of standard water requirement (SWR) is used in the following discussion. SWR is the amount of water required to make one cubic metre of concrete that:

- Is made with CEM I 42,5N cement and 19-mm single-size crushed stone;
- Contains an amount of stone as determined in section 11.2.1 Table 11.4;
- Has a water:cement ratio of 0,6; and
- Has a slump of 75 mm.

Units of SWR are ℓ/m^3. The average SWR in South Africa is about 210 ℓ/m^3, with a range from about 175 ℓ/m^3 for "good" sands to 235 ℓ/m^3 for "poor" sands. It is important for users to understand this property of a concrete sand as it is often the reason for one concrete sand being more expensive per ton or cubic metre than another; the effect on the materials cost of a concrete can be significant if a high strength concrete has been specified.

River sands

The materials carried by streams and rivers consist essentially of the products of weathering of the rocks along the course of the river and its tributaries and for the most part comprise quartz, clays and rock fragments in varying stages of decomposition. The relative abundance of these materials is determined in the first instance by the nature of the rock types present in the area drained by the river but is also greatly influenced by the mechanics of transport to which the materials are subjected.

The ability of a river to transport material varies as the sixth power of its velocity.[3.64] This means that a very slight change in the velocity of the current has a considerable effect on the mass of the particles transported. Many South African rivers are characterised by erratic flow and it is found that both grading and mineral composition of streambed deposits may show considerable variation, both laterally and vertically. In general, deposits in the beds of swiftly flowing rivers are deficient in clays and silt-sized particles, which make up the fines fraction of concrete sands. On the other hand, sluggish rivers tend to have an excess of fines in their bedloads.

The shape of the sand grains is dependent on the distance of transport in the river and also on the number of times the grains have been subjected to an erosion and transport cycle. River sands are commonly graded within the range 2 360 to 150 μm, with coarser sands having the bulk of their mass lying on the 2 360- to 300-μm screens and finer river sands generally ranging from about 1 180 to 150 μm.

Both coarse and fine river sands are thus slightly "bunch graded" although not nearly as markedly as beach and sea sands. Because of their somewhat limited grading distribution, river sands are almost always improved by blending with another, generally finer, sand to extend the grading curve.

Sands composed predominantly of well-rounded grains will produce concrete with a lower water requirement than sands containing a large proportion of angular grains. River sands in general produce concrete with a fairly low SWR, and a value of 205 ℓ/m^3 may be regarded as a fair average for most South African river sands. Sands of the Vaal and Mgeni Rivers are deficient in fines and result in SWRs as low as 190 ℓ/m^3. By contrast, river sand from the Gariep (Orange) River contains an abundance of fines, among which clay minerals are prominent, and its use leads to SWRs in excess of 210 ℓ/m^3.

The shrinkage characteristics imparted to concrete or mortar by river sands are influenced by the content and mineralogical composition of their fines. Should the sand contain sufficient swelling clay minerals amongst the other materials in the dust fraction, high shrinkage is to be expected. Swelling clay minerals may also be present in sand-sized particles of weathered basic rock (basalt and gabbro) and here too these minerals would contribute adversely to the performance of the sand.

Beach sands

These may be defined as the sand lying in the area bounded by low and high water marks but including the area covered by storm waves and by spring tides. By far the most abundant component of most beach sands is quartz. As a result of the effective sorting action of the waves, beach sands suffer from grading limitations even more marked than in the case of river sands. Some beach sands contain essentially only two particle size ranges while a few consist almost entirely of particles of a single size range. Any attempt to make concrete with these sands without the addition of a suitable blending material to compensate for the grading deficiencies will result in a harsh mix lacking plasticity and cohesiveness. The mix will also be highly susceptible to bleeding since beach sands are invariably deficient in fine particles.

The abrasive effect of the surf is similar to that of rivers except that the action of waves breaking on a beach may tend to slide material rather than to roll or lift it.[3.65] This results in a tendency for many particles to be more discoidal in shape than river materials. The discoidal shape is generally more noticeable in particles of approximately 10- to 25-mm size than in the smaller sizes below 5 mm. Whether the sand particles are spherical or discoidal, the marked rounding of corners improves the performance of the material in concrete and reduces water requirement.

Beach sands such as those in the vicinity of Durban, with FM lying between 2,0 and 3,0, result in SWRs well below 210 ℓ/m^3, but very fine sands may produce concrete with an SWR in excess of this figure in spite of their superior particle shape. On the other hand, East London beach sands often have an FM of about 1,0 and result in an SWR of about 210 ℓ/m^3.

Wide variations occur in the chloride content of beach sands. In the vicinity of Durban, for instance, the chloride content of 18 samples of beach sand varied from 0,050 to 0,176% with an average of 0,087% (measured as Cl^-). Along other stretches of the coast in the vicinity of East London samples of unwashed beach sand have been found to have chloride contents as low as 0,010%.

A study of the literature dealing with maximum permissible chloride contents in sand used for the manufacture of concrete revealed widely conflicting opinions. This is attributable to the large number of factors influencing the susceptibility to corrosion of steel in concrete. Bird[3.65] has pointed out that it is impossible to give a single value for the critical chloride concentration that would cause active corrosion of reinforcement in all concretes; the rate of oxygen arrival at the steel surface, and thus the critical chloride concentration, will depend on the permeability and the thickness of the cover for a particular concrete.

A Steering Committee of the National Building Research Institute (NBRI) investigating the use of beach sands in Durban recommended that the sand may be used in concrete provided it is washed to reduce the chloride content to a maximum of 0,01% by mass. It may seem that this chloride limit was unnecessarily restrictive bearing in mind that the sand is often used for mass concrete and mortars or plasters where higher percentages are probably acceptable. The committee pointed out, however, that the authorities administering the regulations could not be certain that sand intended for mass concrete might not, in fact, find its way into reinforced concrete. The revised limits in SANS 1083:2006[3.2] permit the use of higher chloride contents, allowing up to 0,08% for non-reinforced concrete.

The salt content of unwashed beach sand varies directly with its sea-water content.[3.66] Proper drainage of the sand is therefore of considerable importance in reducing chloride levels and in keeping the costs of washing sands to a minimum. The removal of chlorides to a level of around 0,01% is not, in practice, unduly difficult. The owners of a sand washing plant in Durban claimed that, with well-designed equipment, concentrations of approximately 0,10% chloride can readily be reduced at reasonable cost to 0,01%.

In Britain, sea sands dredged from the English Channel and used by ready-mixed concrete suppliers are washed to reduce chloride contents to a maximum of 0,01%.

Shell in beach sand seldom presents any problem in the manufacture of concrete.[3.67] Any fragile pieces of shell that have not been eliminated by the action of the surf will generally be removed during the process of washing to reduce chlorides. Sands with high shell contents require more thorough washing to reduce chlorides than those with low shell contents. A possible explanation for this is the retention of salts in porous shell particles.

Environmental pressure has made it very difficult to get legal access to beach sands and it is believed that very little of this type of sand is used these days in South Africa.

It is interesting to note that in spite of the high costs of dredging, approximately 10% of the concrete manufactured in Britain in 1972 was made with sea-dredged aggregates, both fine and coarse.[3.67] It has been realised in that country that future demands for concrete aggregates cannot be met solely from land deposits and that it will be necessary to utilise more fully the sea-bed deposits. (Recourse is also being made to recycled aggregates, and more information on the use of recycled aggregates is given in the Appendix to this chapter.)

Dune sands

Sand dunes are characteristic of low-lying flat coasts and of deserts where sand is blown by wind into the form of heaps or hills.

The transporting power of wind is much less than that of water and the transported particles are thus, in general, much finer than those carried by water. Dunes are generally composed of sand grains having sizes smaller than 1,0 mm but larger than 0,1 mm.[3.68] The dune sands of the Cape Flats fall into the upper portions of this size range but many other dune sands in South Africa have sizes lying in the lower portion of the range. Wind-blown sands near the Gariep (Orange) River at the site of the Vanderkloof Dam, for instance, have about 90% of their mass passing the 300-µm screen and only 5 to 15% passing the 75-µm screen.

Being windborne sediments, dune sands are characterised by well-rounded grains and in this respect are similar and often superior to river sands but differ in that their grading is usually narrower. The sorting power of the wind results in concentrations of material similar in particle shape, relative density, and even mineral composition.[3.64]

The characteristics of dunes near beaches differ significantly and are strongly influenced by marine conditions. Under the normal cycle of action, the seaward faces of dunes erode during severe storms and deposit their sand on the beaches. During periods of good weather, the dunes are reformed by beach sands blown on to them by onshore winds. As would be expected, these changes are accompanied by marked variations in chloride content of the material in coastal dunes. However, tests carried out at various points on the eastern seaboard have shown the chloride content of dune sands to be considerably lower than that of beach sands.[3.69] Typical figures for chloride content (as Cl^-) obtained from tests carried out on the KwaZulu-Natal and Mozambique coasts are 0,090% for beach sand and 0,010% for sand in dunes adjacent to beaches. In a number of cases, the dunes only a short distance above high water level had chloride contents as low as 0,003%. Similar low figures were obtained from tests on dunes in the Richards Bay area.

The coarser dune sands such as those of the Cape Flats are used with and without the addition of blending sands as fine aggregates for concrete. Because of their well-rounded grains, they produce concrete with low to medium SWRs of about 200 ℓ/m³. When used without blending materials their limited grading is often responsible for lack of cohesiveness in concrete mixes. Very fine-grained dune sands are seldom used on their own as fine aggregate but are occasionally used for blending purposes.

Pit sands or quarry sands

Physical and chemical weathering of rocks have created an in-situ mantle consisting of varying proportions of rock fragments, sand, silt and clay in many areas. These materials may be excavated for use in concrete when supplies of other suitable concrete sands are not readily available. As is to be expected, these sands differ widely in character and in their concrete-making properties. They often require processing to remove coarse rock fragments and a proportion of the silt and clay before they are acceptable for the manufacture of concrete.

The suitability of sand from a given source for use in concrete depends on the method by which it is processed and on the degree of control exercised during production. These factors in themselves are responsible for a considerable range of qualities in sand derived from a single source. To illustrate the contrast in materials in this category of miscellaneous natural sands, mention may be made on the one hand of the sands derived from weathered granite and exploited in vast quantities to the north and east of Johannesburg and, on the other hand, of the fine "pit" sands used in Durban as a blending material for most of the concrete made in that city. The former, often sold as "river" sand, is a rather coarse sand consisting mainly of decomposed feldspars with quartz and some clay and flakes of mica, while the latter is an extremely fine sand composed almost entirely of quartz grains with a small quantity of clay and organic matter. With such wide differences in the compositions of these materials it is clearly not possible to provide an adequate description of concrete-making properties that would he applicable to the group as a whole.

Sands of the Karoo

In an extensive test programme carried out by Roper[3.6] at the NBRI, it was demonstrated that coarse and fine aggregates derived from many of the rocks of the Karoo Supergroup are responsible for excessive drying shrinkage of concrete. The NBRI tests indicated that the presence in the aggregate of swelling clay minerals is not a prerequisite for large shrinkage movement of concrete and that clays such as illite and kaolinite with stable lattice spacings were present in many of the rocks and sands responsible for large moisture movements of concrete.

The NBRI tests showed also that the modulus of elasticity of aggregate, thought by some to play an important role in the shrinkage of concrete, plays only a minor part in the shrinkage behaviour of concrete made with Karoo aggregates. The investigation did establish, however, that a fairly good relationship exists between the total surface area of aggregate, measured by water-vapour absorption, and concrete shrinkage. It also established a relationship between shrinkage and the proportion of dark-coloured particles. Roper observed that almost all the Karoo sands, except those derived from the Molteno formation, were responsible for large dimensional changes in mortars and concretes.

From a subsequent study by Davis[3.70] of the results of a large number of shrinkage tests conducted on mortar specimens made with South African sands, it is believed that the problem is less serious than would appear from Roper's observations. While Davis's tests confirm that many Karoo aggregates do give rise to severe shrinkage movements in concrete, they also indicate that aggregates from very large areas of the Karoo Supergroup behave normally with regard to shrinkage.

Sands overlying and originating from Ecca Group sediments are used in many parts of the Transvaal and northern KwaZulu-Natal and perform satisfactorily in concrete. Sands from the Dwyka Tillites and the aggregates derived from the Molteno and Clarens formation sediments exhibit slightly higher shrinkage than the Ecca materials but are, in general, acceptable for concrete manufacture.

Large quantities of sands from the Molteno formation were used in the concrete of the Orange Fish Tunnel. Sand taken from the Caledon River near Wepener and derived probably from Molteno sediments has provided Bloemfontein with much of its concrete sand for many years. In more recent times, this river sand has been blended with dolerite crusher sand for use in concrete mixes.

It appears from the test results that the aggregates mainly responsible for high shrinkage of concrete are the basaltic lavas of the Drakensberg formation and their weathered equivalents and the sediments of the Beaufort Group, the worst offenders in this respect being the shaly pit and river sands overlying and derived from the Beaufort Group sediments.

Care should be taken not to give undue prominence to the role played by shrinking aggregate in concrete. While many of the coarse and fine aggregates from the Beaufort Group exhibit high shrinkage and should be avoided for concrete work, alternative materials, eg dolerite intrusions, may be available and are preferred for use in concrete. Other regions of the Karoo Supergroup may have strata that are relatively free of badly shrinking aggregates.

It is not suggested that the question of shrinking aggregates should he ignored; the engineer should be aware of its existence and of the likely areas of occurrence, ie the area covered by the Karoo Supergroup. The implications of using these materials in concrete, eg excessive deflections, loss of prestress, in which excessive shrinkage could affect the durability or serviceability of the structure, should be understood, but in order to make the best possible use of available materials, it should be acknowledged that aggregates having shrinkage characteristics higher than average may be acceptable for many concrete applications.

Sands containing mica

Some mention should be made of natural sands having high contents (in excess of about 1%) of biotite and muscovite, both of which are common constituents of many rocks. Biotite, often called black mica, is usually dark brown in colour while muscovite, which is also known as white mica, is often transparent and colourless. The influence on concrete of flakes of free mica in sand depends on the size of the flakes. Tests[3.71, 3.72] have indicated that the finer the size of mica flakes in the sand, the greater the effect in reducing the compressive strength of concrete made with such sands.

On the other hand, tests conducted in Italy[3.73] of quartz-diorite containing 18% biotite by mass showed that the presence of mica in aggregate particles of 4 to 5 mm and larger in size had no adverse effect on strength development, owing probably to the fact that the biotite was enclosed within the rock. This is confirmed by South African experience with granite, for instance, where the mica, being well keyed among the other mineral components, causes little or no strength reduction.

In studying the mica content of sand in Namibia, Müller[3.74] has noted that fine sands have a higher mica content than coarse sands. He found the biotite content of a variety of typical Windhoek sands to be inversely proportional to the FM of the sand.

The influence of different types of mica on the behaviour of concrete differs considerably and for this reason it is necessary to judge each type of micaceous aggregate on its own merits. To compare the effect on concrete of blending mica with the fine aggregate, samples of biotite and muscovite were obtained by C&CI and tested in mixes containing varying proportions of each of these micas. The effect of the biotite admixtures on water requirement was negligible but a slightly adverse effect on strength was noted. Sand containing 5% biotite produced a strength loss of approximately 6% in the concrete while 10% biotite was responsible for a loss of 10% in strength. Both compressive strength and flexural strength were affected equally. It should be pointed out that investigations by Hoon and Sharma[3.72] indicated strength reductions two to three times as great as these for both compressive and flexural strengths.

The reason for this is not known but may be attributable to differences in the types of biotite tested.

The effect of muscovite on both water requirement and strength of concrete was very considerable. The water requirement increased by approximately 6 ℓ/m^3 for every 1% of muscovite in the sand. The measured losses in both compressive and flexural strengths were 35% for 5% muscovite in the sand and 60% for a 10% admixture of muscovite.

It is interesting to note that the measured increases in water requirement are not in themselves sufficient to account for the exceptionally high strength losses caused by the muscovite. A factor contributing to the reduction in strength might be low bond between muscovite flakes and cement paste. The bond between cement paste and various materials embedded within it has been examined petrographically by Farran,[3.75] who found that in the case of mica the bond was particularly poor.

The US Bureau of Reclamation[3.76] conducted a series of tests to assess the effect of mica on the shrinkage of concrete. Increases in shrinkage of concrete containing mica were measured but it seemed from the test results that these increases could be secondary effects attributable primarily to the higher water contents of the mica mixes rather than directly to the mica.

Crusher sands

The manufacture of crusher sand from sound parent rock solves the problem of obtaining concrete sands in regions where natural sands are either not available in sufficient quantities or are of unsuitable quality. Many of the characteristics of crusher sand particles are attributable directly to the inherent properties of the rock type from which the sand was produced. The important property of particle shape, however, can in many cases be modified by the crushing plant operator to meet required standards of quality. Elongation and flakiness of the particles can generally be reduced by the adoption of suitable crushing techniques.

On the Witwatersrand, for instance, some of the more modern crushing plants are producing sands with particle shapes superior to those of the weathered granite pit sands which are commonly used natural sands in the Johannesburg area. A further important advantage of crusher sand manufactured under controlled conditions is the uniformity of grading that may he achieved. By comparison, the grading of many natural sands, particularly river sands, differs greatly depending on conditions existing at the time of deposition of the material.

Certain rocks such as schist and slate possess marked cleavage planes and thus have an inherent tendency to fracture into elongated and flaky particles. Rocks of this type are difficult to process successfully but their use can in many cases be avoided since adequate quantities of more suitable rock types are generally available in places where crusher sand is in demand.

Crusher sands, properly produced from sound rock, have the important advantages of good particle shape and uniform grading and are less likely than pit sands to be contaminated by deleterious substances such as clay minerals and organic matter. It is for this reason that many specifications permit higher fines contents in crusher sands than in other sands, but the premise is only acceptable as a general guide since it would not apply to crusher sands derived from certain shale and basic igneous rocks which may contain deleterious substances. It is for this reason that optional additional tests (SANS 6243 and SANS 6244) are required to ensure that deleterious quantities of clay minerals originating from the sands selected are not used in concrete.

While natural sand is still commonly used for concrete in South Africa, the demand for crusher sand has steadily increased. Previously, many of the crusher plants supplied only "crusher dust" as a by-product of their stone crushing operations. In the early days of producing crusher sands, many engineers understandably rejected this waste material for the manufacture of concrete, as it was produced without regard to its particle shape and grading. Currently, a factor contributing to the rise in popularity of crusher sands is the ability of many suppliers to provide a high-quality sand which has good particle shape and grading. Many such sands produce concrete with a water requirement lower than that of concrete made with pit sands. In Johannesburg, for instance, the use of crusher sands results in an SWR of about 205 ℓ/m^3 whereas the value for concretes made with many natural or pit sands available in the area is about 220 ℓ/m^3 or higher. For typical 30-MPa structural concrete the use of the crusher sand could mean a saving of about 30 kg of cement per cubic metre of concrete.

Sand resources in the major centres

Johannesburg and the Reef

While there is a steady supply of quartzite from mining operations in the Witwatersrand area, production levels of this material are now insufficient to meet current demand for crusher sand and stone. The majority of the operating gold mines in the Witwatersrand lie to the west of Johannesburg and into the Free State goldfields. Old stockpiled reserves are dwindling or have been depleted and suppliers have developed open-face quarries to make good the shortfall. At present the bulk of the stone and crusher sand requirements are derived from andesite lava, granite, dolomite, norite and dolerite, roughly in that order of quantities used.

Witwatersrand quartzite has a proven history of performance although there is some variability within deposits. Quartzite to the west of Johannesburg is generally better in quality than that to the east, the latter having somewhat higher shale contents and thus tending to produce poor particle shape on crushing. Further details relating to the use of this rock type are given earlier in section 3.6.1 under the heading *Quartzite of the Witwatersrand Supergroup.*

Andesite lava deposits to the south of Johannesburg are now widely used. Excellent results have been obtained and consistent premiums on compressive strength are reported. Crusher sands derived from this rock previously gave rise to rather harsh mixes, probably due to poor particle shape of the sand, but this property has been improved with modifications to the crushing techniques. The SWR of concretes made with these sands is a little below average at about 200 ℓ/m³.

Granite fine aggregates are available from Midrand and adjacent areas north of Johannesburg in two forms: processed decomposed material and crusher sand.

A problem associated with the winning of the former is the presence of a high proportion of weathered material in the deposits; the sands produce concrete having relatively high SWRs of approximately 220 to 230 ℓ/m³, these high figures being mainly due to the poorly shaped particles and the presence of weathered feldspars. The water requirement of concretes made with these sands is strongly influenced by the dust content, rising sharply where the minus-75-µm content exceeds 5%. The softer weathered feldspar particles in such sands are also inclined to grind finer due to attrition during mixing, a characteristic that leads to cohesive fatty mixes and good surface finishes.

Decomposed granite sands can often be improved by blending with a fine filler sand.

Blending is especially advantageous when too much minus-300-µm material has been washed out of the sand in an attempt to reduce the minus-75-µm fraction.

Granite crusher sands produced from unweathered material yield concrete with SWRs of around 205 ℓ/m³. Blending crusher sand with decomposed granite sand may also be beneficial in cases where the dust content of the decomposed granite sand is high. The shape of the particles is generally poorer than that of the processed decomposed granite sands, and bleeding of concrete tends to be more severe.

Dolomite deposits to the south, north and east of Johannesburg are exploited for their special application in pipe manufacture (see details of this rock type in section 3.6.1) but also compete in the general aggregate market. Dolomite crusher sands produce concrete with a lower-than-average SWR ranging between 175 and 190 ℓ/m³. The low water requirement of concrete made with dolomite sands, particularly those with good particle shape and high dust content, has led to increased use of these materials in conventional concrete in recent times. This is so particularly in the case of higher-strength concrete (in excess of 30 MPa) where the savings in cement content compensate for the generally higher cost of the sand.

A dolerite deposit is being exploited near Nigel on the East Rand. This quarry was previously also able to produce dolomite aggregates although it is understood that the dolomite deposit has been exhausted.

Cape Town

The bulk of the sand used in the Cape Peninsula area comes from sand mines in the southern Cape Flats. The grading of these dune sands is limited mainly to three size fractions: well over 90% of a sample of the average Cape Flats sand would pass the 1 180-µm sieve and be retained on the 150-µm sieve. The low fines content, coupled with the limited range of grading, is responsible for lack of cohesiveness and for bleeding of concrete made with this material.

Cape Flats sands may be classified into shell-free sands and shell-bearing sands. The former, known locally as "silica" sands, may contain 3 to 5% carbonates, are generally either white or dark grey in colour and are superior to the shell-bearing sands; they generally give concrete SWRs below 180 ℓ/m³. This low water requirement is due not so much to the lack of shell fragments but to the superior shape of the quartz particles. The shell-bearing sands are beige coloured and have shell contents of approximately 25 to 30% (measured as carbonate).

It is interesting that few of the Cape Flats concrete sands have shell contents between the nominally shell-free and those containing about 30% carbonates. Soft or friable shell particles are undesirable constituents of a sand but most of the shell content in Cape Flats sand consists of hard particles that have become well rounded by natural attrition. A high proportion of this type of shell is probably acceptable in sands used for most grades of concrete.

A second source of sand is in the Klipheuwel area north of Durbanville. The siliceous pit sands from this area have a very good particle shape and a continuous grading. Although the percentage fines is very high (up to 20% minus-150-µm and 15% minus-75-µm) these sands do not appear to cause high shrinkage. Concrete water requirements are extremely low with SWR being around 170 ℓ/m³. These are excellent concrete sands, but their use in dry-mix applications causes problems with stickiness unless blended with a proportion of dune sand. It should be noted that these sands have a fairly high organic content; some reports of set-retardation of concrete made with such sand have been received. The original sources are becoming depleted and other deposits of this type of sand have been found along the West Coast as far as Saldanha and inland towards Malmesbury.

Crusher sand of poor quality manufactured from greywacke from the Malmesbury Group is obtained from some of the local quarries. However, like other properly controlled crusher sands produced by many of the Reef plants, other greywacke crusher sands have good particle shape and grading. Nevertheless, because of its higher price relative to that of the natural sands, its use in concrete is not considered economical. The crusher sand is used, blended with dune sand, mainly for the manufacture of concrete masonry units.

Rock flour is produced from the crushing of greywacke and is collected either as a dry dust by electrostatic means or as sediment from the wash water. Being extremely fine, the flour is useful for reducing the bleeding of concrete mixes made with Cape Flats sands by blending with the natural sand in proportion from about 4 to 8% by mass of sand, depending on whether the mix is rich or lean. This practice is however not widespread.

Crushed Cape granite is available from quarries near Malmesbury.

Pretoria (Tshwane Metropolitan Area)

Quartzite and dolomite together provide the major portion of the stone and crusher sand requirements in the Pretoria area. The quartzite, known locally as ferro-quartzite, is derived from the Pretoria Group of the Transvaal Sequence. Concrete sands crushed from this rock give SWRs of about 195 to 200 ℓ/m^3.

Dolomite from the south-west and south-east of Pretoria is used to about the same extent as quartzite. Concrete SWRs of conventionally crushed dolomite crusher sands are approximately 175 to 190 ℓ/m^3 but, despite these fairly low figures, concrete made with this sand has a marked tendency to bleed. With rod-milled or impact-crushed dolomite crusher sands, the dust contents are significantly higher but unexpectedly the SWR of concrete tends to be markedly lower than that of concrete made with conventionally crushed dolomite sand, and bleeding is reduced. Many of the dolomite quarries in this area now operate these more sophisticated crushing plants and are able to manufacture better quality sands.

Granite is available from the Midrand area south of Pretoria but is at present exploited in amounts smaller than those of the rock types mentioned above. Norite, although extensively used in the past, currently provides only a minor proportion of the aggregate requirements in Pretoria. Good-quality natural sands are available to the north, north-west and north-east of Pretoria but long hauls limit their usage.

Most fine aggregates in the Pretoria area consist of crusher sands with Magaliesburg and other fine pit sands blended in when necessary.

Durban

Crusher sand is now widely used in Durban, but is commonly blended with river sand sourced either from the Mgeni or Umdloti rivers. Such a blend will compensate for the lack of fines in the river sands and the slightly higher water requirement of the crusher sand.

Beach sands have not been available since the late 1980s.

Bloemfontein

Karoo Dolerite provides this city with its major source of reliable aggregates, both sand and stone, for the manufacture of concrete. Situated as it is on the sediments of the Beaufort Group of the Karoo Supergroup, Bloemfontein is forced to rely on dolerite crusher sand for its fine aggregate since the use of natural sand in this area is often responsible for excessive shrinkage of concrete.

To assist in overcoming this problem and to augment the supply of dolerite, sand is imported from the Caledon River at Wepener about 100 km from Bloemfontein, but this is not a satisfactory solution. In addition to the high cost of transporting material there is the disadvantage that this sand is itself not entirely reliable from the point of view of shrinkage. Another natural sand sometimes used in Bloemfontein for concrete work comes from the Vaal River, which is roughly 180 km from the city.

The possibility has been considered of exploiting Beaufort Group sandstone at Mazelspoort, about 20 km north of Bloemfontein.

Free State Goldfields

This area extends from Allanridge in the north to Virginia in the south. The stone used is almost exclusively quartzite waste rock obtained from the mine dumps. The shape characteristics of the crushed rock are generally good.

There is a lack of suitable natural sands in the area. The main source of river sand is that from the Sand River, but this material is not suitable for use on its own because of the presence of shrinking material from the Karoo Supergroup. The tendency for higher-than-normal shrinkage caused by this sand can be reduced by blending it with sound crusher sand, usually in the ratio 1:1. The most abundantly used fine aggregates in the area are quartzite crusher sands. These sands perform satisfactorily when blended with suitable local plaster sands.

Port Elizabeth (Nelson Mandela Metropolitan Area)

All the crusher sand manufactured in the Port Elizabeth area is derived from quartzite of the Table Mountain Group of the Cape Supergroup. To improve the workability of mixes made with crusher sand, the sand is blended with a wind-blown dune sand of which there is a plentiful supply. The blended sand is generally used by the local ready-mixed concrete industry which supplies a large proportion of the total concrete requirements in the Port Elizabeth area. The sources of good quality natural sand in the city are limited, being confined to alluvial deposits situated on the banks of the Swartkops River between Perseverence and Despatch.

East London

Most of the fine aggregate used in the East London area is a crusher sand manufactured from fine-grained, thermally metamorphozed quartz-feldspar sandstone of the Adelaide Subgroup. A sound form of this rock, known locally as "arkosic quartzite" and which has a distinctive bluish colour is quarried at Mdantsane and King William's Town. This rock is overlaid by a brownish coloured, weathered sandstone called "lithic quartzite." As the latter is unsuitable for use in important concrete because of its drying-shrinkage characteristics, it is mainly used as base course material in road construction. Deliveries of concrete sand that contain some brown material are acceptable but if the brown colour predominates, individual particles should be carefully examined. The fresh blue particles may have one brown facet while the less acceptable weathered material is more uniform in colour.

Dolerite crusher sand is also available from Queenstown and from a quarry situated between King William's Town and East London. These crusher sands are generally blended with natural sand for economic reasons as well as to improve the properties of the concrete.

Although large deposits exist, beach and dune sands are no longer freely available due to embargoes placed on their removal by various government departments. Sources of natural sand presently available are a river sand from Kidd's Beach and pit sands from the same and a few other sources. These sands are usually very fine with FM of less than 1,0, and the proportion of minus-75-µm material ranges from less than 1% up to 25% depending on the source and method of recovery. However, most of these sands are deficient in fines.

Conclusion

Quartz is the predominant constituent of most natural sands and because of its inert chemical nature and resistance to weathering is probably the most desirable component of concrete sand. While it is important that sands should be hard, durable and chemically stable, several other factors contribute significantly to the quality and effectiveness of sands used for the manufacture of concrete. These factors are shape, surface texture and grading of the sand particles.

Natural sand in the form of river, beach and dune sands as well as a variety of miscellaneous materials commonly known as pit sands account for by far the greatest proportion of fine aggregate used for the manufacture of concrete in South Africa. In a number of the bigger towns, however, the available supplies of these materials cannot meet the sand requirements and sand is then crushed from sound parent rock to augment the supply of fine aggregate.

As the readily available aggregates become depleted, further investigations will bring to light suitable materials to replace them. The utilisation of artificial and synthetic aggregates derived from waste products (see Appendix) should be encouraged.

3.7 The effect of crushing techniques on aggregate properties

A number of different methods have been developed over the years for producing crushed rock suitable for use in concrete. Experience has shown that the equipment used plays a significant role but that crushing technique combined with material factors also have an effect on the quality of the end product.[3.77- 3.80]

Common types of crushing equipment are as follows:

- Jaw crusher
- Roll crusher
- Gyratory crusher
- Cone crusher
- Rod mill
- Impact-type crusher such as horizontal impactor (eg hammer mill) or vertical impactor

There is a general trend towards better particle shape and greater generation of fines with the use of the more modern equipment listed lower down the above list. The capital and operating costs would probably also tend to be higher.

The crushing techniques recommended for the production of well shaped concrete aggregates include using small reduction ratios between one crushing stage and the next, and choke feeding of crushers to prevent splintering.

The parent rock properties that have an influence on the quality of the final product include the following:

- Hardness
- Toughness
- Structure, which may be fine or coarse grained or be laminar
- Moisture content
- Homogeneity

Appendix

Aggregates derived from waste materials

Introduction

Aggregates derived from natural sources make up the bulk of the aggregates used in the manufacture of concrete in South Africa. Indeed, the title of the SA National standard for concrete aggregates SANS 1083, *Aggregates from natural sources – Aggregates for concrete*[3.2], indicates that its criteria only apply to non-synthetic aggregates. However a number of alternative materials have been exploited for use as aggregates. It should be noted that in the developed countries there is considerable pressure to reduce the amount of waste material that has to be disposed of by dumping in landfills. This pressure has led to a proliferation of studies into the feasibility of using some of these materials as concrete aggregates, with the result that most of the recently published research reports on aggregates have focused on the utilisation of waste or recycled aggregates.

It is likely that pressure on companies to minimise their impact on the environment will increase. This will happen sooner for companies wishing to export as their customers are likely to insist on their suppliers being listed as ISO 9000 and ISO 14000 companies, ie companies having quality and environmental management systems. This will increase the need to find constructive ways of utilising waste materials and lead to increasing pressure to use these wastes in concrete.

As stated above, SANS 1083:2006 applies to aggregates from natural sources and does not cover aggregates made from waste materials. Nevertheless, many of the values specified might, for want of any other relevant SA standard, be applied to aggregates from waste materials.

Materials used instead of aggregates derived from natural sources include:

- Processed waste materials such as metallurgical slags, building and construction rubble, sawdust, wood-wool, rice husks, incinerator residues and clinker ash
- Artificially manufactured aggregates such as bloated clay, shale or slate, colliery spoil, glass sintered fly ash, perlite, exfoliated vermiculite and expanded polystyrene

This appendix deals with metallurgical slags, grog, recycled concrete and glass. Low-density aggregates are discussed in Chapter 19.

Metallurgical slags

Slag aggregates are derived from waste products of metallurgical processes.

Most slag aggregates may contain quicklime or burnt dolomite nodules originating from the limestone or dolomite used for fluxing purposes in metallurgical smelting processes; if these nodules have not hydrated before the slag aggregate is used in concrete, the subsequent swelling resulting from the hydration of the nodule in hardened concrete can result in popping. Corroding metals from slags may also cause popping, disfigurement or staining of concrete. Generally speaking, however, slags complying with the requirements of BS EN 1260:2002[3.81] should prove suitable for use as aggregates for concrete.

South African metallurgical slags that have been found suitable for use in concrete include blastfurnace, ferromanganese, ferrosilicon-manganese, phosphate, charge chrome, copper and platinum slags.[3.82] It is possible to make good concrete with sound air-cooled metallurgical slag, either as coarse or fine aggregate. Mixes made using such slags as coarse aggregate may tend to be harsher than mixes made using conventional coarse aggregate, but, by skilful mix proportioning, it should be possible to make concrete with the same placeability as concrete made with conventional aggregate. Concretes made with air-cooled slag aggregate normally have strengths and permeabilities that compare favourably with mixes made with conventional aggregates.

Air-cooled slag sands tend to produce mixes that are harsher than those made with natural or conventional crusher sands. Some slag sands, because of their friability, may produce weaker concretes than those made with conventional aggregates. Reports have been received of extremely high levels of wear on mixing, transporting and placing equipment experienced when slag aggregates have been used in concrete.

In South Africa, air-cooled blastfurnace slag has been used extensively and successfully as a concrete aggregate. The aggregate is usually a light grey to white colour with a rather vesicular structure. The more vesicular particles tend to have a lower particle density than the less vesicular particles.

C&CI has monitored the volumetric stability of concrete made with such slag aggregate in wetting and drying cycles over a substantial period. It was found that the drying shrinkage and wetting expansions tend to be of higher magnitude than for concretes made from most conventional aggregates; in some cases the wetting expansion was found to exceed the drying shrinkage, giving the concrete a net expansion.

Typically, other slags would produce aggregates of higher particle density even though some of the particles might also be vesicular. Some slag aggregates may have relative densities that are significantly higher than many conventional aggregates from natural sources. This could result in segregation problems when transporting fresh concrete made with such aggregates.

According to Davis,[3.83] the variation in the structure of particles of coarse aggregate may cause changes in the relative density of different batches of aggregate. This in turn can lead to inaccuracies in batching when mass batching is used, but does not affect volume hatching. Davis also states that the more porous particles of slag tend to promote segregation in freshly placed concrete, the light, porous particles moving upwards and accumulating at the top of the mix. It has also been noted that some of these low-density vesicular particles may rise to be slightly proud of a finished horizontal concrete surface and this phenomenon may result in disputes over the quality of finish.

The porous nature of some slag aggregate may increase the likelihood of corrosion of steel reinforcement in slag concrete, especially if such aggregate should lie between the steel and the concrete surface, thus reducing the effective cover. The possibility of corrosion of steel reinforcement due to sulphur in slag has also been mentioned, but according to a report by Committee 201 of the American Concrete Institute[3.84] there has never been a case where blastfurnace slag concrete has been shown to have affected the corrosion of reinforcing steel. Everett and Gutt[3.85] have come to a similar conclusion from laboratory experiments they conducted. This should also be true for the other slags mentioned above.

It is possible that in the future, the cost of dumping slags may become more significant as compliance with ISO 14000 becomes more desirable. If, at the same time, the environmental cost of new quarry operations increases, waste utilisation would become more economically attractive. It is not inconceivable that there will come a time when it is feasible to set up a synthetic low-density aggregate plant based on slag or fly ash materials. The technology of such processes has been well researched[3.86, 3.87] to the extent that the use of such aggregate is accommodated in a British Standard, namely BS 3797.[3.88]

Grog (crushed burnt clay bricks)

Aggregate made by crushing broken bricks at brick yards is being used more and more in South Africa, mainly for making precast concrete panels and concrete masonry units.

Work undertaken by the NBRI showed that well burnt clay bricks produce suitable aggregate for use in concrete. However, the following factors should be noted.

- Bricks may contain soluble salts, often in the form of sulphates, which may cause expansion of concrete as a result of reaction with the aluminates in the portland cement or may cause efflorescence on concrete surfaces.
- Grog may undergo irreversible expansion as a result of moisture adsorption (moisture expansion). The NBRI has shown that concrete made with aggregate derived from bricks that exhibit moisture expansion will expand similarly. Bricks suffering from excessive moisture expansion should not be used as aggregate for concrete.
- Being a porous material, there is a similar risk of corrosion of steel reinforcement to that mentioned for porous slag and similar porous materials. Sulphates may also promote corrosion.

The density of crushed-brick-aggregate concretes ranges between 1 840 and 2 160 kg/m^3.

Winkler[3.89] found a pozzolanic effect of using fine crushed brick material. Simpson[3.90] found that the economic problem of obtaining a consistent quality of recycled aggregate was a much harder problem to solve than the technical problems of the use of this type of material. Mansur[3.91] reported that crushed brick aggregates in concrete gave higher tensile strength, lower shrinkage and similar creep but much lower elastic modulus than conventional aggregates.

Recycled concrete

Depletion of normal aggregate sources and waste disposal problems in certain localities have made concrete reclaimed from the demolition of concrete structures an attractive proposition as a source of aggregate.

In the recent past there have been many published reports on the use of recycled concrete as aggregate. Many of the reports suggest that the quantity and quality of adhering mortar on the aggregate plays a significant role in determining the properties of the new concrete.[3.92-3.94]

Useful data have been published by Buck[3.95] on the use of reclaimed concrete as aggregate. He came to the following conclusions:

- The aggregates produced had lower relative density and higher absorption than is typical of conventional aggregates.
- Aggregate derived from crushed concrete having compressive strength less than 14 MPa need not have a detrimental effect on the strength of the new concrete in which it is used.
- The use of crushed concrete as aggregate imposed no problems regarding workability.
- The compressive strength of concrete containing recycled concrete aggregate is lower by as much as 8 MPa than that of concrete of the same water: cement ratio and similar conventional aggregates. Equal strengths can however be obtained by using appropriately reduced water:cement ratios.
- The frost resistance of concrete containing recycled concrete, where chert gravel was the original aggregate, was increased by a factor of five or more in laboratory freezing-and-thawing tests. This is assumed to be the result of a reduction in frost susceptibility of the porous coarse aggregate particles.
- Inorganic, nonmetallic residues from demolition of buildings may be recycled for use as concrete aggregate if the sulphate content is controlled to prevent deleterious expansion due to sulphate reaction.

However, in a recent study undertaken on behalf of a concrete product manufacturer, it was shown that where the recycled concrete aggregate was derived from damaged, but otherwise uniformly high quality concrete precast units, the quality of this aggregate was not noticeably inferior to virgin crushed stone material.

Glass

Waste glass is a potential aggregate for concrete and has been used in precast concrete elements, mostly to produce concrete with a glittering surface. In general, the strength of concrete made with waste glass is lower than that made with gravel.[3.96] Ohlheiser[3.97] reported on the use of

12,5-mm size crushed glass as the aggregate in a flowable fill concrete.

Waste glass is susceptible to alkali-aggregate reaction and may produce concrete with a high expansion if used in combination with a high-alkali cement.[3.98]

Incinerator and other wastes

In the ongoing attempts to minimise wasteful dumping some researchers have investigated the use of incinerator waste as a potential source of aggregate.[3.99] It was found that the use of the waste resulted in an expansive concrete unless the aggregate was pre-treated with sodium hydroxide.

Al-Manaseer[3.100] reported on research into use of shredded plastic car bumpers as aggregate in concrete where it was found that while the compressive strengths obtained were acceptable, the bulk density was reduced, and ductility was increased relative to comparative properties with conventional aggregates in concrete.

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Chapter 4

Mixing water

John Goodman

4.1 Introduction

The quality of the mixing water in concrete can influence the setting time and the strength development of concrete, as well as the protection provided to any reinforcement against corrosion. This chapter deals with the effects of the quality of the mixing water on concrete.

In general the quality of the mixing water has relatively little effect on the final concrete, and is seldom a source of problems.

The information in this chapter is presented in two sections: the first gives practical information on assessing the suitability of a water sample for use in concrete; the second provides more detailed information on common impurities that influence concrete, and discusses the use of natural waters, sea water and some waste waters.

4.2 Practical guidelines

4.2.1 General

Bond completed a geo-chemical survey of the ground waters of South Africa and concluded, "it would appear that practically all natural waters in the Union are suitable (for mixing water), with the exception perhaps of a few of the very highly mineralised waters found in the North West Cape and South West Africa, in which the concentration of dissolved salts exceeds 1 per cent."[4.1] Another exception would be river water in sugar farming areas where the water could periodically be contaminated with sugar from cane processing activities.

It is commonly said that if water is potable, it is suitable for concrete. This is true of municipal waters, but it may be an onerous requirement for mixing water – water that is not potable may still be satisfactory for making concrete, but in such cases it is probably important that the quality and chemical composition should remain constant to avoid unexpected effects on the concrete.

It should be noted that a turbid water should be allowed to settle before use to remove suspended solids which could have unexpected effects.

Evaluation of water of unknown quality may be carried out in accordance with the methodology in BS EN 1008[4.2].

4.2.2 Assessing the suitability of water for mixing water for concrete

This section is a summary of the content of BS EN 1008:2002 *"Mixing water for concrete – Specification for sampling, testing and assessing the suitability of water, including water recovered from processes in the concrete industry, as mixing water for concrete"*.[4.2] Test methods for the various determinations required are given in BS EN 1008.

When assessing the suitability of water of unknown quality for the production of concrete, both the composition of the water and the application of the concrete to be produced should be considered.

Classification of types of water

In general the suitability of water for the production of concrete depends on its origin. The following types may be distinguished:

Potable water

This water is considered to be suitable for use in concrete. Such water needs no testing.

Water recovered from processes in the concrete industry

This water will normally be suitable for use in concrete, but shall conform to the requirements of Annex A of BS EN 1008.[4.2]

Water from underground sources

This water may be suitable for use in concrete, but shall be tested and assessed in accordance with the requirements of BS EN 1008.[4.2]

Natural surface water and industrial waste water

This water may be suitable for use in concrete, but shall be tested and assessed in accordance with the requirements of BS EN 1008.[4.2]

Sea water or brackish water

This water may be used for concrete without reinforcement or other embedded metal, but is in general not suitable for the production of reinforced or prestressed concrete. For concrete with steel reinforcement or embedded metal, the permitted total chloride content in the concrete is the determining factor.

Sewage water

This water is not suitable for use in concrete.

Requirements

Water for use in concrete shall conform to the requirements, given later, for preliminary assessment and for chloride, sulphate and alkali contents. The water shall also conform to either the chemical requirements for harmful contamination, or the requirements for setting time and compressive strength, both given later.

Water supplied as potable water is deemed to conform to the requirements of this standard.

When waters are to be combined, the requirements apply to the combined water.

Preliminary assessment

The water shall be examined in accordance with the test procedures given in BS EN 1008.[4.2] Water not conforming to one or more of the requirements in Table 4.1 may be used only if it meets the requirements for setting time and strength, see sub-sections "Setting time" and "Strength".

Chloride content

The chloride content of the water, expressed as Cl^-, shall not exceed the levels given in Table 4.2, unless it can be shown that the chloride content of the concrete will not exceed the maximum value for the specified class selected from 5.2.7 of EN 206 -1:2000.[4.3]

Sulphate content

The sulphate content of the water, expressed as SO_4^{2-} shall not exceed 2 000 mg/ℓ.

Alkali content

If alkali-reactive aggregates are expected to be used in the concrete, the water shall be tested for its alkali content. The equivalent sodium oxide content of the water shall not normally exceed 1 500 mg/ℓ. If this limit is exceeded, the water may be used only if it can be shown that actions have been taken to prevent deleterious alkali-silica reactions. See Chapter 10 for guidance on minimising the risk of deleterious alkali-silica reactions.

Harmful contamination

In the first instance, qualitative tests for sugars, phosphates, nitrates, lead and zinc may be carried out. If the qualitative tests are not carried out, or they show a positive result, either the quantity of the substance concerned shall be determined or tests for setting time and compressive strength shall be performed.

If chemical analysis is chosen, the water shall conform to the limits given in Table 4.3.

Setting time

The initial setting time obtained on specimens made with the unknown water shall be not less than one hour and not differ

Table 4.1: Requirements for preliminary inspection of mixing water

Item	Requirement
Oils and fats	No more than visible traces.
Detergents	Any foam should disappear within 2 minutes.
Colour	Water not recovered from processes in the concrete industry: The colour shall be assessed qualitatively as pale yellow or paler.
Suspended matter	Water from processes in the concrete industry: see BS EN 1008, A.4.
	Water from other sources: Maximum 4 m ℓ sediment.
Odour	Water from processes in the concrete industry: No smell, except the odour allowed for potable water and a slight smell of cement and where blastfurnace slag is present in the water, a slight smell of hydrogen sulphide.
	Water from other sources: No smell, except the odour allowed for potable water. No smell of hydrogen sulphide after addition of hydrochloric acid.
Acids	pH greater than or equal to 4
Humic matter	The colour shall be assessed qualitatively as yellowish brown or paler, after addition of NaOH.

Table 4.2: Maximum chloride content of mixing water

End use of concrete	Maximum chloride content, mg/ℓ
Prestressed concrete or grout	500
Concrete with reinforcement or embedded metal	1 000
Concrete without reinforcement or embedded metal	4 500

Table 4.3: Requirements for harmful substances

Substance	Maximum content, mg/ℓ
Sugars	100
Phosphates; expressed as P_2O_5	100
Nitrates; expressed as NO_3^-	500
Lead; expressed as Pb^{2+}	100
Zinc; expressed as Zn^{2+}	100

by more than 25% from the initial setting time obtained on specimens made with distilled or de-ionised water. The final setting time shall not exceed 12 hours and not differ by more than 25% from the final setting time obtained on specimens made with distilled or de-ionised water.

Strength

The mean compressive strength at seven days of the concrete or mortar specimens, prepared with the water, shall be at least 90% of the mean compressive strength of corresponding specimens prepared with distilled or de-ionised water.

Sampling

A sample of water of not less than 5 litres shall be taken. The sample shall be correctly identified and representative of the water to be used, due regard being given to the possible effects of seasonal fluctuations.

The sample shall be stored in a clean and sealed container. The container shall be rinsed out with water from the source prior to filling to capacity with the water sample.

The water shall be tested within two weeks of sampling.

Other

BS EN 1008[4.2] also gives test methods for the tests required, applicable frequencies for testing and detailed requirements for the use of water recovered from processes in the concrete industry.

4.3 Additional information

This section gives more details on the effects on concrete and reinforcement of substances that could be found in mixing water. The use of waters from various sources is also discussed.

4.3.1 General

For an average structural concrete the water content is normally only about 8% of the total mass. Thus, in the concrete, the proportion of impurities derived from the water is likely to be low. For instance, 2 000 mg/ℓ of total dissolved solids (TDS) in a water equates to about 400 g of material per cubic metre of concrete, compared with 300 to 400 kg of cement. Whilst this suggests a low proportion of impurity relative to the mass of cement, different documents give different limits for impurities, usually as a proportion of the mass of the water.

The literature has relatively little to say on the topic of mixing water, with some classic (and old) references 4.4 and 4.5 being cited. The most often quoted is that of Duff Abrams[4.4] who conducted a test programme on a large number of specimens and test variables. The conclusions of these references are that, in general, the mixing water quality has relatively little effect on the concrete quality. The fact that so little work has been carried out on this topic also indicates that mixing water quality has generally not been a serious problem.

SANS 10100-2:1992 [4.6] states, in clause 4.2.1: "Water shall be clean and free from injurious amounts of acids, alkalis, chlorides, organic matter and other substances that could impair the strength or durability of concrete or metal embedded in the concrete. (It should be noted that sea water contains injurious amounts of chlorides and alkalis.)"

Neville[4.7] discusses the topic of mixing water in depth; he points out that much of the research upon which our current understanding is based was done 50 to 90 years ago and was based on cements and mixtures of the day. Those cements were substantially different from modern portland cements in their chemical composition, fineness and setting characteristics.[4.7]

4.3.2 Chlorides

Concern over high chloride content in mixing water is mainly due to the possible adverse effect of chloride ions on the corrosion of reinforcing steel or prestressing strands.

SANS 10100-2:1992 [4.6] states:

"To minimise the chloride content in reinforced or prestressed concrete:

- The chloride content of the mixing water shall not exceed 500 mg/ℓ (sea water shall not be permitted as mixing water);
- Calcium chloride and chemical admixtures that contain chlorides shall not be permitted …"

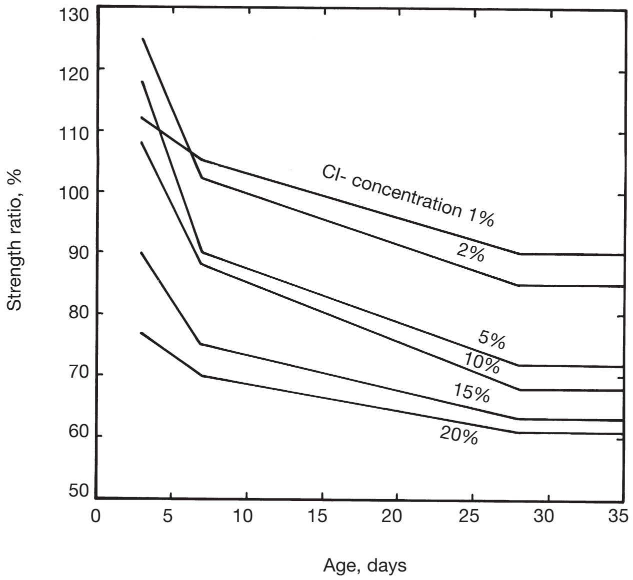

Figure 4.1: Effect of chloride concentration in water on compressive strength expressed as a percentage of compressive strength with no chloride.[4.4]

Abrams[4.4] showed that chlorides in mixing water do not deleteriously affect the hydration of cement, unless in concentrations higher than 20 000 mg/ℓ (2%). At higher concentrations, the long-term strength can be reduced by as much as 30%; for concentrations above 10% the early strength can be markedly affected (see Fig 4.1, prepared from Abrams' data).[4.4]

4.3.3 Sulphates

Sulphates can react with the constituents of hardened cement paste, causing expansion within the concrete and resultant softening and spalling. The severity of sulphate attack depends on the concentration of sulphates and an upper limit of 2 000 mg/ℓ (of water) (as SO_4) is given in BS EN 1008[4.2] to keep the concentration within reasonable limits.

SANS 10100-2[4.6] limits the total water-soluble sulphate content of the concrete mix (as SO_3) to a maximum of 4% by mass of the cement for overall concrete durability. C&CI suggests that this is done on the basis of total cementitious material.

Abrams states that 10 000 mg/ℓ was the limit above which strengths were reduced to less than 85% of control. He also reported that 5 000 mg/ℓ gave a loss of strength of 4%. [4.4]

4.3.4 Alkali carbonates and bicarbonates

Kosmatka, et al[4.8] state, "Carbonates and bicarbonates of sodium and potassium have different effects on the setting times of different cements. Sodium carbonate can cause very rapid setting, bicarbonates can either accelerate or retard the set. In large concentrations, these salts can materially reduce concrete strength. When the sum of the dissolved salts exceeds 1 000 ppm, tests for their effect on setting time and 28-day strength should be made. The possibility of aggravated alkali-aggregate reactions should also be considered."

4.3.5 0il

Kosmatka, et al[4.8] state, "Mineral oil (petroleum) not mixed with animal or vegetable oils probably has less effect on strength development than other oils. However, mineral oil in concentrations greater than 2,5% by mass of cement may reduce strength by more than 20%." It should be noted that oil can be floated off water relatively easily, and this is recommended in order to minimise possible adverse effects.

4.3.6 Sugar

Kosmatka, et al[4.8] state, "Small amounts of sucrose, as little as 0,03% to 0,15% by mass of cement, usually retard the setting of cement. The upper limit of this range varies with different cements. The seven-day strength may be reduced while the 28-day strength may be improved. Sugar in quantities of 0,25% or more by mass of cement may cause rapid setting and a substantial reduction in 28-day strength. Each type of sugar influences setting time and strength differently. Less than 500 ppm of sugar in mix water generally has no adverse effect on strength, but if the concentration exceeds this amount, tests for setting time and strength should be made."

4.3.7 Algae

Water containing algae is often unsuited for making concrete because the algae can cause a reduction in strength. Algae in water lead to lower strengths either by influencing cement hydration or by causing a large amount of air to be entrained in the concrete.[4.8] McCoy reported that concrete made with 0,1% algae in the mixing water entrained 6% air and had a resultant loss in strength.[4.9] A maximum algae content of 1 000 ppm is recommended.[4.8]

4.3.8 pH

Limits given in the literature for the pH of mixing water range from 4 to 6, up to 8 to 8,5.[4.9, 4.10]

It should also be noted that the high alkalinity of the pore water during hydration should neutralise any acidity in the mix water, and be little affected by alkalis in the water. However the total alkali content of the concrete should be determined, including that of the mixing water, if the aggregate is at risk of alkali-silica reaction.

Organic acids should be avoided because their presence may affect the stability of the concrete.[4.11]

4.3.9 Natural waters of South Africa

This section is a summary of reference 4.1.

Bond completed a geo-chemical survey of the ground waters of South Africa and compiled a water-map to illustrate the distribution of different types of water. Figure 4.2 is based on Bond's 1945 map.

Pure water group

In these waters the total dissolved solid content does not exceed 150 ppm, and generally averages about 40 to 100 ppm.

Temporary hard carbonate waters

In these waters the total solids rarely exceed 600 ppm and the pH varies from 7,5 to 8,0.

Alkaline sodium carbonate waters

The total salts rarely exceed 800 ppm and the pH is high and occasionally exceeds 8,8.

Highly mineralised chloride-sulphate waters

Usually the dissolved solids content varies from 1 000 to 4 000 ppm and the pH is usually about 7. These waters contain abundant chlorides and sulphates. At least half of the South Africa has waters of this brackish type.

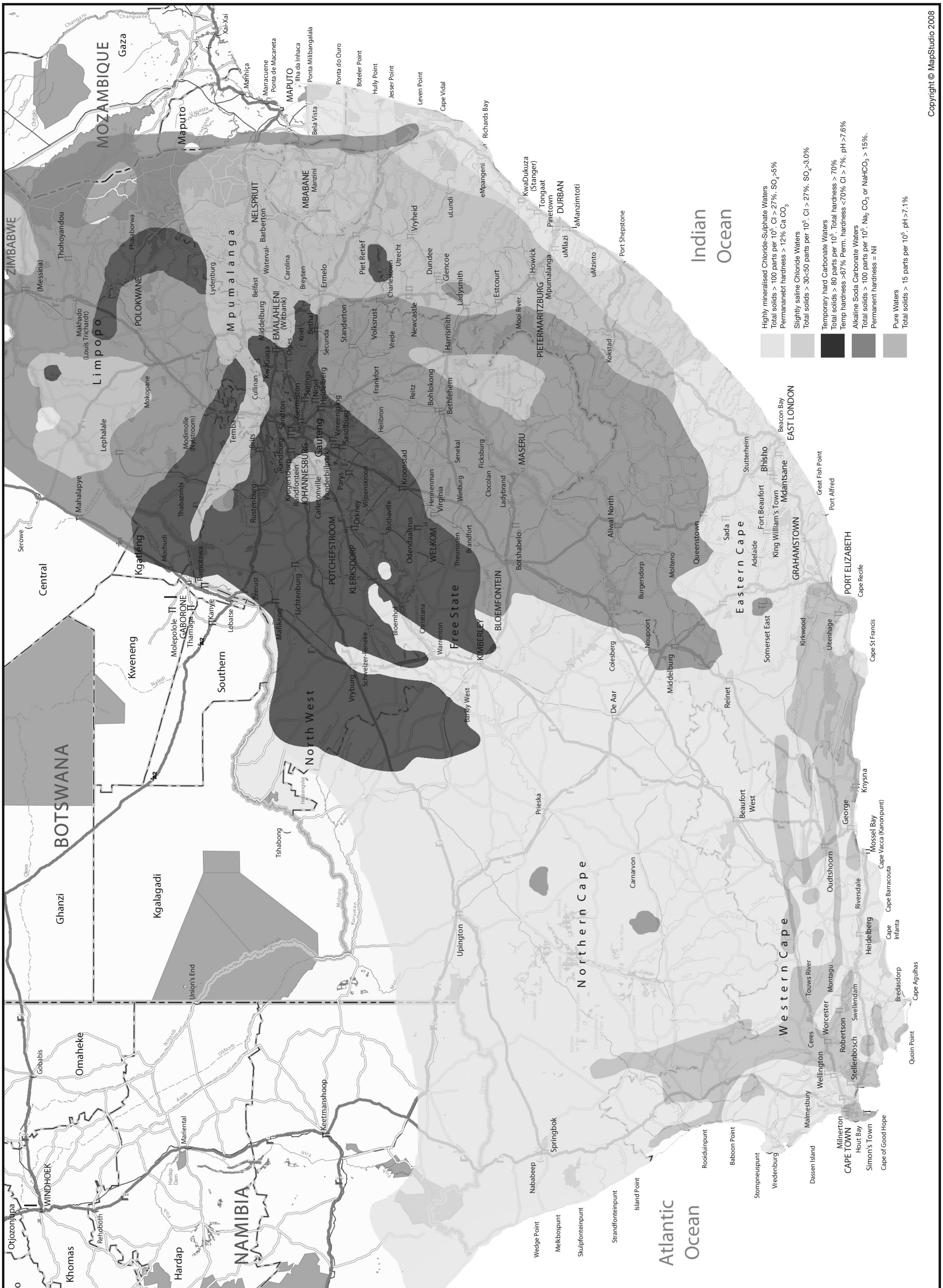

Figure 4.2: Natural waters of South Africa, adapted from a map compiled by G.W. Bond, 1943, and drawn in 1945[4.1]

As stated previously, Bond concluded, "it would appear that practically all natural waters in the Union are suitable (for mixing water), with the exception perhaps of a few of the very highly mineralised waters found in the North West Cape and South West Africa, in which the concentration of dissolved salts exceeds 1 per cent." (1% is the equivalent of 10 000 ppm.)

4.3.10 Wash water

In the past, when mixers or agitators were washed out, wash water may have been disposed of to waste. The National Water Act, Act 36/1998, requires that land owners, occupiers or users, must take all reasonable measures to prevent pollution of a water resource occurring as a result of activities or processes performed or undertaken on land that they own, occupy or use. The act gives limits for properties of waste water that may drain into a river.[4.12] Because of these requirements, wash waters from the concrete industry will have to be re-used. Such water can generally be used as mixing water, especially if blended with water known to be satisfactory. The suitability of these waters, or combinations, can be assessed using the procedures given in BS EN 1008.[4.2]

4.3.11 Sea water

Sea water can contain up to approximately 35 000 ppm of dissolved salts of which about 78% is sodium chloride.[4.8]

Concretes made in South Africa with sea water have shown higher strengths and similar workability, heat of hydration and surface permeability upon exposure to inadequate curing when compared with equivalent concretes made with tap water.[4.13] Work reported elsewhere has yielded contradictory strength results with both higher and lower strengths reported.[4.9]

The effects of using sea water as mixing water in concrete include:

- A very high risk of corrosion of any steel reinforcement[4.13]
- Possible efflorescence and dampness on concrete surfaces[4.8]
- Increased risk of alkali-aggregate reaction, if reactive aggregates are used, and sulphate attack

Kosmatka et al[4.8] state that sea water is not suitable for use in making concrete that is reinforced with steel reinforcement and that it should not be used in prestressed concrete due to the risk of corrosion of the steel.

Neville[4.7] recommends that sea water or brackish water should not be used as mixing water for reinforced concrete, but states that this rule would not apply to plain concrete without any steel inserts.

Pierce[4.11] states that sea water should definitely not be used for making prestressed concrete.

Sea water has chloride and sulphate contents that exceed the limits in BS EN 1008[4.2] and therefore should not be used for mixing water for reinforced or prestressed concrete unless appropriate measures are taken against the applicable effects listed above.

SANS 10100-2 [4.6] states that sea water should not be used in concrete covered by that code.

4.3.12 Domestic waste water

So far, there is very little experimental evidence on the use of biologically treated domestic waste water as mixing water.[4.7] Cebeci and Saatci showed in laboratory tests that such use is feasible[4.14] It would probably be important that the quality and chemical composition of such water should remain constant to avoid unexpected effects on concrete.

4.3.13 Waste waters

The effects of industrial waste waters are entirely dependent on the types and concentrations of contaminants in the waters. The suitability of these waters can be assessed using the procedures given in BS EN 1008.[4.2] It would probably be important that the quality and chemical composition of such water should remain constant to avoid unexpected effects on concrete.

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4.5 Steinour, H.H. Concrete mix water - how impure can it be? **Journal of the Portland Cement Association Research and Development Laboratories,** vol.2, no.3, Sept. 1960, pp.32-50.

4.6 SANS 10100-2:1992, **Code of practice for the structural use of concrete. Part 2: Materials and execution of work**, Pretoria: South African Bureau of Standards, 1992.

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4.10 Editorial notes: Water for making concrete, **Concrete and Constructional Engineering**, vol.42, no.10, Oct. 1947, pp.295-296.

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4.13 Sephton, S.S. and Ballim, Y. A preliminary investigation into the effects of using sea water as mixing water on the properties of concrete, **Concrete Beton,** no.64, Aug. 1992, pp.14-21.

4.14 Cebeci, 0.Z. and Saatci, A.M. Domestic sewage as mixing water in concrete, **ACI Materials Journal,** vol.86, no.5, Sept-Oct. 1989, pp.503-506.

Chapter 5

Chemical admixtures

Andries Marais

5.1 Introduction

Chemical admixtures are materials other than hydraulic cement, water, aggregates and supplementary cementitious materials which can be added to concrete, grout, or mortar during mixing to modify one or more of its properties. These changes to the properties can be in either the fresh or the hardened states.

The properties of fresh concrete can be modified to:

- Increase slump, without increasing water content (plasticising)
- Reduce water content, without changing slump (water reducing)
- Adjust setting time
- Reduce segregation
- Improve pumpability
- Reduce bleeding

The properties of hardened concrete can be modified to:

- Accelerate or retard the rate of strength development at early ages
- Increase strength, without increasing cement content
- Improve potential durability and reduce permeability

These lists are not exhaustive but summarise the effects that can be achieved with the use of admixtures.

Admixtures can be a single chemical or a blend of several chemicals able to produce a wide range of properties. Chemical admixtures are usually supplied as aqueous solutions but powdered admixtures are also available. The active chemical content of the aqueous solutions is typically 20 to 40%.

Powder admixtures can contain up to 100% active ingredients, but fillers are often added to the active ingredients to reduce the sensitivity, and allow for better distribution of the admixture. Under normal circumstances and at standard dosage rates, the added water from the admixtures is not enough to significantly change the water:cement ratio and adjustments are not required.

Admixtures should not be confused with additives or other additions that are inter-ground or pre-blended with the cement or dry cementitious mixes. Fibres, pigments, limestone powders, pozzolanic or supplementary cementitious materials such as FA, GGBS or CSF are not admixtures.

There is currently no South African specification covering the quality and performance of admixtures. South African manufacturers and specifiers therefore generally rely on the European EN 934[5.1] and American ASTM C494[5.2] specifications.

Much of the information in this chapter has been drawn from the very authoritative publications listed under "Further reading" at the end of this chapter. They are invaluable sources for those wishing to research the subject in more detail.

5.1.1 Types

Admixtures are normally categorised according to their effect, and the following categories are covered by the EN 934 specification.[5.1]

- Water-reducing or plasticising admixtures
- High-range water-reducing or superplasticising admixtures
- Retarding and retarding plasticising admixtures
- Accelerating (setting and/or hardening) admixtures
- Air-entraining admixtures
- Water-resisting admixtures
- Mortar admixtures
- Sprayed concrete admixtures
- Grout admixtures

Admixtures available on the market and not covered by EN 934 include:

- Admixtures for foamed concrete and low-density fill material
- Admixtures for precast, semi-dry concretes
- Corrosion-inhibiting admixtures
- Pumping aids
- Anti-washout admixtures
- Polymer dispersion admixtures
- Shrinkage-reducing admixtures

5.1.2 Mechanism of action

An admixture's mechanism of action depends on the type of admixture, but generally involves one or more of the following:

- Chemical interaction with the different cement phases to modify the rate of hydration, typically causing the retardation or acceleration of the concrete's setting time.
- Adsorption onto the surface of the cement particles, generally by steric or electrostatic interaction, causing particle dispersion within the concrete.
- The use of surfactants that affect the surface tension of the water. This will generally result in the entrainment of air.

All the actions mentioned will modify the properties of the fresh concrete, with the effects from using the admixture apparent in the properties of the hardened concrete.

Most admixtures will show primary effects (the property for which the admixture is used) as well as secondary effects, which are additional effects caused by the use of the admixture. These secondary effects can cause an admixture to work well using one group of materials, yet be unsuitable when using a different cement or aggregate.

The admixture must be optimised with the aggregate, cement and the mix design that will be used. The use of different admixtures in the same mix design will result in different properties in the fresh concrete. This is the result of the admixtures reacting differently on the plastic viscosity of the concrete. For example, a dispersing admixture will reduce the yield stress (slump) but may increase or decrease the plastic viscosity (consistency of the mix), depending on the secondary properties of the chosen admixture (see Figure 5.1).[5.3]

It is fair to say that admixtures have come of age nationally and internationally, and that without them it would be difficult to supply the concretes and mortars now demanded by fast, modern, construction processes. New technologies such as high-strength, high-performance, ultra-high-strength and self-compacting concretes and self-levelling screeds, would not be possible.

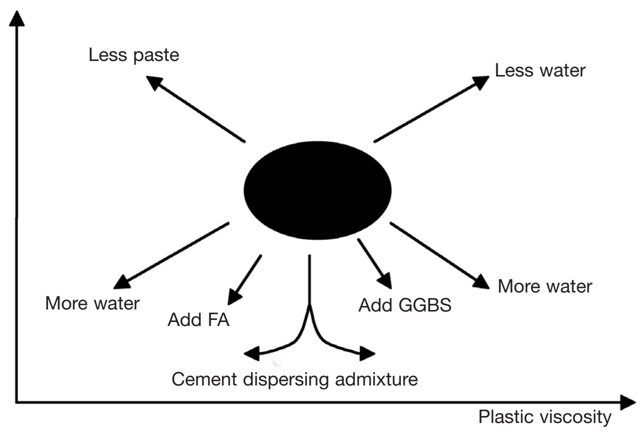

Figure 5.1: Concrete rheology, effect of mix materials [5.3]

5.2 Admixture categories and guidelines for use

Several products are available within each of the following categories, but only generic information on each category is provided in this section.

It is always good concrete practice to confirm, with the supplier, the primary and secondary properties of the admixtures to be used and to follow the instructions and technical information provided by the supplier.

5.2.1 Dispersing admixtures

Dispersing admixtures fall into two main groups, which are divided by their water-reducing abilities. This division, as recognised by EN 934[5.1] and ASTM C494,[5.2] is into normal plasticising admixtures and water-reducers, and superplasticising admixtures and high-range water-reducers. Terms such as multi-dosage, mid-range and first-, second- and third-generation plasticising admixtures are used by admixture companies.

Dispersing admixtures are the most widely used admixtures in South Africa.

This group of admixtures is adsorbed onto the surface of the cement. Once adsorbed, the admixture interacts with the cement particles causing a uniform distribution of the particles in the aqueous phase and increasing the fluidity of the mix. This interaction can be as a result of electrostatic repulsion or steric stabilisation.

Dispersing admixtures are used: [5.4]

- To increase slump at a given water content without affecting the water:cement ratio.
- To reduce the water content at a given slump to improve the strength and durability of the concrete.
- To reduce water and cement contents, thereby maintaining the water:cement ratio, to give equivalent strength more economically.

Water-reducing or normal plasticising admixtures

Normal plasticising admixtures are water-soluble organic materials usually based on salts of lignosulphonate. These materials reduce the amount of water needed to achieve a given workability in the plastic concrete, without significantly affecting the air content or setting times. Admixture manufacturers will use different quantities of the lignosulphonate salts and other chemicals to control the possible secondary effects of the admixture.

Characteristics

- Achieves a water reduction of between 5 and 12% compared with a control mix.
- Low dosage: 0,2 ℓ to 0,6 ℓ per 100 kg of cementitious content.
- Active ingredient content of the liquid admixture is normally between 30 to 40%.
- High dosages will not significantly increase the dispersion effect but will increase the secondary effects.
- The secondary effects are retardation and/or air entrainment.
- Can be used with all cements complying with SANS 50197-1.[5.5]
- Air content, mix cohesion and stiffening time may be slightly affected when blended cements are used.

Overdosing

Workability will increase until saturation point is reached, after which the fluidity will not significantly increase. Overdosing may result in retardation, settlement, bleeding and excess air entrainment. The retardation is unlikely to last for more than 24 to 36 hours and, without air entrainment, the gain in strength and other properties will develop normally after the initial retardation. Retardation beyond 36 hours can adversely affect the final properties of the concrete.

Overdosing can also result in concrete that is very fluid if the operator has not compensated by reducing the mix water. This very fluid concrete will segregate and produce a weak, honeycombed concrete caused by the fines leaking from the formwork.

Recently normal plasticising admixtures based on polycarboxylate ether were introduced into the market. These manufactured molecules were originally developed as superplasticising admixtures and are diluted to reduce sensitivity. Thus the molecules are also effective as normal or multi-dosage plasticising admixtures with minimal secondary effects.

Multi-dosage plasticising admixtures can be used over a wider range than normal plasticising admixtures. The dosage can be up to 1,2 ℓ per 100 kg of cementitious content without secondary effects.

Effects on concrete properties

Admixture performance is dependent on concrete materials, mix design, and admixture dosage.

Typically, normal plasticising admixtures will give 5 to 12% water reduction. EN 934-2 requires a strength improvement of at least 10% after seven and 28 days[5.6] when tested under laboratory conditions.

Normal plasticising admixtures will increase the air content of the concrete by 1 to 2% when measured against similar concrete without admixture. The additional air will improve the cohesion of the mix and can reduce the tendency to bleed.

Most normal plasticising admixtures retard the setting time of the concrete by 30 to 90 minutes over the measured setting times of similar concretes without admixtures.

The principal application of normal plasticising admixtures is in the readymix industry where they are used to optimise cement contents. Water reduction at constant water:cement ratio will reduce costs.

High-range water-reducing or superplasticising admixtures

Superplasticising admixtures are synthetic, water-soluble organic materials, with very high water-reducing properties at various dosages. Superplasticising admixtures function similarly to normal plasticising admixtures, but they are more powerful dispersers that can be used at higher dosages than normal plasticisers, without the unwanted secondary effects such as air entrainment or set retardation associated with normal plasticising admixtures.

Superplasticising admixtures consist of manufactured molecules based on one of the following:

- Salts of sulphonated naphthalene formaldehyde
- Salts of sulphonated melamine formaldehyde
- Polycarboxylate ether derivatives

Characteristics

- Water reduction of over 12% compared with a control mix.
- Dosage: 0,6 ℓ to 2 ℓ per 100 kg of cementitious content for naphthalene- and melamine- based admixtures.
- 0,4 ℓ to 1,0 ℓ per 100 kg of cementitious content for polycarboxylate ether-based admixtures.
- Active ingredient content of the admixture is normally between 20 and 45%.
- The secondary effects of superplasticising admixtures are negligible.
- Superplasticising admixtures can be supplied as pure products, eg sodium naphthalene formaldehyde, sodium melamine formaldehyde or polycarboxylate ethers. The chemicals can be blended with one another, and with lignosulphonate, to produce admixtures which can produce concrete with a wide range of rheological properties.
- Dual-function products that are designed to give two primary functions, eg water reduction and retardation for use in hot weather, are common.

Overdosing

Water reduction and fluidity of the concrete will increase and cohesiveness will be reduced to such an extent that the concrete will show segregation. The concrete may be retarded and air entrainment can occur at extreme overdosing. If the segregation was not serious and the concrete was placed efficiently and correctly, and retardation is observed, it is unlikely to continue for longer than 24 hours. Extra curing and protection of the concrete will be required.

Admixture selection

The molecules used in superplasticising admixtures will give good performance in most applications. Various types of molecules have specific characteristics that will make them more suitable for particular applications.

- Naphthalene salts achieve good water reduction and strength development with reasonable workability retention. Typically, 16 to 25% water reduction can be expected, but there may be problems with air entrainment and poor surface finish.
- Melamine salts give good early strength development at low temperatures with good surface finish. Typically, 16 to 25% water reduction can be expected. The concrete may lose workability quickly at higher temperatures and tends to bleed and segregate more in mixes with low cohesion.
- Polycarboxylate ethers produce maximum water reduction and early strength development. Typically, 20 to 35% water reduction can be expected. They are perfectly suited to concretes with high fluidity and flow retention. The water reduction is good at low dosages. These products are, however, sensitive to cement chemistry and performance will differ with different cements.

A wide range of superplasticising admixtures with different properties and functions is available. The supplier's advice should be sought and trial mixes should always be made in conjunction with the technical support of the supplier to ensure optimised usage of the products.

Superplasticising admixtures are an essential ingredient of high-performance concretes. They are not as cost effective as cement reducers because their own cost is substantial, and the savings effected by reducing the cement content are not usually sufficient to cover the admixture cost. The benefits of using superplasticising admixtures lie in the improvement of concrete properties to suit specific applications, eg:

- Increased fluidity enabling the concrete to penetrate around dense reinforcement
- Self-compacting or self-levelling concretes
- The reduction of the w:c ratio to give high strength (early and late)
- Improved durability
- Reduced shrinkage

Effects on concrete properties

The slump of the concrete with superplasticiser at equal water content will typically be 12% higher than the control mix without any admixture.[5.6]

EN 934-2 requires superplasticising admixtures to give strength of the test mix ≥ 140% of the control mix after one day and the test mix must be ≥ 115% of the control mix after 28 days[5.7] when tested under laboratory conditions.

The removal of the water reduces the concrete's permeability, resulting in reduced penetration by aggressive molecules thereby producing concrete of greater durability.

5.2.2 Retarding and retarding plasticising admixtures

Retarding admixtures are water-soluble chemicals that cause a decrease in the rate of hydration of hydraulic cement. They extend setting time by between 90 and 360 minutes and have little or no effect on the water demand of the concrete.

Retarding plasticising admixtures have the added benefit of changing the fluidity of the concrete while delaying the setting time. They are generally used to achieve extra workability retention – especially when cold joints are not permitted, such as in mass concrete pours and in ready-mixed concrete for delayed placement because of long distances to remote sites.

The chemicals generally used include polysaccharides and phosphates.

Characteristics

- Dosage: 0,2 ℓ to 0,6 ℓ per 100 kg of cementitious content, if supplied and used as a retarding admixture only.
- Retarding plasticising admixtures are generally dosed similarly to plasticising admixtures.
- Temperature has a significant effect on the performance of retarding admixtures. Trial temperatures must be taken into account, as even a 5°C change will have a significant effect on setting time results.
- The use of FA and GGBS will also increase retardation and this should be taken into account, especially if the temperature is likely to be low.

Overdosing

Any overdosing will increase the concrete's setting time. Where overdosing is not too serious, the concrete should recover within two to three days, provided that it is protected and cured properly. Generally, if the concrete has not recovered within five days, it is likely that the cement in the concrete may never hydrate fully, significantly increasing the possibility that the concrete will not gain full strength.

Retarding admixtures are very sensitive and great care must be taken with their dosage and general use.

Effects on concrete properties

The total volume of bleed water arising from the concrete is related to the setting time and retarded concrete can be expected to bleed more. Retarded concrete will also be more prone to settlement and plastic cracking than normal concrete. Retarding admixtures do not reduce the concrete's heat of hydration but will prolong the time taken to reach peak temperature. This delay can allow some of the heat to dissipate so that the peak temperature reached may be slightly lower in smaller sections.

Since it is plastic for longer, retarded concrete is more susceptible to environmental changes. Curing and protection of retarded concrete should be a priority and more attention to detail will be required than for normal concrete.

The demand for exposed aggregate surface finishes has led to the development of specialised surface retarders. These retarders are applied on the surface of the concrete to delay the hydration of the cement on the surface. During cleaning of the concrete surface with high-pressure water jets, the mortar between the aggregates is removed and the aggregates are exposed. These products are very specialised because of the different products combined to achieve the depths of exposure required.

To obtain good results the following points are very important:

- The concrete should be homogeneous.
- The concrete should only be vibrated for a very short period to minimise the mortar layer on the surface.
- Temperature conditions will affect the performance of the admixture.
- High cement contents and rapid-hardening cements should be avoided.
- Inert fillers such as limestone will affect the performance of the surface retarder.
- Microfibres can be used to increase the cohesiveness of the stone-rich mixes required for this application.

Following the manufacturer's instructions on the use of these products is critical to the success of the project.

Two types of surface retarders are available: positive and negative surface retarders.

Positive surface retarders are applied on the concrete surface after placement of the concrete. The surfaces are then washed with high-pressure water jets after the reaction time has passed. Typical applications would be roads and pavement areas.

Negative surface retarders are applied to the mould or formwork surface. After demoulding, the concrete surface that was exposed to the surface retarder is washed with high-pressure water jets. Typical applications would be precast elements for buildings.

The application and the stone size used affect the choice of surface retarder.

The preparation of a test section is important to ensure that the required effect and depth of exposure is achieved with the chosen surface retarder and exposure method.

5.2.3 Accelerating admixtures

Two groups of accelerating admixtures are available, based on their application and their performance characteristics:

1. Set accelerating admixtures that are used to reduce the time for a concrete mix to change from a plastic state to the hardened state. There are two groups of set accelerating admixtures in general use. The first is for sprayed concrete applications, where set acceleration is very fast, normally less than 10 minutes (see section 5.2.7); the second group will, in accordance with EN 934, reduce the setting by at least 30 minutes at 20°C and at 5°C: test mix $\leq$ 60% of the control mix.[5.8]
2. Hardening accelerating admixtures that will, under normal curing conditions, increase the 24-hour strength of the concrete by at least 20% at 20°C and at least 30% at 48 hours.[5.8]

Calcium chloride is the most effective accelerator and can be used in both setting and hardening applications.

However, the introduction of chloride ions is not permitted in steel-reinforced concrete and only chloride-free accelerators should be used in reinforced and prestressed concrete or concrete containing steel fittings. Salts of nitrate, nitrite and thiocyanate are most often used in chloride-free accelerating admixtures.

Characteristics

- Dosage is typically at 0,5 ℓ to 2,0 ℓ per 100 kg of cementitious content.
- Accelerating admixtures are most effective at low temperatures where early strength is required.
- Hardening accelerating admixtures are appropriate for strength gain up to 24 hours at low temperatures and 12 hours at higher temperatures. Beyond these times, high-range water-reducing admixtures would be more cost effective.

Overdosing

Overdosing will cause rapid stiffening rates and considerable heat evolution. This will cause problems with placement, compaction, and thermal cracking.

Effects on concrete properties

Accelerating admixtures will significantly increase early strength of concrete (30% over a control mix without accelerator at low temperatures). The slump loss will be quicker and concrete will not bleed under normal circumstances. The heat of hydration rate will increase and peak temperature will be reached more quickly than normal concrete without accelerators. The total heat of hydration generated is however unaffected.

5.2.4 Air-entraining admixtures

Air-entraining admixtures are surface-active chemicals that allow small stable air bubbles to form in the concrete mix. The air bubbles are distributed evenly throughout the paste and are smaller than 1 mm in diameter, with the highest portion below 0,3 mm. Air entrainers affect concrete in both the fresh and hardened states.

This air is quite distinct from entrapped air resulting from incomplete compaction during placing, which is seen as large voids within and "blow-holes" on the surface of the hardened concrete. Entrained air, unlike entrapped air that can and should be removed with proper compaction, will not be removed with compaction but will be reduced in quantity.

Air-entraining admixtures are used to produce a number of effects in the concrete mix. Entrained air in the concrete will:

- Improve cohesion and reduce bleeding.
- Improve compaction of low-workability concrete.
- Provide stability to extruded concrete.
- Give improved handling properties, stability and cohesion to bedding mortar.
- Improve the freeze/thaw resistance of hardened concrete (not a serious problem in South Africa).

Air entrainers are mainly synthetic surfactants or blends of surfactants used to produce stability and improved void characteristics.

Air-entraining admixtures should not be pre-mixed or come into contact with other admixtures before dosing into the concrete.

The volume of entrained air for a given dosage is influenced by the following factors:

- Some types of sand, eg crushed dolerite, reduce air volume.
- An increase in sand content of 5% will lead to an increase in air content of 1 to 1,5%.
- An increase in the fineness of the sand will lead to an increase in the air content.
- Increased mixing time reduces air content.
- Forced-action mixers entrain larger volumes of air.
- Increasing temperature tends to reduce air volume.
- GGBS and FA tend to reduce air entrainment.
- The time and order in which admixtures are added to the mix will influence the total air content.

Characteristics

- Dosage: 0,2ℓ to 0,4ℓ per 100 kg of cementitious content.
- The active ingredients are very powerful and only 1 to 5% solids content is used in the admixtures.
- Obtaining and maintaining the correct volume of air over a series of mixes can be difficult. Control over the mix process and maintenance of constant material quality are essential.
- It is very difficult to entrain air in low-slump mixes.
- Strength is reduced by the entrained air at a rate of 5 to 6% per 1% air entrained, except in lean concrete where strengths may be increased. Air entrainment should, therefore, be limited to the minimum amount required when used in normal concrete applications.

Overdosing

This will result in more air being entrained and consequently in a greater loss of strength. The air content of plastic concrete can be measured using an air-meter, and in hardened concrete by means of microscopy.

Effect on concrete properties

Air-entraining admixtures will reduce the density of the concrete. The cohesiveness of the concrete will increase, and the bleeding and segregation will also be improved. Air-entraining admixtures are successfully used in South Africa in extruded and semi-dry concrete mixes, to aid compaction and hold the shape without further slumping and deformation. In areas where freeze/thaw and the resulting scaling of the concrete are major problems, air entrainment is used to increase the resistance of the concrete.

5.2.5 Water-resisting admixtures (waterproofing)

Water-resisting admixtures are defined by EN 934 as "materials that reduce the capillary absorption of water into the hardened concrete."[5.9]

The primary function is to reduce the passage of water through the concrete and this is done by one or more of the following:

- Reducing the size, number, and continuity of the pore structure.
- Blocking the pore structure by adding fine un-reactive or reactive fillers or using insoluble polymers such as emulsified bitumen.
- Lining the capillaries with a hydrophobic material to prevent the water from being drawn in by capillary suction.

Pore blockers are most effective when used in combination with high-range water-reducing admixtures. They can also reduce the amount of efflorescence on the concrete.

Water-resisting admixtures will not reduce water penetration through cracks or through poorly compacted concrete.

Hydrophobic products are generally effective against:
- Rain
- Surface water
- Low pressure heads in structures
- Water ingress in tidal and splash zones
- Build-up of absorbed chloride at the surface of the concrete

Characteristics
- Dosage: Hydrophobic admixtures at 1 to 2% by weight of cement, and pore blockers at 5 to 10% by weight of cement or more often as a dosage of total concrete volume.[5.10]
- Combinations of the above functions are available as one- or two-part systems depending on the requirements of the project.

Overdosing
It is usual for these admixtures to cause loss of both workability and strength, but specific effects would depend on the type of admixture used. It is recommended that the supplier be contacted for details.

Effects on concrete
These admixtures are used mainly as durability enhancing materials. They reduce the ingress of water and air into the hardened concrete thereby producing concrete with reduced permeability.

5.2.6 Mortar admixtures

These admixtures are combinations of plasticisers, air entrainers and retarders. They are formulated to produce workability that lasts, typically, for 36 hours. Once the mortar is placed between the absorbent bricks, setting is accelerated, the mortar stiffens, and strength development starts. These admixtures are normally added to mixes under controlled conditions at readymix plants and this process, together with the mix design of the mortars, should always be done in collaboration with the admixture supplier. The manufacturer's instructions must always be followed to ensure the successful use of the ready-mixed mortar.

Mortar plasticising admixtures are used to compensate for poor workability, which results from the use of single-sized sands. They are formulated to balance workability and strength requirements of a mortar and to inhibit bleeding. The type and dosage of admixture will be affected by:
- Grading of the sand, particularly the minus-75-μm fraction
- Organic impurities in the sand
- Specific surface of the cement
- Lime in the mortar

5.2.7 Sprayed concrete admixtures

Wet-process sprayed concrete is pumped to the point of application and then pneumatically projected into place at high velocity. Admixtures are used in the fresh concrete to give stability and hydration control before the spray process starts. The admixtures used are normally superplasticisers and retarders.

The process requires the concrete to set as soon as it reaches the substrate. The accelerators are fast acting and are added at the nozzle of the spray equipment.

The two accelerating admixture groups used are:
- Alkaline, which gives the quickest set, often less than 1 minute, but tends to give very low late strengths.
- Alkali-free, with which setting can take up to 10 minutes, but which does not affect the strength development at 28 days.

Because alkaline products are highly corrosive, they present a safety hazard to the applicators. Consequently, alkali-free accelerators are most commonly used. The added advantage is that 28-day strengths are unaffected.

Sprayed concrete is often supplied to the client as a complete system including: concrete, admixture, machine, and application personnel. As the success of sprayed concrete is affected by cement chemistry and admixture use, this application requires both technical knowledge and practical experience. It is thus best left to specialists in the field.

5.2.8 Grout admixtures

Grout admixtures are normally gas-expanding systems that release a gas while the grout is still fluid. This bulks the grout and fills the voids in the grouting duct. Anti-washout admixtures can be used in vertical ducts in conjunction with the grout admixtures to reduce possible bleeding.

Because these are admixtures used for very specialised applications, it is recommended that only specialist contractors be employed to ensure the success of the project.

5.3 Admixtures not covered by EN 934 or ASTM C494

All the admixtures previously discussed are covered by EN 934 and ASTM C494. The following admixtures are not covered by known standards, but they can be important for certain specialised applications.

5.3.1 Admixtures for foamed concrete and low-density fill materials

The admixtures in this section are used in low-density materials for various applications.

Foamed concrete

Admixtures for foamed concrete are surfactants that are diluted with water and then pumped through a foam generator to produce stable foam. This foam is mixed with a cementitious mortar to produce foamed concrete.

The dosage of these admixtures depends on the expansion of the foam and the density required. The lower the density the higher the dosage. As a guide for use, a final density of 1 100 kg/m^3 will require a dosage of 0,5 to 0,8ℓ/m^3 of foamed concrete.

Low-density fill materials

These materials are sometimes called CLSM or controlled low-strength materials. They have good flow properties and are used for trench filling and other low-strength void filling applications.

Admixtures for low-density fill materials are surfactants that are added to sand-rich, low-cement-content concrete to increase the air content by up to 25%.

The admixture dosage for low-density fill material varies between manufacturers. The concrete design is critical and higher than normal sand fractions are necessary. Generally, densities below 1 700 kg/m^3 are difficult to achieve. Varying the sand:cement ratios will give some control over the density.

Guidance from the supplier followed by trials with the materials to be used is necessary to establish the correct dosage, mix and application conditions.

Two types of admixtures are available: synthetic surfactants or blends of synthetic surfactants and protein-based admixtures.
Overdosing will have very little effect on foam generation and the density of the low-density fill material.

5.3.2 Admixtures for precast semi-dry concrete

These admixtures are used in the manufacture of dry, or semi-dry vibrated and pressed concrete products, eg masonry units, paving bricks and flagstones. They are used to give early strength, to assist in compaction, disperse the cement and colouring agents, and to minimise lime staining or efflorescence. The admixture used can be one of the conventional admixtures described in this chapter or a specially formulated multifunctional admixture.

Discussion with the admixture supplier regarding specific properties required is necessary to ensure that the correct admixture and/or combination of conventional admixtures is selected.

5.3.3 Corrosion inhibitors

Corrosion inhibitors are used to increase the passivation of reinforcing steel after the concrete has hardened. Corrosion inhibitors can be added to the fresh concrete mix or applied to the hardened concrete.

Three generic types are available:

- Calcium nitrite with residual amounts of calcium nitrate
- Amino alcohols
- Amino alcohols blended with inorganic inhibitors

Calcium nitrite also has accelerating properties and is used in chloride-free accelerators.

Overdosing will accelerate the concrete.

Admixture dosage is dependent on the client's expected serviceability requirements. Other factors that affect the durability of the concrete include:

- Cement type
- Water:cement ratio
- Concrete cover to steel
- Levels of exposure

All three types of inhibiting admixtures have been used successfully for many years. The addition of the corrosion-inhibiting admixtures will not affect the long-term durability of the concrete, but can offer extended life to the overall structure.

5.3.4 Pumping aids

Pumping aids are used to reduce pump pressures by acting as a lubricant. The chemicals used are long-chain polymers or surfactants similar to air entrainers. Modern pumps are more sophisticated and are able to cope with most concrete mixes without the use of such admixtures.

The use of superplasticising admixtures and improved concrete mix designs have largely limited the use of pumping aids.

5.3.5 Anti-washout admixtures

Anti-washout admixtures are used in concrete that is placed under flowing or moving water to decrease the washing out of cement and fine aggregates. The use of an anti-washout admixture will decrease the risk of failure when concrete is placed deep under water, or in inter-tidal and splash zones.

The admixtures are organic polymer thickening agents normally used in conjunction with superplasticisers to produce self-levelling flowing concrete for easy placement. They are powder based and are difficult to dose. Overdosing will make the concrete very cohesive, and may increase setting time. The placement of the concrete, when it is very cohesive, will be difficult. Technical advice from the supplier is essential.

5.3.6 Polymer-dispersing admixtures

Polymer-dispersing admixtures contain generic polymers that will form a continuous film in the concrete to:

- Reduce permeability
- Give a reliable bond to the existing concrete as well as to the steel
- Increase ductility and toughness

Typical dosage is 10 to 30 ℓ per 100 kg of cementitious material and the application is restricted, for economic reasons, to screeds, mortars, plasters and grouts.

5.3.7 Shrinkage-reducing admixtures

These admixtures are mainly used in floor areas where, for technical reasons, normal joints are undesirable and in strengthening or repairing old concrete where shrinkage could cause failure

Shrinkage-reducing admixtures are 100% active liquids that are water-soluble and can significantly reduce the shrinkage, both early and long term, of hardened concrete.

The admixtures are mainly glycol ether derivatives with a specific gravity of less than 1 and a distinct odour. Overdosing will extend the setting time and can lower compressive strength.

5.4 Using admixtures: practical considerations

5.4.1 Design of mixes

Literature from admixture suppliers gives general information on the use, characteristics, precautions and effect of specific admixtures.

A laboratory test programme should be undertaken to optimise dosages with the cementitious materials and aggregates to be used on site. This programme should be based on good concrete mix design methods and correct laboratory practice. It is also essential to verify laboratory mixes in site trials.

The control mix should be prepared first, to prevent trace contamination with the admixture under test. Equipment such as measuring cylinders, trowels, floats, etc, should also be cleaned regularly to prevent contamination as some admixtures have a marked effect even at trace levels.

As an admixture may have side effects that differ from its primary effects, mixes should always be tested to ensure that the concrete behaves the way that is intended.

Changes in materials or ambient conditions may also alter the effectiveness of admixtures; trials are essential if such changes are envisaged.

If more than one effect is desired, then more than one admixture may be required. When two or more admixtures are used, particularly if from different suppliers, compatibility needs to be tested using materials from the site. This is particularly important if cement extenders or extended cements are being used.

5.4.2 Quality assurance

Sometimes it is necessary to determine whether an admixture is similar to the ones tested previously or to determine if successive lots are constant. The following tests can be used:

- Specific gravity
- pH
- Viscosity
- Solids content
- Reflective index
- Infrared spectrophotometer measurements

Admixture suppliers can advise on the product specifications and must provide compliance certificates for each batch delivered. Personnel should become familiar with the appearance and odours of the products, as this knowledge can prevent errors.

End users of admixtures are advised to use suppliers that implement quality assurance management systems complying with ISO 9001 [5.11] and use only laboratories that are accredited to ISO 17025.[5.12]

5.4.3 Storage and handling

Most admixtures are biodegradable active chemicals that are available as aqueous solutions or in powder form.

Liquid admixtures are normally available in bulk, or in 1 000- 200- and 25-ℓ containers. Powdered admixtures are normally packed in 10- to 25-kg bags.

Labels: Admixtures are potent modifiers of concrete properties, and it is important to prevent errors. It is essential to ensure that, on receiving the admixtures, all containers are properly marked and are stored so that the labels are clearly visible.

Admixtures should be protected from extreme temperature conditions. As most have freezing points of about 0°C, when an admixture freezes, the chemicals crystallise out of solution and may not readily go back into solution, even on reheating. At high temperatures, the water in the admixture can evaporate leaving behind the active ingredients. This will change the formulation and concrete properties will be affected. The manufacturer's literature gives specific details on temperature tolerances for each product.

Admixtures generally contain preservatives to reduce bio-degradation and extend shelf life. Despite this, the activity and effectiveness of an admixture will gradually decrease with time. They should therefore be used on a first-in-first-out basis.

Powdered admixtures must be stored in similar conditions to cementitious materials. Most powdered admixtures are wettable and will therefore have some hygroscopic properties. Keep powders dry and, if packed in bags, protect the bags from damage and spillage.

Admixture containers must be left closed to reduce the risk of contamination, which can affect the performance of the admixture or the dosing equipment.

Certain admixtures have a high pH and are therefore corrosive/harmful, eg air entrainers, waterproofers and shotcrete accelerators. These products can only be stored in certain types of containers. Always check with suppliers on the type of container to be used if there is a need to repack from original containers.

All admixtures are supplied with a shelf life indicated on the packaging or containers or, if delivered in bulk, on the delivery documentation. Do not use admixtures when the shelf life has expired. Contact the admixture supplier to determine the process to follow when expired admixtures are discovered.

Disposal of old unwanted admixtures must always be done through the supplier or through approved contractors.

5.4.4 Dosage and dispensing

The dosage range of most admixtures is only 0,2 to 2,0 litres per 100 kg of cementitious materials and correct dosage is crucial for satisfactory mix performance. The proper use of well-maintained and calibrated dispensing equipment is essential. Small changes in actual dosing can have a significant effect on concrete quality and properties. Admixture suppliers normally provide, install and service manual and automatic dosing equipment.

The main methods of batching admixtures are:

- Positive displacement volumetric batching, using a sight glass into which a measurable volume of the admixture can be pumped
- Positive displacement flow meters similar to water meters
- Timer control systems
- Mass batching

All dosing systems have a successful history but each has its own strengths and weaknesses. The admixture supplier can advise on the most appropriate system to be used. Integration into normal batching procedures is important for quality control purposes.

Where more that one admixture is dosed through one pump or system, an automatic water-flushing system must be installed to clean the system and add the entire measured product into the mix.

Admixtures should be added to the concrete during mixing with the last portion of the mix water at the end of the batching process, in order to ensure even dispersion of the admixture throughout the concrete. The admixture should not be added directly to the dry cement or aggregates. Powdered admixtures should be batched by mass.

The admixture must be mixed with the water and then added to the dry ingredients to ensure proper dispersion of the admixture in the concrete mix. In some specialised applications where pre-batched materials are bagged for use at a later stage, the powdered admixture must be batched with dry aggregates and not with cement.

For the majority of admixtures, it is not necessary to change the concrete mixing procedure. Any special requirements will be given in the admixture manufacturer's literature.

Multiple admixtures are often used in the production of concrete. Compatibility of the different admixtures needs to be checked and the sequence of dosage must be determined to ensure that there is no undesirable interaction.

Re-dosing on certain superplasticising admixtures can be done on site to extend the workability of the concrete. Any additional dosing must be done using calibrated, metered pumps, properly supervised, and to an agreed plan.

5.4.5 Safety

Most admixtures are not hazardous to health, although certain admixtures are classified as irritants and pollutants and some may be flammable. The following safety procedures apply to all admixtures.

- All admixture installations must be clearly identified and the tanks and pipes must be clearly marked with their contents.
- Avoid eye, mouth and skin contact.
- Should skin contact occur, wash off immediately with plenty of fresh water and soap.
- Contaminated clothing should be removed and washed.
- Seek medical advice in cases of serious eye contact, ingestion or the excessive inhalation of fumes or powders.
- Admixture spillage will cause floor areas to become slippery and unsafe. Spillage should be immediately hosed down with water to prevent accidents.
- As a precaution, treat all admixtures as dangerous in fire situations.
- Always keep a copy of the material safety data sheet at hand for reference in case of an emergency.
- Major spillages can have an environmental impact and need to be contained and properly cleaned by experts.

Further reading

The following documents were consulted in the preparation of this chapter and are regarded as invaluable resources should additional information be required.

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Chapter 6
Properties of fresh concrete

John Kellerman and Steve Crosswell

6.1 Introduction

Concrete is in the fresh state for only a few hours, compared with a service life that may be several decades. But the properties of fresh concrete are important because they influence the handling of the concrete, the degree to which it can be compacted and the uniformity of distribution of constituents within the concrete. Handling of concrete has practical implications for construction, while compaction and uniformity affect the properties of hardened concrete.

This chapter discusses workability and the factors affecting it, consistence and its measurement, cohesiveness, bleeding, plastic cracking, slump loss, density and air content.

6.2 Workability

A review of literature relating to workability shows the difficulty in defining this property. It is accepted, however, that the behaviour of the fresh concrete must relate to the way the concrete is handled on site. After thorough mixing the concrete is transported by one or more of a variety of methods; when placed, it must be able to flow into corners and around congestion such as reinforcement or box-outs; it must be capable of being fully compacted using the available equipment, and it must provide an adequate finish without honeycombing, excessive blow holes, sand streaking, etc. Where necessary it must respond adequately to a finishing process such as power trowelling. Workability requirements are also specific to the application: concrete that would be suitable for placing in a large foundation would probably be unsuitable for placing in a thin member; concrete that is considered workable when compacted using internal vibration would be unworkable if hand compaction were used.

Full compaction and minimum segregation and bleeding of the fresh concrete are essential to achieve the required strength and durability characteristics of the hardened concrete. The workability must therefore be such to allow full compaction to be achieved with the available equipment and without segregation occurring.

Perhaps an adequate definition of workability is that given by ACI Committee 116R00:[6.1] *"that property of freshly mixed concrete or mortar which determines the ease with which it can be mixed, placed, consolidated and finished to a homogeneous condition."*

Workability is a composite property and is not measured directly by any of the methods described in section 6.3 although an assessment of workability can be made by an experienced technologist. In describing workability, the two most commonly used qualitative terms are:

- *Consistence,* which describes mobility or the ease of flow and is related to the wetness or dryness of the mix. Generally, wet concrete is more workable than dry concrete, but concretes of the same consistence may vary in workability.
- *Cohesiveness,* which describes the tendency to resist segregation and bleeding.

Other terms such as compactability, finishability and pumpability are also used to describe a particular property of the fresh concrete.

6.2.1 Factors affecting workability

Workability is affected by a number of factors other than the most obvious one of water content. These factors include proportioning of ingredients, aggregate and cement types and characteristics, presence of admixtures, time elapsed since mixing, and temperature.

The effects of these factors are not necessarily independent of each other. For example, the effect of time depends on the mix characteristics, and the effectiveness of an admixture is often influenced by the type of cement.

The effects of proportioning, aggregate characteristics, cement type and admixtures are discussed in this section; the effects of time and temperature are discussed in section 6.7.

Mix proportions

The proportions in which the constituents of a mix are combined significantly affect workability.

Water content

In an average mix using 19-mm crushed stone, a total water content of about 210 ℓ/m³ is required to give a slump of 75 mm. The effective or free water content (total water less water absorbed by the aggregates), which in this case would be about 190 ℓ/m³, is needed to make the concrete workable enough for handling, placing and compacting.

In a well-proportioned mix, an increase in water content will make the concrete more mobile or flowable.

With concretes of low total water content there is a general tendency for segregation to occur. This tendency can be reduced by increasing the water content, but further water increases (leading to high slumps) may also cause segregation as well as excessive bleeding.

Paste content

For a concrete mix to be plastic, it is necessary to have more cement-water paste in the mix than the amount required to fill the voids between aggregates in a packed state. Within the range of normal mix proportions, workability tends to improve with increasing paste content (and increasing paste:aggregate volume ratio).

Cement content

Concrete with a low cement content tends to be harsh and have poor finishability. On the other hand, very rich mixes tend to be sticky and rapidly lose workability.

Sand fraction

Where the sand fraction of total aggregate (or ratio of sand to stone) is too low, concrete tends to be harsh and unworkable. But over-sanded mixes tend to be sticky and may segregate internally if over-vibrated.

Aggregate characteristics

Aggregate characteristics that influence workability are shape, surface texture, average particle size, grading and fines content.

As the shape of a mass of concrete is changed, aggregate particles rotate and move relative to each other. The ease with which such movements can occur determines workability and depends on aggregate characteristics such as shape and surface texture. Rounded smooth particles can move more easily, and consequently impart greater workability, than rough angular particles.

In well-proportioned mixes, a smaller stone size tends to improve workability although water requirement for a given slump increases. The grading of coarse aggregate has little effect on workability, although graded materials tend to improve properties such as pumpability.

The fineness modulus of the sand (see section 3.2.3) has relatively little effect on workability. Of more importance is the grading. Graded sands improve workability whereas single-sized sands contribute to excessive bleeding, poor finishing and may increase the water demand. Coarse sands, lacking material passing the -300-μm sieve, increase bleeding and the likelihood of segregation, but excessive fines can create sticky mixes with high water requirement, especially in the case of rich mixes.

Cement type

Workability may be affected to a limited degree by differences in cement types eg, CEM l 42,5N and CEM l 42,5R/52,5N, and between the same type of cement from different sources.

The use of ground granulated blastfurnace slag (GGBS) may slightly improve workability; the use of fly ash (FA) usually improves workability; the use of condensed silica fume (CSF) in excess of 5% by mass of cement tends to reduce workability; the use of ground granulated Corex slag (GGCS) makes mixes more sticky but does not materially change the workability. This applies equally to site blends of cement and extender and to factory-blended cements containing extenders.

Admixtures

The use of plasticisers or superplasticisers (see section 5.2.1) leads to an increase in workability at a given water content and admixture dosage, but the degree of increase is affected by other constituents of the concrete, particularly the cement.

Air entrainment increases the volume of the paste. At a given water content, workability is improved and segregation and bleeding are generally reduced, especially in lean mixes or where coarse sands are used.

6.3 Consistence

Consistence is defined in section 6.2.

6.3.1 Measuring consistence

Sampling

Tests should not be carried out using “spot” or unrepresentative samples.

SANS 5861-2:2006[6.2] provides a method for sampling freshly mixed concrete, in which several increments equally spaced throughout the batch are to be taken. BS EN 12350-1:2000[6.3] allows for a modified sampling method to be adopted in the case of concrete delivered in a truck mixer. The slump tolerances for concrete sampled in this method are, however, greater than for concrete sampled in the standard manner.[6.4]

Once the increments have been obtained, it is necessary to thoroughly remix them on a suitable non-absorbent surface to produce a composite sample. The time between forming the composite sample and carrying out the test must be as short as possible. Where necessary the sample must be protected against drying out, etc.

Timing

Especially in the case of laboratory mixes made using air-dry aggregates, a suitable time should elapse between adding the mixing water and carrying out any test on the fresh concrete. This is to allow initial absorption of some mix water by the aggregates to take place.

If done too soon, different test results, eg higher slumps, are likely to be obtained.

Figure 6.1[6.5] shows the effect of workability with time, in particular the effect on the compaction factor within the first few minutes of adding water to a mix. BS 1881-125 [6.6] describes a method for mixing concrete in the laboratory in which the dry aggregate plus about half of the mixing water are mixed together and allowed to stand for five to 15 minutes before adding the other materials. A minimum mixing cycle of ten minutes is required for this method.

Methods of measuring consistence

In his book on workability, Tattersall[6.7] has incorporated a review by Weiring of consistence tests as carried out in various countries. The table shows almost complete acceptance of the slump test, with most authorities including a flow, Vebe and compaction test.

None of these tests is suitable for the whole range of consistence met in practice, thus the various tests have a limited application. Recommendations for the test method to be used are shown in Table 6.1 and are derived from SANS 5862:2006-1 to 4.[6.8 - 6.11]

SANS 5862-1:2006 [6.8] states that the slump test is applicable to plastic and cohesive concrete having a measured slump greater than 5 mm or less than 175 mm. The method is not suitable for concrete with stone larger than 40 mm in size.

SANS 5862-2:2006[6.9] states that the flow test is suitable for concrete of high workability with a slump greater than 150 mm and a stone size of 19 mm or less.

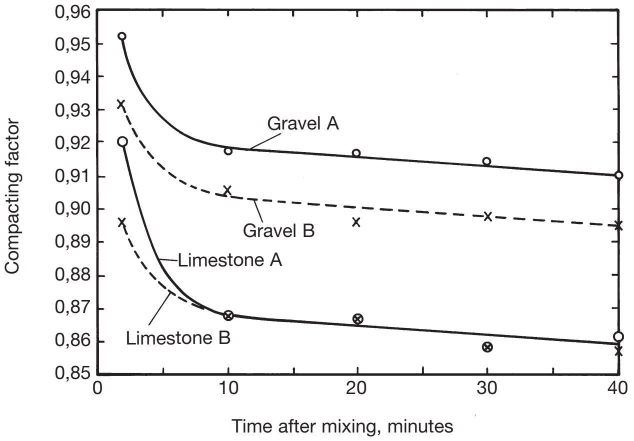


Figure 6.1: Workability of concrete versus time[6.5]

Coarse aggregates:	gravel and limestone
Fine aggregate:	river sand
Aggregate:cement ratio:	6:1
Water:cement ratio:	0,6

Allowance for absorption of aggregates:

A: Extra water added at time of mixing (simulating the use of a dry aggregate)

B: Sufficient water added to dry aggregate to give saturation 30 minutes before mixing

Mixing and ambient temperature: 18°C

Evaporation prevented throughout test

Table 6.1 Recommended ranges for test methods

Workability	Method
Very low	Vebe time
Low	Vebe time, compacting factor
Medium	Compacting factor, slump
High	Compacting factor, slump, flow
Very high	Flow

SANS 5862-3:2006[6.10] states that the Vebe test is valid for concrete having a slump of 10 mm or less and containing stone of maximum size 40 mm. The test is suitable for concretes having a Vebe time of between five and 30 seconds.

SANS 5862-4:2006[6.11] simply states that the compacting factor and compaction index tests apply to concrete of low, medium and high workability.

Other methods of test have been devised but few of these have been widely accepted.

No unique relationship exists between results of the various tests as each test measures the behaviour of the concrete under different conditions. An indication of the relationship between slump, Vebe and compaction factor is given in Figure 6.2 but the relationship is applicable only to the specific mixes used.

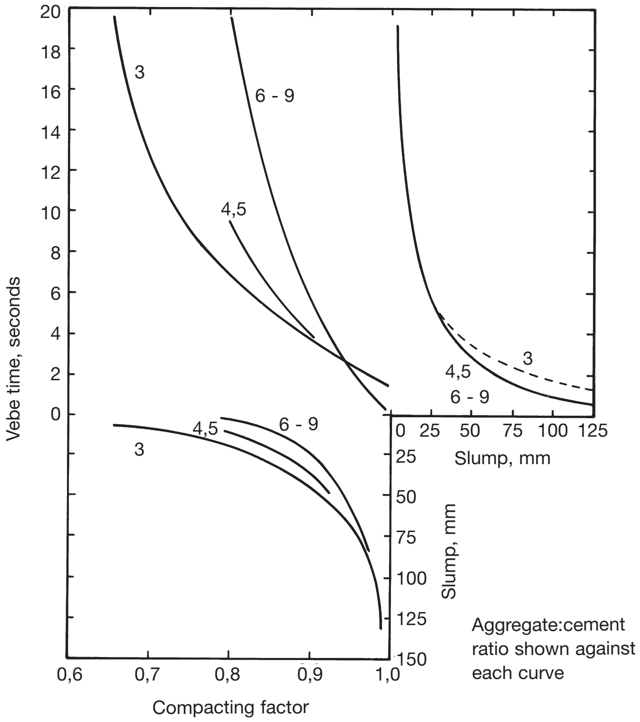


Figure 6.2: Relationship between slump, compacting factor and Vebe time[6.12]

The development of stable, highly workable mixes with slumps in excess of 180 mm has led to extensive use of the flow table test in Europe. In Asia, the slump flow test is widely used because of its simplicity and has been incorporated in Japanese standards.[6.13] In this test the flow is simply the measurement of the diameter of the concrete after it has collapsed in the normal slump test.

Domone[6.14] carried out a series of tests to evaluate highly workable mixes and confirmed the following:

- There is a good relationship between slump and slump flow results up to a slump of about 200 mm.
- Above this value the slump test is insensitive to increased workability.
- There exists a good linear relationship between slump flow and both initial spread and final spread after jolting in the flow table test.
- The linear regression gave the relationship: Flow table = 0,73 x slump flow + 240; the correlation coefficient was 0,88.

This work would suggest that if a single-point test is to be used, the slump flow should be preferred for convenience and simplicity.

Slump, Vebe, compacting factor and flow tests, as well as some lesser-known tests, are discussed below.

Slump test

Details of the test are given in SANS 5862-1:2006.[6.8] Details of the mould are shown in Figure 6.3.

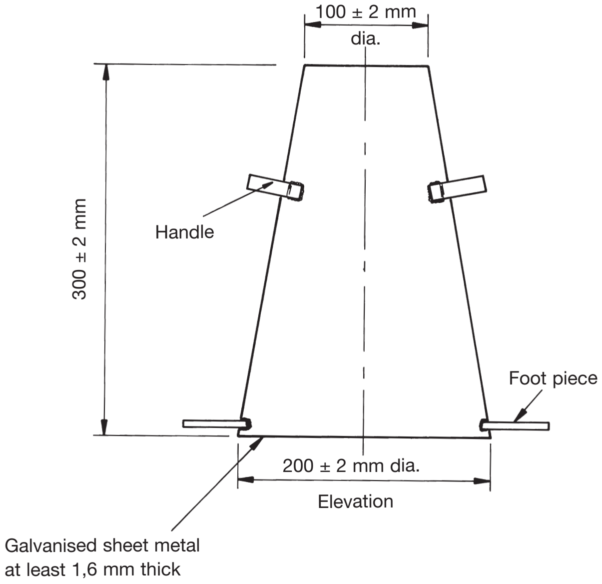


Figure 6.3: Mould for slump test[6.8]

The dampened mould is filled in three approximately equal layers. Each layer is subjected to 25 blows from the tamping rod, the mould being firmly held down by standing on the foot pieces. The blows should be evenly distributed over the whole area of the layer; for the second and third layers, the rod should just penetrate the previous one.

The surface is struck off by rolling the tamping bar across the top edge of the mould, not by applying a vigorous trowelling action with a float. After careful removal of the mould, the

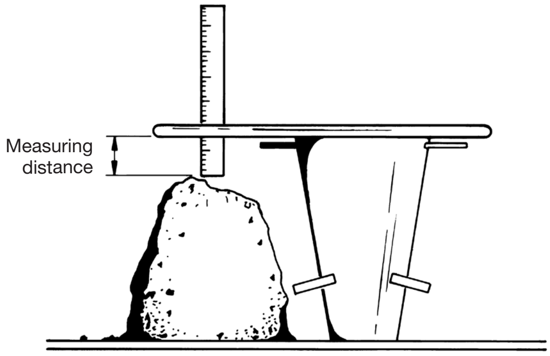


Figure 6.4: Measuring the slump

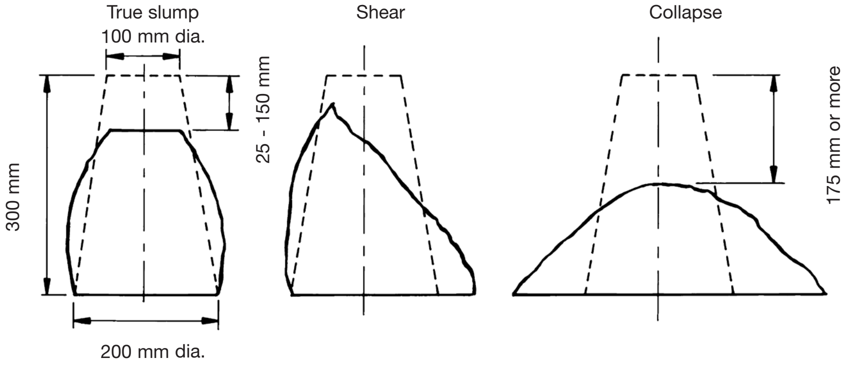


Figure 6.5: Types of slump

slump is measured to the nearest 5 mm, the slump being the distance between the top of the inverted mould and highest point of the concrete, as shown in Figure 6.4.

The more common forms of slump are shown in Figure 6.5. Where the slump is not normal, this should be noted.

The occurrence of a shear slump is an indication of lack of cohesiveness of the mix if it persists on repeating the test. Shear slumps may also occur with lean mixes and when rodding has been carried out unevenly.

Some indication of the cohesiveness and workability of the mix may be obtained by lightly tapping the slumped concrete with the tamping bar, or tapping the base plate. Workable, well-proportioned concrete will gradually slump further in a single mass while a concrete lacking cohesiveness is likely to fall apart. The effort needed to deform the concrete is also an indication of workability. With two mixes of the same slump, one mix may be assessed as being far more workable than the other.

Vebe test

The method is described in SANS 5862-3:2006[6.10] and the basic apparatus is shown in Figure 6.6.

A standard slump mould is placed in the cylinder, which is clamped onto a vibrating table which has a prescribed frequency and amplitude.

The mould is filled as for the slump test but via the funnel to prevent spillage. The funnel is removed, the surface of the concrete levelled and the mould removed. The slump, if any, may then be measured.

The transparent disc rider is placed on the surface of the concrete and the vibrating table is activated. Vebe time, measured in seconds, is the time taken to completely compact the concrete. Complete compaction is assumed to have been achieved when the transparent disc is totally in contact with concrete. In some cases this is difficult to judge, especially when short times are recorded. Adequate lighting is essential to help judge the end point of the test.

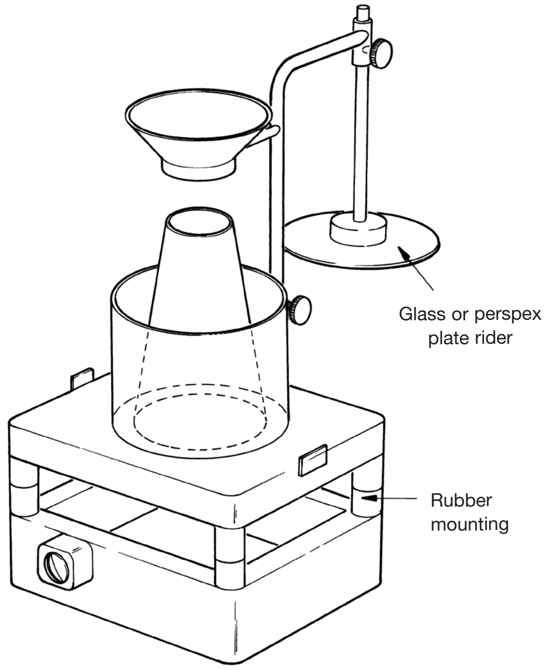


Figure 6.6: The Vebe consistometer[6.12]

The Vebe test is most suitable for dry mixes. (The slump test cannot differentiate between no-slump concretes of varying workability; error may be created in the compacting factor test by dry mixes sticking in the hoppers.)

Compacting factor test

The method is described in SANS 5862-4:2006[6.11] and the apparatus is shown in Figure 6.7.

In the test, the degree of compaction achieved by a standard amount of work is determined. The degree of compaction, ie the compacting factor, is a ratio of density achieved in the test to the density of the concrete when fully compacted.

The test is carried out with the interior surfaces of the hoppers and cylinder in a damp condition. At the start of the test, the two trap doors are closed and two floats are placed

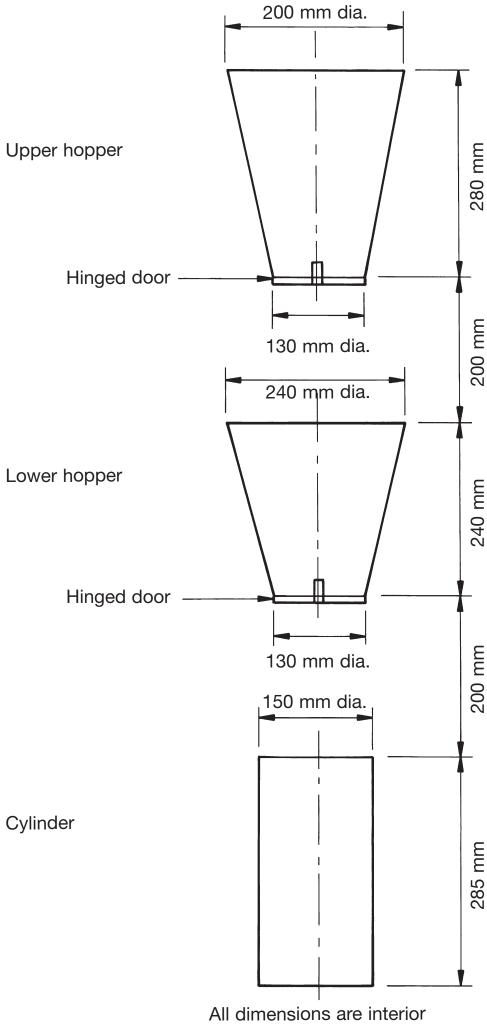


Figure 6.7: Dimensions of compacting factor apparatus[6.11]

over the top of the cylinder. The upper hopper is gently filled up to the rim with concrete. The trap door is opened, allowing the concrete to fall into the lower hopper. The two floats are removed from the cylinder and the lower trap door opened, allowing the concrete to fall into the cylinder. Excess concrete is cut from the top of the cylinder using the two floats. The mass of the partially compacted concrete (mp) is determined.

The concrete is emptied from the cylinder, which is then filled in six equal layers with concrete from the same sample, each layer being fully compacted using either a tamping bar or vibration. When the top surface has been struck off level, the mass of the fully compacted concrete (mf) is determined. The compacting factor is calculated as (mp/mf) and is always less than one.

Mixes of low workability often stick in the hoppers and need to be forced through using a slump rod. This could have a substantial effect on the test result.

Alternatively, a compaction index may be determined in accordance with this method.[6.11] A standard container, with height = 400 ± 2 mm and base dimension of 200 ± 2 mm x 200 ± 2 mm, is gently filled with concrete which is struck off level with the rim. The height of concrete is the interior height of the container H_1. The concrete is then fully compacted and S, the distance from the top of the container to the compacted surface of the concrete, is measured. The compaction index is calculated from the formula:

$$\frac{H_1}{H_1 - S}$$

The compacting factor apparatus is heavy and inconvenient to use on site. In the UK it is often used for monitoring the consistence of paving concrete placed by machine but in South Africa the Vebe test is generally preferred for this application.

Flow Test

The method is described in SANS 5862-2:2006[6.9] and the apparatus is shown in Figure 6.8.

The test is suitable only for high-slump concrete (slump in excess of about 150 mm), for example for self-levelling/self-compacting mixes and concrete for mine-shaft linings. Such mixes normally incorporate a superplasticising admixture. In the test, a dampened mould is centrally placed on the dampened table. The cone is filled with concrete in the prescribed manner, the concrete struck off level with the top of the mould and any superfluous concrete is removed.

After removal of the mould, the table top is lifted to the stops without jolting, and allowed to drop. This cycle is done 15 times, each cycle taking three to five seconds. The concrete spreads with each drop, and flow is determined as the average diameter of the concrete, measured in two directions parallel to the table edges.

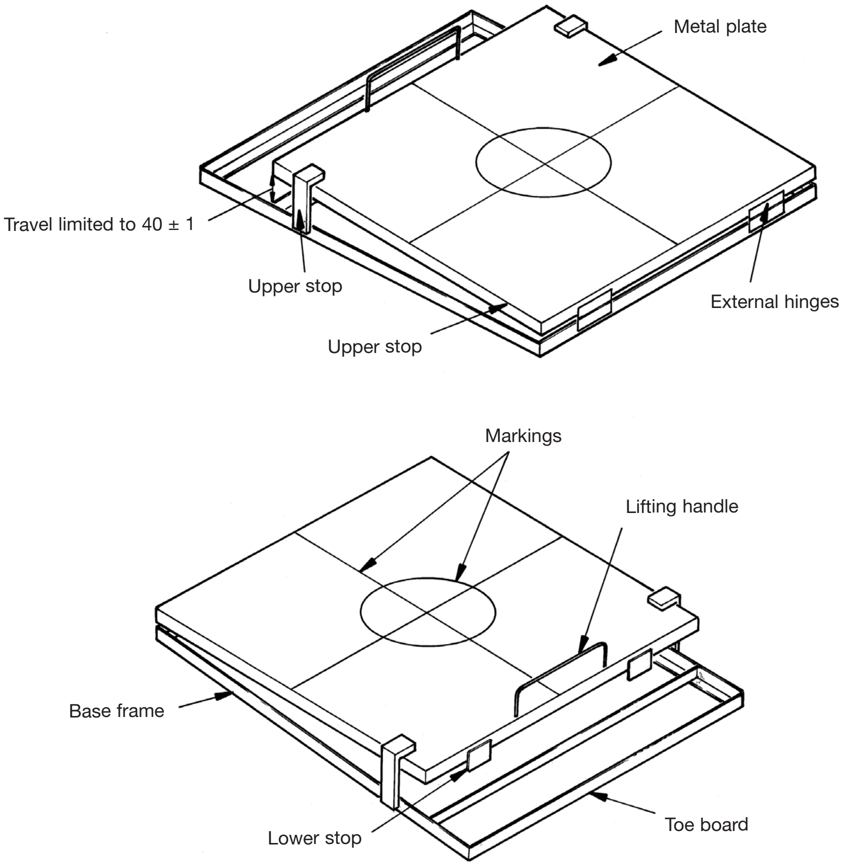
An assessment of the cohesiveness of the mix can be made:

- If stone particles separate from the matrix, the mix is too stony and the test is not appropriate.
- If a wide band of mortar is observed along the outside edge of the spread concrete, the mix is too sandy.

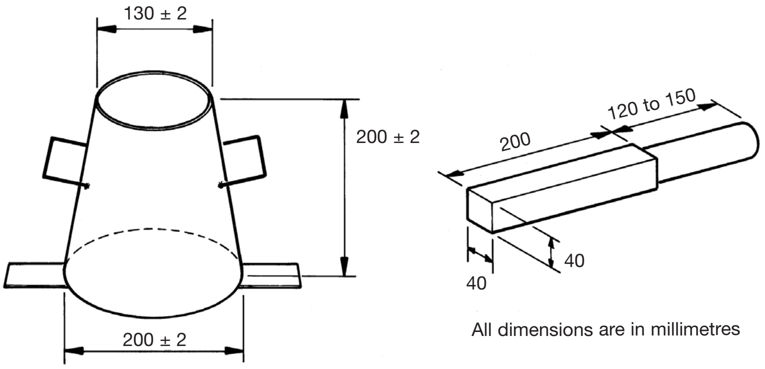
Miscellaneous tests

Ball penetration test

This test was developed by J W Kelly and is also known as the Kelly ball test. In the test, a 152-mm diameter metal ball with a mass of 14 kg is placed onto the surface of compacted concrete, and its penetration measured. It is claimed to be simple and quick to perform, and can be carried out at any time in the concreting cycle.



Typical flow table



Concrete mould

Tamping bar

Figure 6.8: Apparatus for flow test[6.9]

K-Slump tester

The K-slump tester was developed by Nasser[6.15, 6.16] as a simple and quick means of checking consistence. A good correlation is claimed between K-slump value and slump measured using a standard mould; it is further claimed to provide a guide to the workability of the concrete. In the test, the tube is inserted into the fresh concrete as far as the collar. The openings allow mortar from the mix to enter the tube. After one minute the measuring rod is lowered to rest on the mortar to give the K-slump value. The measuring rod is then raised, the apparatus removed from the concrete and the rod again lowered to the mortar surface. This latter value gives a workability index.

6.3.2 Control of consistence

For a mix of given materials and properties, consistence is affected mainly by the water content. Generally, consistence tests are done in order to control the water content and thus the water:cement ratio, by specifying limits for test results. If the values obtained are outside the limits, an adjustment to directly added water content should be made. Particularly in the case of the slump test, an assessment of cohesiveness and finishability can be made on the slumped concrete. A check should also be carried out to determine whether the change in slump was due to, eg, a change in raw material properties, type or source.

Tolerances for results of consistence tests, as given in BS EN 206-1:2000[6.4] are shown in Table 6.2.

Table 1 of SANS 878:2004 *Ready-mixed concrete*[6.17] specifies slump tolerances for concrete for a period of 30 minutes from arrival at the job site. It also specifies the allowable tolerance for air content of air-entrained concrete. These tolerances are given in Table 6.3.

Table 6.3: Tolerances for slump and air content of ready-mixed concrete[6.17]

Specified slump, mm	Tolerance, mm
50 and less	-15 to +25
More than 50, up to 100	± 25
More than 100	± 40
Note: Where applicable, the allowable tolerance in air content shall be ± 1,5%	

6.4 Cohesiveness

In the definition of workability it is stated that the concrete should retain its homogeneity, ie it should not segregate. If it does, full compaction and an adequate surface finish are unlikely to be achieved. In order for concrete not to segregate it needs to be cohesive. Cohesive concrete is able to hold both water and coarse aggregate within its mass so that their distribution remains uniform.

Segregation can therefore be defined as separation of the constituents of the fresh concrete resulting in non-uniform proportions. Normally, segregation occurs because the coarse aggregate separates out, creating zones with an abnormally high stone content; this is likely to lead to zones of incomplete compaction and honeycombing. Alternatively, stone, especially if it has a particle relative density substantially higher than that of the sand, may settle within the mixture, creating a thick layer of mortar and laitance at the top surface which could lead to cracking and dusting.

Wet mixes with poorly graded aggregates and a high stone content are most prone to segregation.

Chapter 12 gives examples of how segregation may occur at various points in the concrete handling process. Although

Table 6.2: Tolerances for concrete consistence determined by means of various tests[6.4] (Water content of concrete does not need adjustment if test result is within tolerance.)

Slump			
Target value, mm	≤ 40	50 - 90	≥ 100
Tolerance, mm	± 10	± 20	± 30
Vebe time			
Target time, seconds	≥ 11	10 - 6	≤ 5
Tolerance, seconds	± 3	± 2	± 1
Degree of compactability (Compaction index)			
Target value	≥ 1,26	1,25 - 1,11	≤ 1,10
Tolerance	± 0,10	± 0,08	± 0,05
Flow diameter			
Target value, mm	All values		
Tolerance, mm	± 30		

difficult to measure, segregation is easily recognised and steps may have to be taken to improve the cohesiveness of the concrete.

Cohesiveness can be increased by using one or more of the following measures:

- For very dry mixes, increase the slump.
- For very wet mixes, reduce the slump, but the concrete must still be sufficiently workable to allow for full compaction.
- Increase the fines content by the use of a finer blending sand or increase the cementitious content.
- Use CSF in the mix.
- Use GGCS in the mix.
- Use less stone, graded stone, smaller stone or a stone of lower particle density.
- Use an air-entraining agent.

Note that many of these adjustments will require other modifications to the mix proportions to maintain the correct water:cement ratio and yield.

6.5 Bleeding

Bleeding is a form of segregation in which some of the mixing water rises to the surface of the concrete as the solid materials settle. Bleeding may result in a layer of clear or slightly green water on the surface and will continue until the cement paste has stiffened sufficiently to prevent further settlement. (The absence of such a layer of water does not necessarily indicate that the concrete is not bleeding.)

The use of retarding admixtures and high additions of GGBS or FA will prolong the setting time and thus increase the time during which bleeding may occur.

Many defects can be attributed to excessive bleeding. These include:

- The formation of voids under aggregate particles and reinforcing steel as water migrates upwards.

  This reduces the strength and increases the porosity of the hardened concrete and reduces bond between the concrete and reinforcement.
- Sand streaking, where water runs up the form face, taking with it finer sand and cement particles, and leaving an unsightly finish.
- Surface laitance in which water and fine particles migrate to the surface of the lift.

  If laitance is trapped by the following placement of concrete a weak layer may be formed. In slabs there is a danger of this material being trowelled into the concrete, resulting in a porous, dusty surface.
- Settlement cracking (see section 6.6.2).

The rate of bleeding and the total bleeding capacity of a concrete mix can be determined by using the method given in ASTM C232-92.[6.18] Concrete is compacted in a container and bleed water is drawn off at intervals, using a pipette. The volume of bleed water can be expressed in terms of unit area of surface and percentage of net mixing water.

Bleeding may be reduced by modifying the mix in the following ways:

- Using a finer cement
- Using CSF or very finely ground GGBS
- Reducing the water content
- Increasing the proportion of minus-300-μm material in the mix
- Using an air-entraining admixture

Measures which may be taken on site to reduce the detrimental effects of bleeding include:

- Delaying power trowelling until bleeding has stopped and bleed water has evaporated or been otherwise removed
- Avoiding overworking of the surface
- Removing laitance by washing and brushing
- Recompacting concrete as discussed in section 12.9.3
- Vacuum dewatering

6.6 Plastic cracking

Plastic cracking typically occurs within half an hour to three hours of placing and finishing the concrete. There are two forms: shrinkage and settlement.

6.6.1 Plastic-shrinkage cracking

Although not confined to slabs, these cracks often occur in both reinforced and unreinforced horizontal slabs or pavements.

The principal cause of these cracks is the rapid removal of water from the concrete. In the case of concrete slabs placed in situ, water loss is mainly from the exposed surface of the concrete. When the rate of evaporation exceeds the rate of bleeding, the surface concrete loses water and decreases in volume. Tensile stresses are induced in the surface concrete because of restraint by the non-shrinking inner concrete.[6.19] When these stresses exceed the tensile strength of the surface concrete, cracks develop.

The rate of evaporation of water from the concrete surface increases with:

- Higher concrete temperatures
- Higher ambient temperatures
- Lower ambient relative humidities
- Higher wind velocities

In addition, water may also be absorbed by dry subgrade or absorbent formwork in direct contact with the concrete. This may aggravate water loss and, if shrinkage is restrained by subgrade or formwork, increase the likelihood of plastic-shrinkage cracking.

If the rate of evaporation exceeds 0,5 kg/m^2/hour,[6.20] exposed concrete surfaces should be protected to prevent plastic cracking. It should be noted however that plastic cracking may occur in cool weather with high wind velocities as well as in warmer conditions.

Typically, plastic-shrinkage cracks are roughly straight but discontinuous and parallel, spaced 0,3 to 2 m apart. They do not normally extend to the edge of the slab but often penetrate right through the depth of the slab. They usually appear within about two hours after compacting but are often not noticed until the following day.

Practical measures necessary to minimise plastic shrinkage cracking are discussed in section 12.11.1.

6.6.2 Plastic settlement cracking

After concrete has been compacted, there is a tendency for solid particles to settle and displace some mixing water which rises to the surface. This settlement will continue until the concrete stiffens.

In a section where there is no restraint such settlement rarely causes any problems. However, restraint may be present in the form of:

- A change in section, eg at the junction of web and flange in a T-beam or coffer slab
- Top reinforcement in beams or slabs
- Links or stirrups in columns
- Form ties
- Bridging of coarse aggregate between the form faces in narrow sections

Where horizontal reinforcement is present, settlement immediately above the steel is far less than below it. Thus such cracks often form immediately in the vicinity of the reinforcing steel and on flat slabs the crack pattern may virtually map the position of the top steel.

Because plastic settlement cracking is associated with bleeding, decreasing the bleed potential as described in section 6.5 will reduce settlement. The detrimental effects of plastic settlement may be overcome by revibrating the concrete (see section 12.9.3).

6.7 Slump loss

With time, all fresh concrete undergoes a reduction in consistence known as slump loss. This is a normal phenomenon and gives rise to problems on site only when the concrete becomes too stiff to place properly and compact fully. It tends to be less of a problem with site-batched concrete, where transport times are short and concrete manufacture can be stopped quickly, than with ready-mixed concrete where long haulage times are common and delays on site affect large volumes of concrete.

Slump loss is mainly due to:

- Absorption of water by (dry) aggregates or by surfaces in contact with the concrete
- Loss of water by evaporation
- The cement hydration process

Slump loss is further aggravated by high ambient and concrete temperatures and rich mixes.

Previte[6.21] showed that the amount of slump loss is proportional to the initial slump, ie the higher the initial slump, the higher the slump loss, and that conventional water-reducing admixtures do not have a significant effect on slump loss. The results also indicated a higher rate of slump loss with higher temperatures. Rash[6.22] recorded substantially greater slump losses at a temperature increase of only 6ºC, from 28ºC to 34ºC, and also found that the use of 25% FA mixes had a slightly higher rate of slump loss than the control mixes.

To overcome problems caused by slump loss, concrete is often batched at a higher initial slump, but as shown above the rate of slump loss then increases. Where supervision is poor the practice of retempering – adding water to restore workability – is often adopted, but this usually has the effect of reducing concrete strength.

The use of superplasticisers to regain initial slump does not result in lower concrete strength because the water:cement ratio is unchanged. It is however an option only for concrete delivered in mixer trucks.

In order to minimise slump loss, the concrete temperature may be reduced and site planning should be such as to minimise delays in handling the concrete.

6.8 The "green" state

For some hours after being mixed, concrete remains plastic and workable, although with time it loses slump (see section 6.7). Then, gradually, concrete starts to set, ie change from a plastic material to a weak solid. Further setting changes the concrete into a rigid solid, namely hardened concrete.

Concrete in the weak solid state is known as "green" concrete. The green state has practical importance because certain operations can be carried out on it only while it is in this state. (See section 12.10.)

6.9 The density of fresh concrete

The density or unit mass of concrete is affected mainly by the density of the raw materials, water content, air content and degree of compaction.

Typically the mass of one cubic metre of well compacted concrete ranges from about 2 360 kg using granite aggregate to about 2 600 kg using dolerite aggregate.

A change in water content of 10ℓ/m^3 will alter the density by about 15 kg/m^3; an increase in air content of 1% will reduce the density by about 25 kg/m^3.

6.9.1 Measuring the density of fresh concrete

SANS 6250:2006[6.23] requires that the cylinder used has a capacity of at least 0,01 m^3 and has a known capacity V (m^3) and mass M_1 (kg). Concrete is fully compacted in the cylinder by hand tamping or vibration and the surface carefully struck off level with the rim. Any excess material on the cylinder is removed before the mass of the cylinder plus compacted concrete M_2 (kg) is determined.

The density D, expressed to the nearest 10 kg/m^3, is calculated from the formula:

$$D = \frac{M_2 - M_1}{V}$$

Measurement of density can be used to confirm the mix proportions calculated using absolute volumes, and can also give an indication of the air content.

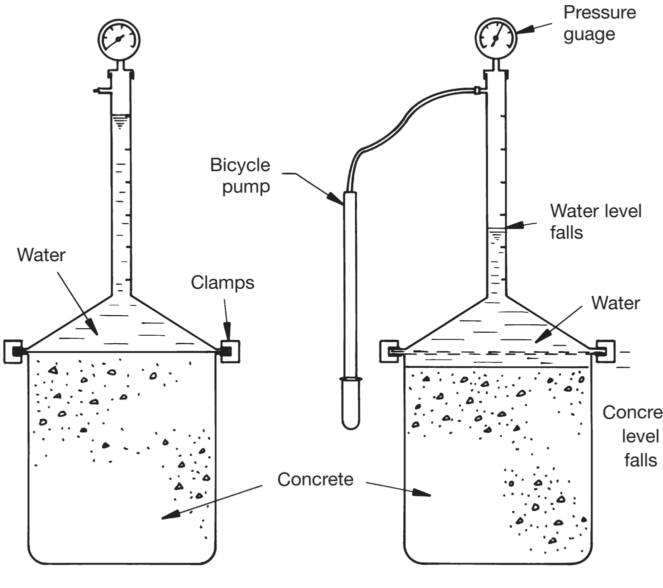
6.10 Air content of fresh concrete

Fully compacted concrete normally contains a small percentage of entrapped air, which further vibration would not remove. This air volume is typically 0,5 to 1%. When air is intentionally entrained in the concrete, eg when using an appropriate admixture, the air content must be continually monitored to ensure it is kept within prescribed limits (see section 5.2.4).

6.10.1 Measuring air content

The test is covered by SANS 6252:2006.[6.24] One type of apparatus is shown in Figure 6.9. The cylinder is filled with concrete, and the cover assembly fitted carefully to ensure a water-tight fit. Water is poured into the top of the apparatus to a zero mark and the cap fitted. A bicycle pump is used to increase the air pressure in the container to the calibration mark on the dial.

As air pressure is increased, the air in the concrete is compressed and this slightly lowers the concrete surface. At calibration pressure, the percentage of air A_1 is read; pressure is released allowing the concrete to recover its initial volume and this value (A_2) is read. The air content is A_1 - A_2%.



a) Before applying pressure b) With pressure applied

Figure 6.9: Diagram of pressure-type apparatus for determining the air content of air-entrained concrete

6.11 Conclusion

It has been stated that the various standard methods of measuring consistence do not measure workability, do not measure the same property of the fresh concrete and cannot differentiate between concretes having the same consistence but which can be seen to behave differently.

In his book on workability (see Further reading, below), Tattersall describes an apparatus which, it is claimed, can be used to ascribe changes in workability to changes in water content, admixture dosage or fines content. Such a test is, however, unlikely to find any widespread use in the near future except perhaps in research programmes.

Experienced technologists will continue to assess the workability of concrete by checking the consistence of the mix by traditional methods, and by observing the behaviour of the mix during the various stages of handling and finishing.

Further reading

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Chapter 7

Strength of hardened concrete

Bryan Perrie

7.1 Introduction

The strength of a material is defined as the ability of that material to resist stress without failure. The strength of hardened concrete is of fundamental importance to structural designers. It is also extensively used as an index of other concrete properties and of concrete quality. The concrete in a structure is subjected to multiaxial, highly variable stresses whereas when the strength of concrete is determined in the laboratory it is on small specimens manufactured, handled and cured differently to the concrete in the structure and subjected to a specific loading rate and almost uniaxial stress. The strength of concrete determined in the laboratory is only therefore an index or indication of the strength of the concrete in the structure.

The strength of concrete is time-dependent. Provided adequate moisture is available and that an appropriate temperature is maintained, concrete gains strength with time at an ever decreasing rate. Concrete strength is therefore generally specified and measured at a specific age.

The strength of any material may be determined in various modes: compressive, tensile (both direct and indirect) and shear. For concrete, compressive strength is the property most frequently specified and used for structural design, whereas flexural or tensile strength is important for the design of concrete pavements and fibre-reinforced concrete such as shotcrete.

The measured strength of a given concrete depends on many factors and for results to be comparable it is important that test methods be standardised.

This chapter discusses methods of measuring strength, the relationship between compressive and tensile strength, the mechanism of failure of concrete, factors which influence strength, and bond strength. A list for further reading is provided.

The scope is limited to the material aspects of concrete and excludes structural design and the statistical evaluation of the test results.

7.2 Methods of measuring strength

7.2.1 Compressive strength

The compressive strength of concrete is the calculated ratio of maximum possible uniaxial load, applied at a specific rate, sustained by the concrete, to cross-sectional area of a specimen, of constant cross-sectional area and with ends that are plane, parallel to each other and at right angles to the axis of the specimen, loaded between flat rigid platens. See Figure 7.1. Failure is defined as the maximum stress the sample can withstand even when no external signs of failure may be visible.

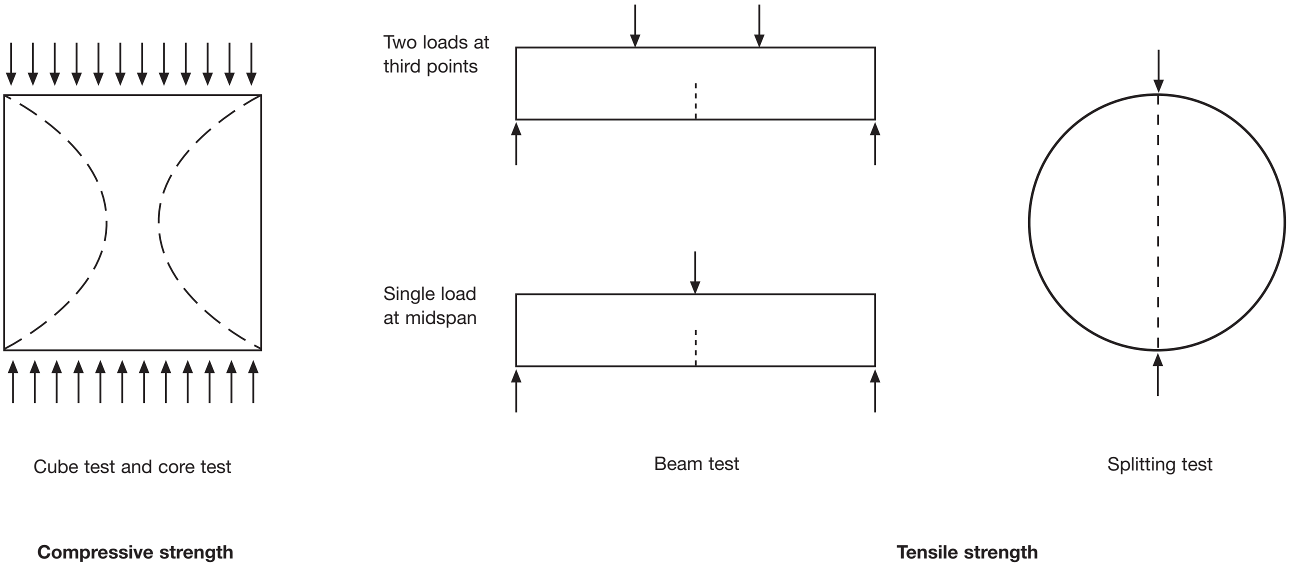

Figure 7.1: Diagrammatic comparison of tests to measure compressive and tensile strength. Dotted lines indicate cracking at failure.

The measured compressive strength of a given concrete depends on the intrinsic properties of the concrete and on the way in which it is tested. It is therefore essential that methods of determining compressive strength should be standardised.

In this section, standard South African test methods are briefly described and information is given on methods used elsewhere.

Cube test

In South Africa, the compressive strength of concrete, as determined in accordance with SANS 5860,[7.1] 5861-2,[7.2] 5861-3,[7.3] and 5863,[7.4] is used as a criterion of concrete quality and as an index of the strength of concrete in the structure.[7.5, 7.6]

The methods describe:

- Cube moulds: material, sizes, dimensional tolerances, design
- Tamper: mass, dimensions
- Sampling of concrete
- Testing machine: accuracy, bearing platens
- Preparation of specimens: moulding, curing
- Testing procedure: determining dimensions and mass, placing the specimen in the testing machine, rate of loading
- Calculating results: maximum stress, criterion for validity of test

In essence, the compressive strength test consists of crushing three cubes from the same sample of concrete. The cubes are tested in a saturated condition. The strength of the concrete is defined as the average of the strengths of the three cubes. For the test to be valid, the range of strengths within the set of three cubes must not exceed 15% of the average.

Core test

An indication of the compressive strength of concrete in the structure may be determined by testing cores removed from the structure. Cores are crushed after they have been trimmed to the correct length and their ends capped or ground flat. The method is described in SANS 5865.[7.7] The core test may be carried out when cube results are unsatisfactory or to assess the strength of concrete in an existing structure.

The method describes:

- Apparatus: core drill, saw, grinder, capping plates, melting pot, thermometer, and compression testing machine
- Materials: capping materials
- Drilling procedure: number of cores, core diameter, length, marking and inspection
- Preparation of cores: measurement, trimming, preparation of core ends, immersion in water
- Determination of compressive strength: placing the specimen in the testing machine, rate of loading
- Calculating results: area of bearing surface, correcting for reinforcement in the specimen, calculating measured compressive strength
- Report: information to be included

Results of core tests should be interpreted according to the criteria given in section 14.4.3 of SANS 10100-2:1992.[7.6]

Cubes and cores made from a given concrete and tested in accordance with the standard methods mentioned above will achieve different strengths due to various factors which are described in more detail later in this chapter.

Test methods used in other countries

Methods used to determine strength for the control of concrete quality and as an index of in-situ strength differ from country to country. Apart from differences in detail, there are essentially two methods, differentiated by specimen shape. A cube is used in some countries, eg the UK. A cylinder, with height being twice diameter, is used in countries such as the USA and Australia. The curing requirements for specimens may also differ from country to country.

It is important to note that cubes and cylinders made from a given concrete will achieve different strengths. This is because both direction of testing and height-to-width ratio are different for cubes and cylinders. Direction of loading, relative to direction of casting, is at right angles for cubes and parallel for cylinders. The height-to-width ratio is one for a cube and two for the cylindrical specimen specified in the test methods. The difference in strength is due to the frictional forces which develop between the platens of the testing machine and the contact faces of the test specimen. These end forces produce a multiaxial stress state which increases the measured compressive strength of the concrete. The multiaxial stress effects are significant throughout the cube; in the cylinder, however, the height-to-width ratio is large enough for the mid-height region to be reasonably free from these effects.[7.8]

The overall effect of these differences is that measured cube strength is greater than measured cylinder strength for the height-to-width ratios mentioned above. The ratio of these strengths is not constant; cube strength is generally 15 to 20% higher than cylinder strength for the same maximum dimension. The mechanisms responsible for these differences are discussed in section 7.5.3.

7.2.2 Tensile strength

It is generally impracticable to measure tensile strength directly due to the difficulty in applying a truly concentric tensile load on the specimen. In South Africa, as in the UK and USA, tensile strength is determined indirectly by breaking specimens (beams) in flexure or by splitting specimens by applying (nominally) line loads. See Figure 7.1. The point of failure is normally indicated by fracture of the specimen.

Beam or flexural test

This test is normally required only for concrete used for floors on the ground or road and airport pavements but may also be required in structural elements such as unreinforced dolos units where the flexural strength is critical. The method of test is described in SANS 5864.[7.9]

The method describes:

- Testing machine: accuracy, loading rollers
- Testing procedure: determining dimensions, placing the specimen in the testing machine, rate of loading
- Calculation of results

In essence, the beam test consists of subjecting specimens to loading, either by two loads at third points or a single load at midspan. The load at failure is used to calculate the theoretical maximum tensile strength on the assumption that stress varies linearly with distance from the neutral axis.

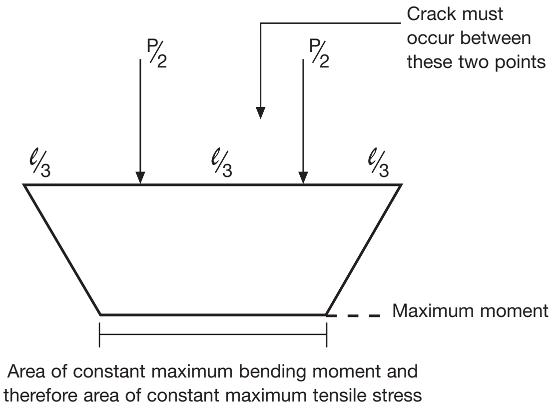

Figure 7.2 Bending moment for third-point loading

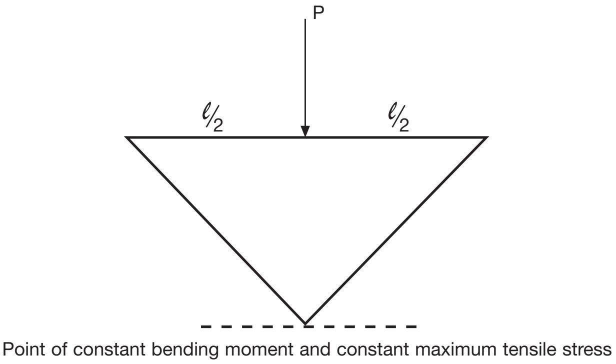

Figure 7.3 Bending moment for centre-point loading

For third-point loading a constant bending moment and therefore a constant maximum tensile stress is produced in the zone between the upper roller bearings (Figure 7.2), whereas for centre-point loading the maximum bending moment and therefore maximum tensile stress only occurs at the centre of the beam (Figure 7.3). When failure does not occur at the position of maximum tensile stress, the standard formulas for calculating the flexural stress are theoretically not valid, and the bending moment and stress at the actual point of failure should be determined from first principles.

Therefore:

for loading at third points $f = \frac{P\ell}{bd^2}$ .... (1)

for midspan (centre-point) loading $f = \frac{3P\ell}{2bd^2}$ .... (2)

where

f = flexural strength, MPa

P = breaking load, N

ℓ = distance between axes of supporting rollers, mm

b = width of specimen, mm

d = depth of specimen, mm

Variations of the beam test

Standard flexural strength testing is carried out using equipment in which the rate of loading is controlled. Similar equipment and tests exist in which the rate of displacement or deflection rather than the rate of loading is controlled.

This deflection-control equipment is used when assessing materials such as foamed concrete when the foamed concrete is used for its ability to deform under loading.

The deflection-control equipment is also used to determine the flexural strength and ductility/post-crack stiffness of fibre-reinforced concrete. Some methods include the EFNARC[7.10] round panel test and the Japanese shotcrete test.[7.11]

Splitting test

In this test a cylinder or cube specimen is subjected to compressive forces applied along two diametrically opposed lines. The method of test is described in SANS 6253.[7.12]

The method describes:

- Testing apparatus and ancillary equipment
- Testing procedure: placing the specimen in the testing machine, rate of loading
- Calculation of results

The principal tensile stress in the plane joining the loaded lines causes splitting in this plane. The tensile strength of the concrete is calculated according to elastic theory. [7.13, 7.14, 7,15] Therefore:

$$\text{for a cylinder } f = \frac{2P}{\pi d \ell} \quad \text{..........} \quad (3)$$

$$\text{for a cube } f = \frac{2P}{\pi a^2} \quad \text{..........} \quad (4)$$

where

f = tensile strength, MPa

P = compression load at failure, N

d = diameter of cylinder, mm

ℓ = length of cylinder, mm

a = size of cube, mm

7.3 The way concrete fails

Due to the complexity of the microstructure of concrete, the failure modes of concrete under stress are also very complex and vary with the type of stress and the internal structure of the specimen. Various theories have been put forward[7.16] to explain the mechanisms involved and of these Griffith's hypothesis of brittle fracture will be discussed here in more depth.

Concrete is considered to be a brittle material, even though it exhibits a small amount of plastic action.[7.17] A brittle material is one in which fracture occurs before plastic deformation in response to a tensile stress[7.18] and in which this leads to the formation of cracks that rapidly propagate.[7.19] In brittle fracture, these cracks form and propagate through the cross section of the material in a direction perpendicular to the applied stress[7.20] and involve breaking of atomic bonds that do not re-form.[7.21] Fracture in ductile solids is preceded by breaking and re-forming of atomic bonds as the atoms and molecules move relative to one another to first deform plastically, ie the atoms do not return to their original positions upon removal of stress.[7.21] According to Griffith's theory, the measured fracture strengths, ie resistance to failure under tensile stress, are substantially lower than expected in brittle materials due to small and omnipresent flaws in the material that serve as stress raisers. These stress raisers may be minute surface or interior cracks (microcracks), internal pores and grain corners that always exist under normal conditions at the surface and within the interior of a body of material. These flaws are a detriment to the fracture strength because an applied stress may be amplified or concentrated at the tip, the magnitude of this amplification depending on crack orientation and geometry as shown in Figure 7.4. They are more significant for brittle than ductile materials because of the inability of brittle materials to deform plastically under stress, and better distribution of stress in the vicinity of stress raisers in plastic materials.[7.22]

Griffith's hypothesis can also be extended to fracture under bi- and triaxial stress and also under uniaxial compression.[7.23] In compression more energy is needed to form and extend cracks in the matrix. According to Mehta[7.24] it is generally agreed that in a uniaxial compression test on medium- or low-strength concrete, no cracks are initiated in the matrix up to about 50%of the failure stress. However, at this stage, a stable system of cracks already exists in the vicinity of coarse aggregate. Neville[7.25] confirms that in normal-strength concrete very fine microcracks exist at the interface between coarse aggregate and cement paste prior to application of load on the concrete and that these remain stable up to 30% or more of ultimate load. At higher stress levels, cracks are also initiated within the matrix, their number and size increasing progressively with increasing

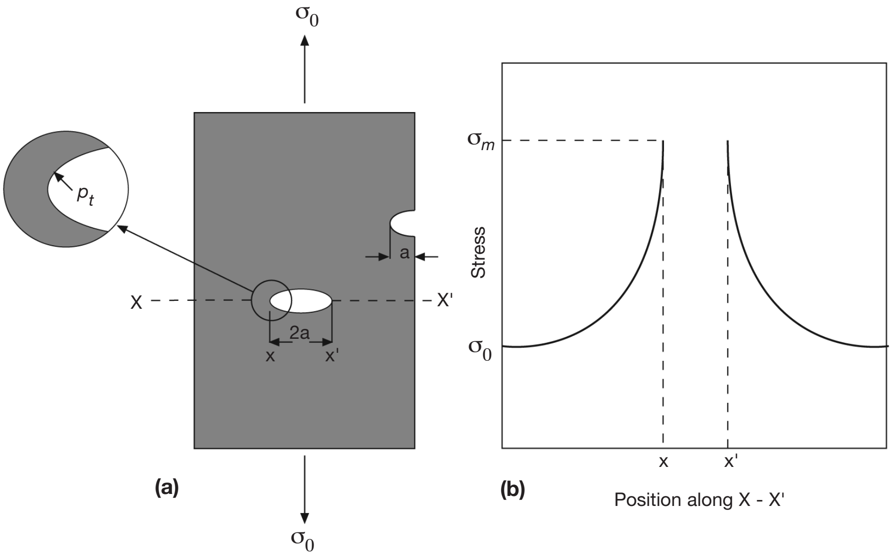

Figure 7.4 (a) The geometry of surface and internal cracks. (b) Schematic stress profile along the line X-X' in (a), demonstrating stress amplification at crack tip positions[7.22]

stress levels.[7.24, 7.25] The cracks in the matrix and the interfacial zone eventually join up, and generally a failure surface develops at about 20 to 30° from the direction of the load.[7.23, 7.24] This is in contradiction to Griffith's theory in which cracks develop perpendicular to the load, but as Neville indicates, it is possible that failure is governed by lateral strain induced by Poisson's ratio. The value of Poisson's ratio for concrete is such that for elements sufficiently removed from the platens, the resulting lateral strain can exceed the ultimate tensile strain of the concrete and that failure then occurs by splitting at right angles to the direction of load as in the tensile splitting test.[7.23] According to Neville the ultimate failure under the action of a uniaxial compression is either a tensile failure of cement crystals or of bond in a direction perpendicular to the applied load, or a collapse caused by the development of inclined shear planes.[7.26] It is clear that the nature of failure mechanisms in concrete is complex and not yet well established. Agreement does seem to have been reached that it could be linked to stress and strain intensification and breakdown of bond between the cement and the aggregate.[7.24, 7.26, 7.27]

7.4 The relationship between compressive and tensile strength

The compressive strength of concrete is much higher than its tensile strength. Furthermore, there is also no specific or simple relationship between tensile and compressive strength. The reason is that factors such as cement: water ratio, curing conditions, age, mix proportions and properties of the aggregate do not affect tensile strength and compressive strength to the same degree. It is also important when comparing compressive and tensile strength to bear in mind that different test methods give different results.

According to the Loedolff and Chambers[7.28] the relationship between the indirect tensile strength f_t and the compressive strength f_c of concrete is dependent on whether the indirect tensile strength is determined on cubes or cylinders. They reported the following relationships:

For cube splitting $f_t = 0{,}192 f_c^{0{,}802}$ and for cylinder splitting $f_t = 0{,}185 f_c^{0{,}783}$ and both strengths are in MPa.

The design of concrete road and airport pavements is based on the flexural strength of the concrete. According to the ACPA,[7.29] the flexural strength may be taken as $f_r = C(f'_{cr})^{½}$ where f_r and f'_{cr} are in MPa; f'_{cr} is the cylinder strength of the concrete; and C has a value of 0,75.

In a large number of tests carried out by the C&CI it was found that, for third-point loading, C ranged between 0,73 for crushed Witwatersrand quartzites and 0,48 for rounded river pebble aggregate. This work was based on cube rather than cylinder strength.

7.5 Factors which influence the strength of concrete

The observed strength of concrete in a given mode depends on the nature of concrete, intrinsic factors and on how the concrete is tested or extrinsic factors. In this section, factors which influence compressive strength will be discussed. Tensile strength would be similarly affected although not necessarily to the same extent.

7.5.1 The nature of concrete

Good quality concrete is a multiphase or composite and relatively porous material which includes various flaws of different types and sizes. In this section, each of these properties and its effect on strength is discussed.

Heterogeneity

Concrete in the hardened state consists of large and small aggregate particles (stone and sand) and hardened cement paste (HCP). It can be said that concrete consists of phases that differ from each other by having different properties. Taking the phases as aggregate and HCP, in normal concrete the aggregate is considerably stronger, stiffer and dimensionally more stable than the HCP. If size is used as a criterion for differentiating phases, stone particles are normally much larger than sand particles or elements of the HCP matrix between aggregate particles.

Concrete is thus markedly heterogeneous in terms of material properties and size. The significance of heterogeneity is that, when the concrete is loaded, internal strains and stresses are not uniform but tend to be higher in some locations, especially at interfaces between phases that have different properties. This concentration of stresses effectively weakens the material.

Porosity and flaws

All concrete is porous to some extent. A conventional structural-grade concrete, for example, has an overall porosity of about 16% even if well compacted and cured. This porosity is caused by the water in the fresh concrete. Typically more water than is necessary for cement hydration is present in the mix to ensure adequate workability. This unreacted water creates the pores.

The significance of porosity is that strength decreases with increasing porosity. This inverse relationship is not limited to cementitious materials but is generally applicable to a wide range of materials.[7.30] This is also the reason for the relationship that exists between water:cement ratio and compressive strength.

Flaws, in the form of microcracks and discontinuities, are normally present even in good quality concrete. Cracks in aggregates may be caused by the stresses developed during the crushing process or by stress relief in rocks obtained from great depths. The interface between HCP and aggregate is inferior, with respect to density and strength, to the bulk HCP (see section 1.12.1). Interfacial zones on the undersides of aggregate particles, especially coarse aggregate particles, are often further weakened by the accumulation of trapped bleed water.

Flaws are significant because, when the concrete is under load, internal stresses tend to be higher immediately adjacent to flaws. At high stresses, failure tends to initiate at flaws. The severity of this effect is influenced by the orientation of the flaw relative to the direction of loading and by the size of the flaw.

7.5.2 Intrinsic factors

Aggregate

The aggregate is generally the strongest component in the concrete. Using the 10% fines aggregate crushing test (SANS 5842)[7.31] as a measure of aggregate strength, Davis[7.32] investigated the effect of aggregate strength on the compressive strength of concrete. He concluded that:

- The compressive strength of normal structural-grade concrete made with the types of stone commonly used in South Africa (10% FACT of 160 kN and over) is not significantly affected by the crushing value of the aggregate.
- The 70-kN lower limit of 10% FACT (specified in SANS 1083[7.33]) appears to be realistic for most structural-grade concrete of compressive strength up to about 35 MPa. For concrete of strength up to about 50 MPa it would be preferable to raise this limit to about 140 kN.
- For very-low-strength concrete (up to about 15 MPa) the crushing value of the stone is of little importance except that some difficulty might be experienced in handling and mixing friable materials without causing excessive attrition.
- Only in the manufacture of very-high strength concrete will the crushing value of the stone become a significant factor. For instance, a nominal 80-MPa concrete made with stone having a 10% FACT of 140 kN would be weaker than the same mix made with stone having a 10% FACT of say 270 kN.

The degree to which the aggregate influences concrete strength depends on the strength of the concrete: the higher the strength, the greater the influence of aggregate on strength.

The effect of aggregate mineralogy, aggregate size and grading and other factors are covered in Chapter 3. The durability of aggregates can often be assessed by comparing both wet and dry values for the 10% FACT or ACV tests, and the wet value should be at least 75% of the dry value.

Aggregate-paste interface

The strength of the aggregate-paste interface is generally the weakest area in the concrete.

Interface strength is influenced by:

- Water:cement ratio of the paste
  Strength improves with decreasing water:cement ratio.
- Age of concrete
  Strength improves with age provided that sufficient moisture is available to sustain cementin reactions.
- Amount of bleeding of the fresh concrete
  Strength improves as bleeding is reduced.
- Type of cementitious material
  Materials containing very fine particles, eg silica fume, or those found in cement extenders and fillers, tend to develop a superior interface micro-structure because of the fine-filler effect. This effect is discussed in section 1.12.2.
- Ultrafines in aggregates
  Very fine aggregate particles reduce bleeding and may cause a fine-filler effect at the interface. Both phenomena would tend to improve strength of the interface.
- Surface texture of aggregates
  Interface strength increases with increasing roughness of surface texture.
- Absorbent aggregates
  Interface strength is increased when slightly absorbent aggregate is used.

Hardened cement paste

The strength of HCP strongly influences the strength of concrete.

Factors which influence the strength of HCP are:

Porosity

Strength increases with decreasing porosity.

HCP contains both gel pores, which form about one third of the volume of gel, and capillary pores. A detailed explanation of these is given in section 1.12.1.

Powers[7.34] found that compressive strength was related to the gel:space ratio, ie the ratio of the volume of gel to the original space available. Because this relationship is based on the overall volume of gel, only capillary pores are relevant.

Capillary porosity is a function of water:cement ratio and of the degree of reaction of the cement. Water:cement ratio can therefore be used as a parameter for the design of concrete for strength. The degree of cement reaction depends on the age of the paste, the availability of water, the type of cement and curing temperature.

In addition to pores, which are spaces originally filled with water, the paste may include voids. Such voids result from the failure to expel air from the fresh paste or from the deliberate entrainment of air in the form of very small bubbles. Voids contribute to overall porosity and have a significant effect on strength. In fact, strength is affected equally by capillary porosity and air-void porosity.

Microstructure

Although the effect of HCP microstructure on strength is not as pronounced as that of porosity, there are aspects of microstructure that can make a significant difference. In this section, the effects of pore size, crystalline solids and unhydrated cement are discussed.

Pore size

At a given porosity, strength of HCP depends on the size of pores in the paste.[7.35] Large pores may be caused by uneven dispersion of cement particles and by air bubbles. When cement and water are mixed in the conventional way, as they are in most concrete, cement particles do not disperse uniformly through the water but tend to form agglomerations, leaving relatively large spaces which do not contain cement. These spaces form large capillary pores in mature HCP.

In practice, even with good compaction, it is not possible to expel all of the air entrapped in the paste. Air may also be deliberately entrained in paste. Air bubbles cannot be filled by gel because gel can form only in water-filled spaces.[7.36] Large capillary pores, entrapped air and entrained air may all, because of their size, limit the strength of HCP.

Maximum pore size may be reduced by using admixtures, such as superplasticisers, to disperse cement particles more uniformly. The inclusion of very fine particles of limited solubility, such as found in cement extenders and aggregate ultrafines, provides nucleation sites for the formation of gel in spaces which do not contain cement. This fine-filler effect reduces the size of large capillary pores. The amount of air left in the paste may be reduced by applying increased compactive effort. All of these techniques can increase strength.

Crystalline solids

Crystalline solids, especially calcium hydroxide, are included in HCP. Although such materials are in solid form they do not contribute significantly to strength. For mixes with equal water content, the strength of HCP may be increased by the addition of a pozzolanic cement extender which reacts with the calcium hydroxide to form calcium silicate hydrate, thus converting a weak crystalline phase to a strong gel phase.

Unhydrated cement

Some unhydrated cement is normally present in HCP. This is either as a result of coarse cement particles not being fully hydrated or because of incomplete dispersion of the cement particles. The former is influenced by the particle size distribution of the particular cement and the latter is affected by plasticisers and the amount of mixing energy. Cement clinker is a hard strong material and, in HCP, would be firmly bound to the surrounding gel. Unhydrated cement, the cores of the original cement particles, therefore functions as a very fine aggregate. Because of its strength and the good bond to gel, unhydrated cement makes a contribution to strength, although this contribution is smaller than the effect of hydration which effectively converts water into a solid. Unhydrated cement plays an important role in the microstructure and strength of high-strength concrete.[7.37]

7.5.3 Extrinsic factors

The observed compressive strength of a given concrete is influenced by factors that relate to how it is tested. These factors are direction of loading relative to direction of casting, shape of specimen, specimen size, moisture content of specimens, rate of loading, eccentricity of loading, temperature, and testing machine issues. In the case of cores an additional factor is the size of aggregate relative to the core diameter. The effect of each factor depends on the microstructure of the concrete and the mechanism of failure and is to some extent specific to the type of concrete.

Direction of loading relative to direction of casting

The formation of lens-shaped flaws on the undersides of aggregate particles, especially coarse aggregate particles, resulting from the entrapment of bleed water, results in non-isotropic strength properties of the concrete. The concrete tends to behave like a horizontally-layered material with roughly horizontal planes of weakness.

Because the mode of failure under compression is mainly tensile, concrete will be stronger when the direction of load is perpendicular to, and weaker when the load is parallel to, these planes of weakness. In the standard test methods for compressive strength, concrete cubes are loaded parallel, and cast cylinders perpendicular, to the planes of weakness.

The difference in strength obtained by loading perpendicular or parallel to the direction of casting is specific to the concrete and no simple or general relation exists. In an extensive test programme, involving about 1 000 cores, carried out by the C&CI, it was found that the difference ranged between zero and 10%, with an average of 4%.

Shape of specimen

Because concrete fails primarily in lateral tension or splitting, and because the platens of the crushing machine provide some degree of lateral restraint, the shorter or squatter the specimen, the higher the observed strength.

An interesting demonstration of this shape effect is to load-test a stack of two cubes from the same batch and to compare the result with that of a single, similar cube tested conventionally. Tests done at the C&CI showed that the stack of two cubes indicated a strength of between 79% and 86% of the single cube strength, with an average of 81%.

Because of the shape factor, a cube will indicate a higher strength than a cylinder with length equal to diameter, if all else is equal (including the direction of loading as discussed above).

Table 7.1: Strength correction factors for different length: diameter ratios (Observed strengths to be multiplied by the tabulated factors to bring result to the standard cylinder of length:diameter ratio of 2,0)

Length: diameter ratio	ASTM Standard C42	British Standard	Sangha and Dhir
1,0	0,91	0,92	0,82
1,5	0,97	0,96	0,98
1,75	0,99	-	-
2,00	1,00	1,00	1,00
3,00	-	-	1,02

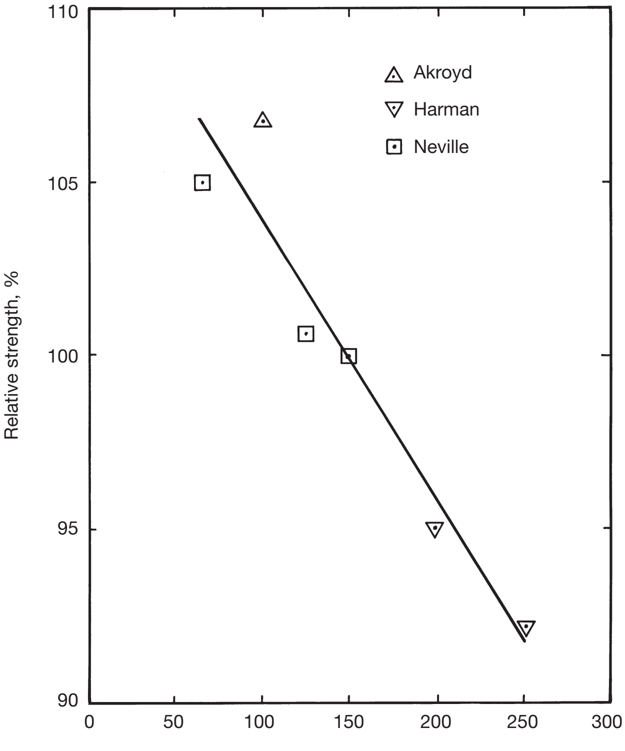

Figure 7.5: Strength of cubes of different sizes[7.43]

If cylinders of different length-to-diameter ratios are considered, the smaller this ratio the higher the observed strength. Malhotra[7.38] quotes the correction factors recommended by different authorities. These are shown in Table 7.1.

The relationship between shape and observed strength is not simple or constant but appears to be specific to the concrete concerned. It is therefore important that tests to measure this relationship should be carried out with the specific concrete being assessed.

Specimen size

As with many brittle materials, the apparent strength of concrete specimens and the variability of results decrease as the physical size of the specimens increases.[7.39, 7.40] As concrete contains a large number of flaws or cracks of various sizes, this effect has been explained by use of Griffith's crack theory and the weakest-link theory: failure is assumed to be caused by the propagation of minute cracks, the largest crack, like the weakest link in a chain, being responsible for complete fracture and failure.[7.41]

The bigger the specimen, the greater the probability that it contains a flaw that will induce failure at a given applied stress.

Derucher[7.42] for example states that a cylinder of 1 823-mm height by 914-mm diameter may be only about 82% of the strength of a standard (USA) cylinder of 305-mm height by 152-mm diameter. Neville[7.43] gives the results of work done by himself and others as shown in Figures 7.5 and 7.6. But in a limited investigation done in the C&CI laboratory, 100-mm and 150-mm cubes were found to have virtually identical seven and 28-day strengths when made with 19-mm andesite stone, sand derived from decomposed sandstone and ordinary portland cement used at a water:cement ratio of 0,6. It is possible that the magnitude of the effect of specimen size on strength could be specific to the concrete.

In the case of cores, the ratio of maximum aggregate size to core diameter has an effect on the measured strength.

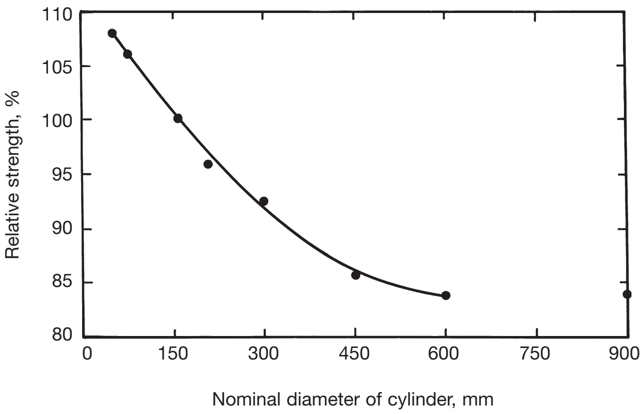

Figure 7.6: Strength of cylinders of different sizes[7.43]

Annexure 1 of EN 12504-1[7.44] indicates that for concrete containing 20-mm aggregate, 100-mm diameter cores were 7% stronger than 50-mm diameter cores, which in turn were 20% stronger than 25-mm diameter cores. SANS 5865[7.7] recommends that 100-mm diameter cores be used and that cores as small as 65 mm in diameter may be used, but that whatever diameter is used the diameter should be at least three times the maximum nominal aggregate size in the concrete.

Moisture content

The compressive strength of concrete increases as the concrete is allowed to dry out.

Mills[7.45] suggests that part of the free water that enters the gel structure exerts a positive weakening action which can be likened to a wedging action. Accordingly, if free water leaves the gel due to a depression of the vapour pressure in the capillaries, the opposite effect, that of increasing the apparent crushing strength of the concrete, may be expected. He found that oven-dry cubes could be 35% stronger than equivalent saturated cubes.

To standardise the conditions of test, SANS 5863[7.4] requires cubes to be crushed in a saturated condition; cubes may also not be allowed to dry out at any time before they are tested. Similarly, cores tested in accordance with SANS 5865[7.7] must be soaked in water for 48 hours and crushed immediately after being removed from the water.

The strength of specimens tested in direct tension or flexure decreases on drying mainly due to the formation of shrinkage cracks.

Rate of loading

The compressive strengths of cubes and cylinders increase with the rate of loading.

Newman[7.41] states that standard control tests are usually considered as static loading tests, but when the rate of loading or straining is no longer small compared with the velocity of stress waves through the concrete or the natural frequency of the structural unit under load, dynamic effects due to inertia become important. There can be no sharp distinction between the modes of behaviour at different loading rates, but general limits have been proposed to define various types of loading of concrete specimens:

- Long-term static loading-slower than 0,007 MPa/second
- Short-term static loading - 0,07 to 7 MPa/second
- Dynamic loading - faster than 70 MPa/second

Although at least 50% of the ultimate static load can be applied at any rate without affecting the ultimate strength, at higher loadings slow rates of loading can cause failure at stresses 75 to 80% of that under more rapid loading,[7.42] probably due to limiting rate of crack propagation.

SANS 5863[7.4] requires that cubes are to be loaded, without shock, at a rate of 15 MPa/minute (ie 0,25 MPa/second).

With repetitive or fatigue loading, however, the maximum stress must not exceed 50 to 55% of the short-term static strength if failure is not to occur.[7.41]

The effect of temperature at the time of testing

Temperatures within the range of room temperatures do not have any appreciable effect on the strength of concrete. Although the temperature at the time of testing will not influence the strength at failure, the curing temperature of the concrete significantly influences the rate of strength gain. See section 12.11.2 and Figure 12.14.

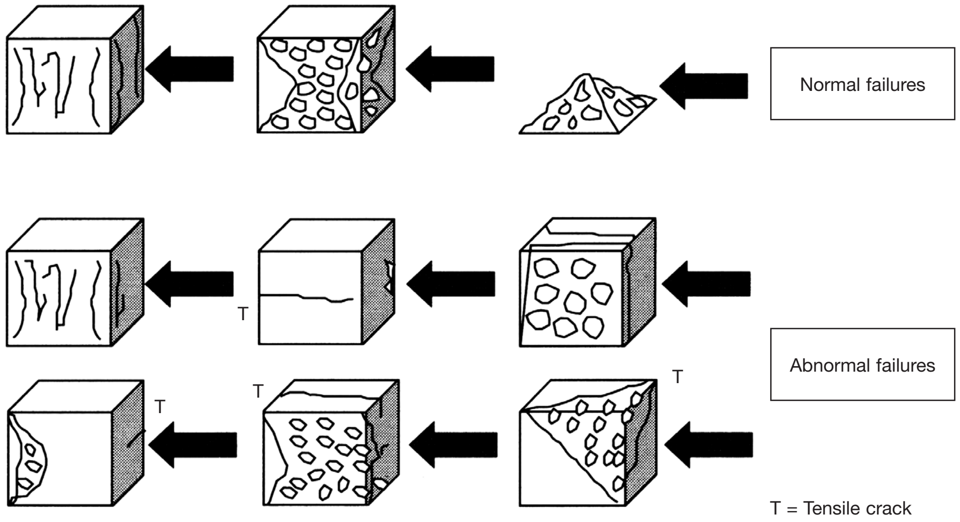

Figure 7.7 :Normal and abnormal cube failures[7.47]

Testing machine influences

Factors which may cause errors in the measured failure load in specimens include:

- Load indication
- Lateral and axial stiffness of the machine frame
- Design of the spherical seating
- Platen hardness, flatness and texture
- Specimen location
- Testing in appropriate range

To ensure that the first four factors listed above do not influence the measured failure load it is recommended that all testing machines comply with the requirements of EN 12390-4.[7.46] and an appropriate class of accuracy for the testing to be carried out.

It is important that the specimens be located correctly in the machine and that the mode of failure be observed and recorded (see Figure 7.7).

One-sided modes of failure are associated with reduced strengths and can be caused by one or more of the following:

- Placing cubes off-centre in the machine
- Faulty cube moulds resulting in eccentric cubes
- Insufficient lateral stability of the machine frame
- Premature locking of the ball seating
- Rotation of the ball seating under load
- Lack of planeness of cube surfaces or platens

It is important that the expected failure load of the specimens be within the range of the testing machine and not near the bottom or top of the range. For example, if the expected failure load is 10 kN, the specimen should not be tested on a machine with a range of 1 000 kN. It should be tested on a machine with a range of 100 or 200 kN.

Multiaxial loading

A detailed discussion of the behaviour of concrete subjected to biaxial and triaxial loading is outside the scope of this section. This aspect of concrete strength is dealt with in reference 7.48. The following brief notes, based mainly on this reference, may help the reader to understand better the implications of compressive strength testing and the significance of compressive strength as a property of concrete.

The standard compression test is commonly referred to as uniaxial because load is applied in one direction. This description is not strictly true because the lateral expansion of the concrete is restrained by friction at the specimen-platen interfaces. The specimen is thus subjected to a quasi-triaxial loading at these interfaces while loading at mid-height is closer to uniaxial. The stress state of the specimen is thus extremely complex.

The observed compressive strength of a concrete specimen depends therefore to some extent on the roughness of the platens or the coefficient of friction between platen and specimen. A condition of negligible restraint has been approximated in research laboratories by the use of brush platens. Triaxial compression tests have been done using steel platens to load the ends of a cylindrical specimen and a fluid confining pressure along the length of the specimen. The application of a confining pressure results in large increases in measured strength.

Although the above information could be considered to be academic, it does demonstrate that the standard test for compressive strength is done under arbitrary conditions and is not an attempt to model the stress-state or environmental conditions of structural concrete in service.

Nevertheless the compressive strength of concrete has been the basis for the successful empirical development and application of codes of practice for structural design for many decades.

7.6 Bond strength

Bond strength has been defined[7.49] as "resistance to separation of mortar and concrete from reinforcing and other materials with which it is in contact; a collective expression for all forces such as adhesion, friction due to shrinkage, and longitudinal shear in the concrete engaged by the bar deformations that resist separation."

Bond stress has been defined[7.49] as "the force of adhesion per unit area of contact between two bonded surfaces such as concrete and reinforcing steel or any other material such as foundation rock; shear stress at the surface of a reinforcing bar, preventing relative movement between the bar and the surrounding concrete." It is important to note that without bond, there is no load transfer between the steel and the concrete.

The following discussion is limited to the bond between concrete and reinforcing steel.

For plain bars, bond depends mainly on adhesion and friction. There is also a certain amount of mechanical interlocking due to the roughness of the bar. In the case of deformed bars, bond depends primarily on mechanical interlocking although adhesion and friction also play a part.

It has been pointed out[7.50, 7.51] that there are important differences between plain and deformed bars in the mechanics of bond and slip in concrete. As Roberts[7.51] has mentioned, a straight, plain, round bar can usually he pulled out of a block of concrete without bursting the block. On the other hand the surface shape of a deformed bar generates a radial bursting force that tends to split the concrete along the line of the bar; the degree to which this bursting is inhibited has a profound effect on the ultimate average bond stress. The European Concrete Committee [7.52] has emphasised that the limiting bond stress for calculating the safe length of the anchorage depends on the transverse reinforcement.

In prestressed and post-tensioned concrete, bond is dealt with differently to reinforced concrete. The tendons used for the stressing are sometimes bonded and sometimes unbonded. This is outside the scope of this chapter.

Determining bond strength

Methods consist essentially of testing specimens of hardened concrete with embedded steel rods. A force, which tends to move the rod(s) relative to the concrete, is applied and resulting bond stresses calculated. Such tests are outside the scope of standard testing; information may be found in references 7.53 to 7.60.

Factors affecting bond

It has long been recognised that bond between concrete and steel is dependent on the quality of the concrete. In 1913, Abrams[7.61] indicated that the bond stress that can be developed is proportional to the compressive strength of the concrete. He and others[7.61 7.62] subsequently noted, however, that above a certain limiting strength, bond is not proportional to the compressive strength of concrete; the ratio of bond to compressive strength decreases at compressive strengths greater than about 20 MPa.

The extent of the concrete cover to the reinforcing bars is important as it has an effect on splitting, particularly when deformed bars are used. Ferguson and Thompson[7.63] showed that for concrete cover greater than about 20 mm, the bond strength, for a given bar diameter and embedment length, increased linearly as the depth of cover increased. Not only the confinement provided by the concrete cover but also that provided by transverse reinforcement, such as stirrups, affects bond strength.[7.50, 7.53, 7.64]

Bond strength is also affected by settlement of the fresh concrete and it has been observed that the use of high-slump mixes results in poor bond strength. Menzel and Woods[7.65, 7.66] have attributed this to the concrete settling away from the undersides of the bars when the bars are held rigidly in position. Welch and Patten[7.67] carried out investigations on both plain and deformed bars and clearly indicated that settlement has a greater effect on bars near the top of a reinforced member. This is because the amount of settlement depends on the depth of the concrete below the bar. Settlement had hardly any effect on the bond strength of vertically embedded bars. The effect of settlement of fresh concrete is recognised by the South African structural design code which requires the designer to reduce bond stresses for top bars in elements deeper than 300 mm.

Reference has already been made to deformed bars; it is clear that any characteristic of the reinforcement itself which affects adhesion, friction and mechanical interlocking with the concrete will have an effect on bond. Codes of practice allow an increase in permissible bond stresses when deformed bars are used. It should be noted, however, that the ultimate bond strength, when using different types of deformed bars, depends on the characteristics of the deformations.[7.68, 7.69]

Several investigators[7.61, 7.62, 7.70] have studied the effects of surface rust on the bond strength of plain bars. The general conclusion is that firm rust increases the bond strength. Investigations[7.66, 7.69] on rusted deformed bars gave similar results. Johnston and Cox[7.71] noted that the total amount of slip attained before reaching ultimate bond strength was greater for unrusted or slightly rusted bars than for heavily rusted bars. Kemp et al[7.58] concluded that the bond characteristics of deformed reinforcing bars do not appear to be adversely affected by varying degrees of different types of surface rust or ordinary mill scale, provided that a cleaned specimen from the bar meets the minimum ASTM requirements for unit mass and height of deformation. They also stated that concrete strength appears to control the overall behaviour, particularly the amount of slip and deformation, to a much greater extent than the surface condition of the bar.

Permissible bond stresses

There are differences in the maximum bond stresses permitted in different countries. For example, the French regulations[7.72] express permissible bond stress in terms of tensile strength of the concrete. In the International Code of Practice[7.52] recommended by the European Concrete Committee, it is stated that the limiting bond stress must be considered in relation to many factors such as the transverse reinforcement and the position and orientation of the reinforcing bar in the structure.

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Chapter 8

Deformation and volume change of hardened concrete

Mark Alexander and Hans Beushausen

8.1 Introduction

8.1.1 Importance of deformations in concrete structures

Deformations of modern concrete structures due to elastic strains, creep and shrinkage, and thermal strains are important for several reasons:

- Use of forms of construction such as incrementally launched bridges, and the trend towards more slender members and taller buildings
- Use of higher working stresses without a corresponding increase in elastic modulus, leading to more flexible members for the same span and load
- Use of high-strength and high-performance concretes
- Increasing use of admixtures and cement extenders such as fly ash (FA), ground granulated blastfurnace slag (GGBS), ground granulated Corex slag (GGCS) and condensed silica fume (CSF)
- Exploitation of alternative aggregate sources (both natural and artificial)
- Drive towards greater economy of material
- Combining precast and in situ elements with associated differential movement (eg shrinkage) problems
- Differential movement problems between an existing structure and concrete applied for repair or strengthening purposes (elastic strains, creep, shrinkage and thermal strains)
- Application of prestressing to accommodate long spans in slabs and bridges, resulting in the need for design-relevant creep deformations
- Greater use of heating and insulating in buildings tending to exaggerate thermal and moisture movements

Designers of concrete structures are concerned with safety, serviceability and economy. Generally, deformations do not materially affect the load capacity of a member at collapse. However, when instability is a factor, long-term deformations can affect the failure load, for example the safety factor of a column under sustained load may reduce with time.

Regarding serviceability, deflections and cracking result from concrete deformations. Ultimate limit state design methods generally allow more slender members which may then suffer excessive long-term deflections and cracking, and these in turn may affect secondary structural elements and partitions.

In concrete repair and strengthening projects, the performance of composite members consisting of the existing structure and repair concrete is of major concern. Differential deformations under load, such as elastic deformations and creep, may need to be considered in modelling the load-bearing behaviour of repaired or strengthened structural members. Shrinkage and relaxation characteristics, as well as thermal deformations, are important as they relate to the repair concrete's tendency to crack and delaminate.

8.1.2 Components of deformation

Figure 8.1 shows the three components of deformation (or strain) that can occur in concrete under a uniaxial load, applied at time t_1, sustained for the period t_1 - t_2, and then removed. (Thermal strains are omitted, since the response of concrete to a temperature difference is relatively quick, and for practical purposes thermal strains can be regarded as immediate.) The elastic strain is shown as instantaneous, but the elastic modulus of concrete depends on the maturity/age of the concrete. With time, concrete shrinks (or swells) depending on its environment, and under load further volume changes occur due to creep. On unloading, there is an immediate strain recovery which is usually smaller than the immediate strain on loading. A further time-dependent strain recovery follows in the form of creep recovery, which is smaller than the preceding creep and, unlike creep, reaches a final value in a relatively short time. In many cases, it is the cumulative deformation of a structure or member that is important, ie the sum of the elastic, shrinkage and creep strains. For calculation purposes, the three components are usually assumed to be independent and algebraically additive, which is not strictly true.

8.1.3 Factors affecting concrete deformations

Concrete deformations may be considered by regarding concrete as a three-phase material consisting of hardened cement paste, aggregate and an interfacial transition zone

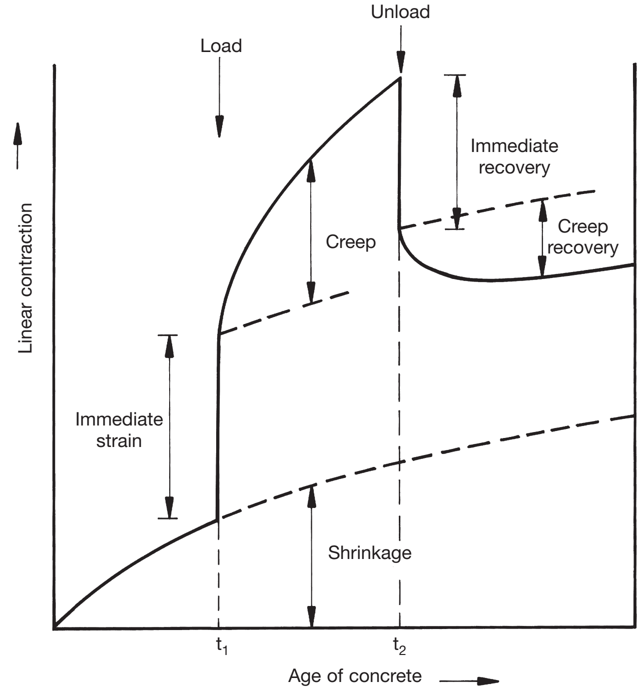


Figure 8.1: The response of concrete to compressive stress in a constant drying environment[8.1]

(ITZ) between paste and aggregate. The paste imparts strength and cohesion to the material, and has a complex structure resulting in a behaviour under load and with time that is complex. The aggregates impart bulk, rigidity and dimensional stability to the material. Notwithstanding, aggregate properties are particularly important in governing concrete elastic modulus (see section 8.2), but are somewhat less important with reference to shrinkage and creep provided the aggregate exhibits no time-dependent deformations of its own. Paste on the other hand is hygrometrically unstable and will respond to changes in load and environment in a time-dependent way. The ITZ influences elastic modulus by partially de-coupling the paste and aggregate to varying degrees, thus influencing interaction between these two components. The influence of the ITZ on shrinkage and creep is less well understood at present. The paste provides the source of creep and shrinkage in concrete, and for a fundamental understanding it is necessary to look at the response of paste to the different variables. Deformation of the concrete will then depend on properties of the paste modified by the presence of the aggregate and the ITZ according to a law of mixtures. Table 8.1 shows that concrete deformation is affected by many variables, some of them intrinsic properties of the mix, others associated with extrinsic environmental or member factors.

8.1.4 Design approaches

The structural designer requires formulations that will allow the estimation of deformations at any time, subject to the loading and climatic environment. In many cases, simplified approaches involving "single-value" estimates will suffice, and part of the designer's skill is to decide when sophistication is required. Actual material properties, applied loadings, and environment may differ markedly

Table 8.1: Factors affecting deformation of concrete

		Deformation		
		Elastic	**Creep**	**Shrinkage**
Intrinsic factors	**Paste factors**			
	Water:cement ratio	x	x	x
	Degree of hydration	x	x	x
	Age of paste	x	x	x
	Portland cement type	o	possibly	possibly
	Cement extenders	x	x	x
	Admixtures	–	possibly	possibly
	Temperature	o	x	x
	Moisture content	o	x	x
	Concrete factors			
	Aggregate properties and content	x	x	x
	Nature of interfacial transition zone	x	o	o
Extrinsic factors	Level of applied stress	**x**	**x**	**–**
	Duration of load	–	x	–
	Curing	o	x	o
	Age at loading	x	x	–
	Relative humidity and temperature	–	x	x
	Rate and time of drying	o	x	x
	Member geometry and size	o	x	x
x = Significant effect, should usually be considered o = Minor effect, can usually be neglected – = No significant effect		Load dependent	Load and environment dependent	Environment dependent

from those assumed in design, causing the best calculations to be in error. It is therefore wise to select a range of possible deformation values and assess likely structural performance against the limits of the selected range. A wide range of approaches for detailed calculations is available. These vary from graphical methods (eg the British code BS 8110[8.2]) to algebraic approaches (eg ACI 209[8.3]) and are based on results of extensive laboratory tests. The various approaches are often cumbersome to use, and their degree of sophistication is not necessarily matched by greater prediction accuracy. For this reason, a relatively simple method (for example the BS 8110 approach adopted in the SANS 10100-1:2000 code[8.4]) may suffice for most practical situations.

8.1.5 Levels of estimation

Elastic strain, shrinkage and creep components can be estimated individually taking into account the appropriate variables expressing the mix (intrinsic factors) and the environment (extrinsic factors). Estimation is inherently imperfect, and is greatly affected by the choice and quality of information of the variables. Therefore the level of estimation must be identified. The choice of level will depend on the particular structure and the quality of the initial data. A useful three-level approach is:[8.1]

(1) Lowest level – minimum initial information; crude estimate suitable only for approximate calculations, eg a single value quoted for "normal" conditions; estimate based on design cube strength, with possible allowances for humidity and member sizes.

Examples: some previous codes (eg SABS 0100:1980).[8.5]

(2) Intermediate level – more detailed initial data available, such as mix proportions and materials; development of strain with time can be estimated; requires a comprehensive set of design charts.

Examples: BS EN 1992:2004[8.6] approaches this level; CEB-FIP 1990;[8.7] ACI 209 (1997);[8.3] RILEM Model B3.[8.8]

(3) Highest level – relevant to structures where time-dependent movements are critical; considerable data required, usually assembled in a laboratory programme with use of mathematical and statistical analysis techniques.

8.2 Elastic deformation of concrete

8.2.1 Elastic modulus (E_c)

This is defined as the ratio of uniaxial stress to the resultant axial strain. Because strain is dimensionless, the dimensions of elastic modulus are the same as those of stress. Elastic modulus represents the material stiffness of the concrete to an imposed stress.

The stress-strain relationship for concrete is not constant or linear, ie it does not obey Hooke's law, although this is not an implicit requirement in the definition of elasticity. The non-linearity of the stress-strain relationship for concrete is mainly due to the non-linear stress-strain responses of the paste and ITZ, and to microcracking in the matrix. The initial portion of the curve (where the stress represents approximately 30 to 40% of ultimate strength) may however be regarded as effectively linear, and at stresses within this range the elastic modulus may be taken as the slope of this linear portion; it is referred to as *the initial tangent modulus* (Figure 8.2). Two other forms of elastic modulus may be considered, namely the *tangent modulus,* represented by the slope of the tangent to the curve at a particular stress, and the *secant modulus* represented by the slope of the line connecting the origin to the point on the curve corresponding to the stress selected.

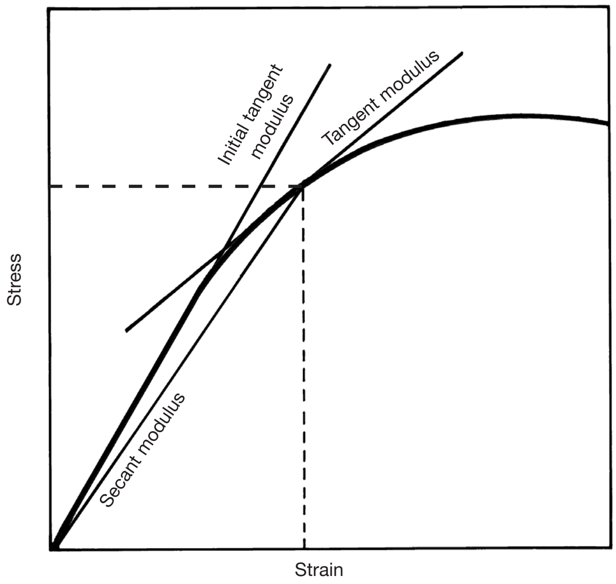


Figure 8.2: Three forms of elastic modulus

8.2.2 Factors affecting elastic modulus

The major factors influencing elastic modulus of concrete are the strength (and hence stiffness) of the paste phase, the stiffness of the embedded aggregate, and the nature of the ITZ between aggregate and paste.

The stiffer the individual phases, the higher will be the elastic modulus and the lower the long-term movement of the concrete. The greater the volume concentration of the stiffer phase (usually the aggregate), the greater will be the concrete elastic modulus. For a given aggregate, the E_c increases with the strength of the concrete or, more correctly, with the stiffness of the paste phase, determined by its porosity. Typical paste elastic moduli vary from 5 to 25 GPa, depending on water:cement ratio (w:c), degree of hydration and air content.

Research has shown that concrete strength per se, makes a relatively minor contribution to the E_c value, contrary to what most empirical expressions imply. The major contribution comes from stiffness of the aggregate and its volume concentration, a fact appreciated earlier by Kaplan.[8.9] This is shown by the empirical expression for concrete elastic modulus, suggested originally by Teychenne,[8.10] and given as equation (1) in Section 8.2.3.

The implication of equation (1) is that concretes with similar compressive strengths, but with aggregates of differing stiffness, may differ substantially in their E_c values, in fact by as much as 100 %.[8.10] At least a portion of this variation may be attributed to the characteristics of the interfacial transition zone (ITZ) between paste and aggregates: denser transition zones give higher concrete moduli.[8.11]

The influence of cement extenders is complex, and is not limited merely to the effect on concrete strength. In general, though, elastic moduli do not differ greatly in blended cement concretes proportioned for equivalent 28-day strength from those in normal portland cement concrete.[8.12] However, early-age E_c values in blended cement concretes may be lower by as much as 20 to 25%,[8.13] which can largely be related to slower hydration commonly experienced in concretes with blended cements. A South African study[8.14] has shown that the interrelationship between different aggregate types and different cement blends is important; while FA and GGBS blends generally give slightly lower concrete elastic moduli, the use of CSF often gives increased elastic moduli particularly at early ages. This is ascribed to the ability of CSF to densify the ITZ from early ages.

8.2.3 Estimation of elastic modulus

Practical approaches

The relationship between elastic modulus and compressive strength is the most practical approach, since engineers will almost always possess a knowledge of concrete strength at the design stage.

Many attempts have been made to correlate the modulus of elasticity and the strength of concrete. The factors that affect the modulus of elasticity of concrete do not always have a correspondingly similar effect on concrete strength. For example, the use of an aggregate with a higher modulus of elasticity does not necessarily produce a concrete of greater strength although it will increase the modulus of the concrete. The work of Kaplan[8.9] showed conclusively that different aggregates and mix proportions affect the relationship between dynamic modulus and strength.

With regard to South African data, reference can be made to work by Davis,[8.15] who used a power equation to describe the relation between elastic modulus and strength.

Most structural design codes contain guidance or expressions for estimating elastic modulus based on concrete strength, and some are briefly reviewed here:

SANS 10100-1:2000[8.4]

The 2000 version of SANS 10100 (Part 1, Annex. C) has adopted the old British code approach, reviewed below. However, in the main section of Part 1, elastic moduli can still be estimated from a table, based purely on concrete compressive strength. In this case, for low-density-aggregate concrete having a density between 1 400 kg/m³ and 2 300 kg/m³, the code recommends multiplying the estimated E values by $(D_c/2\ 300)^2$ where D_c is the density of the concrete in kg/m³.

BS 8110-2:1985[8.2]

The 28-day elastic modulus of concrete, $E_{c,28}$, is estimated from the 28-day characteristic cube strength, $f_{cu,28}$, from the expression

$$E_{c,28} = C_o + 0{,}2\ f_{cu,28} \qquad (1)$$

A C_o value of 20 GPa is suggested for concrete of normal density, and 10 GPa for low-density concrete. It is also advised to consider a range of values of concrete E based on C_o varying from 14 GPa to 26 GPa. This approach recognises the important role aggregates play in governing concrete elastic modulus.

For ages other than 28 days, the code gives the formula

$$E_t/E_{28} = 0{,}4 + 0{,}6\ f_t/f_{28} \qquad (2)$$

where subscript t refers to the required age in days (t > 3d) and f to characteristic cube strength.

ACI 318M-05[8.16]

The ACI recommends an empirical relationship.

$$E_{ct} = 0{,}043[\rho^3(f'_c)_t]^{\frac{1}{2}}\ \text{(MPa)} \qquad (3)$$

where E_{ct} is the secant modulus of elasticity at age t (days), ρ is the density of the concrete in kg/m³, and $(f'_c)_t$ is the cylinder strength (MPa) of the concrete at age t, determined from tests on 152 mm x 305 mm cylinders.

In simplified form the ACI formula is

$$E_{ct} = 4\ 700\ \sqrt{f'_c}\ \text{(MPa)} \qquad (4)$$

assuming the density of concrete to be 2 300 kg/m³.

The ACI formula is also empirical and based on concrete compressive (cylinder) strength. The expression includes a term for concrete density, ρ, raised to the power of $^3/_2$ which gives this term greater weighting than that for strength. In essence, the approach makes an implicit, albeit very partial, allowance for aggregate stiffness via the density term, since aggregate E is related to aggregate density, and thus to concrete density.

CEB-FIP[8.7]

The CEB-FIP MC 90 relationship is

$E_c = 10\ (f_{ck} + 8)^{\frac{1}{3}}$ (GPa).. (5)

where E_c is the modulus of elasticity and f_{ck} is the characteristic cylinder strength (MPa) at 28 days.

The formula effectively adjusts the characteristic strength to a mean strength. The expression is a power formula in which concrete E_c depends only on concrete strength; no allowance is made for aggregate effects.

All the above approaches fail to account adequately for the important role that aggregates play in governing concrete elastic modulus.

BS EN 1992:2004[8.6]

The EN expression which is essentially identical to the CEB-FIP MC 90 relationship (eg 5) is:

$E_{cm} = 22\ [(f_{ck} + 8)/10]^{0.3}$.. (6)

where E_{cm} is the modulus of elasticity and f_{ck} is the characteristic cylinder strength (MPa) at 28 days. The code makes allowance for different aggregates. For limestone and sandstone aggregates the value should be reduced by 10% and 30% respectively. For basalt aggregates the value should be increased by 20%.

For ages other than 28 days, the code gives the relationship

$E_t/E_{28} = (f_t/f_{28})^{0.3}$...(7)

Figure 8.3 illustrates the above time-dependent development of the modulus of elasticity in comparison to the development of compressive strength (based on typical strength development for PC-42,5 concretes[8.17]). It should be noted that the modulus of elasticity develops at a significantly faster rate, compared to compressive strength.

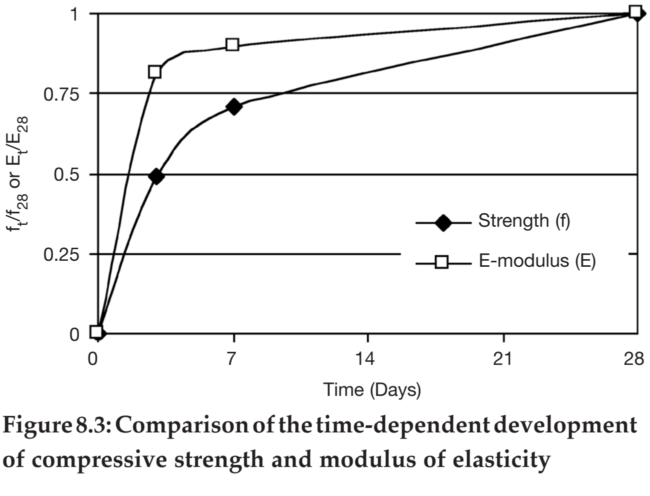


Figure 8.3: Comparison of the time-dependent development of compressive strength and modulus of elasticity

South African expression based on BS 8110

This approach deals with elastic modulus of portland cement concrete as a function of aggregate type from sources throughout the country.[8.18 - 8.22] It is based on the work of Teychenne,[8.10] and uses the form of equation (1).

$E_{c,28} = K_0 + \alpha f_{cu,28}$..(8)

where $E_{c,28}$ is concrete elastic modulus at 28 days, $f_{cu,28}$ is characteristic concrete (cube) strength at 28 days, α is a coefficient, and K_0 is an aggregate stiffness factor related to the elastic modulus of the aggregate and to its volume concentration. SANS 10100-1:2000[8.4] gives α as 0,2, whereas local research has established various values for α based on specific aggregate types, as discussed below.

Experimental results from concretes made with 23 different aggregate types showed that there was no unique E-f_{cu} relationship for concrete, but each aggregate type had its own specific relationship. Equation (8) was applied to the results for two ages: early age between three days and 28 days, and a later age of six months or greater. For applying the expression, concrete (cube) strength at the particular age in question should equal or exceed 20 MPa.

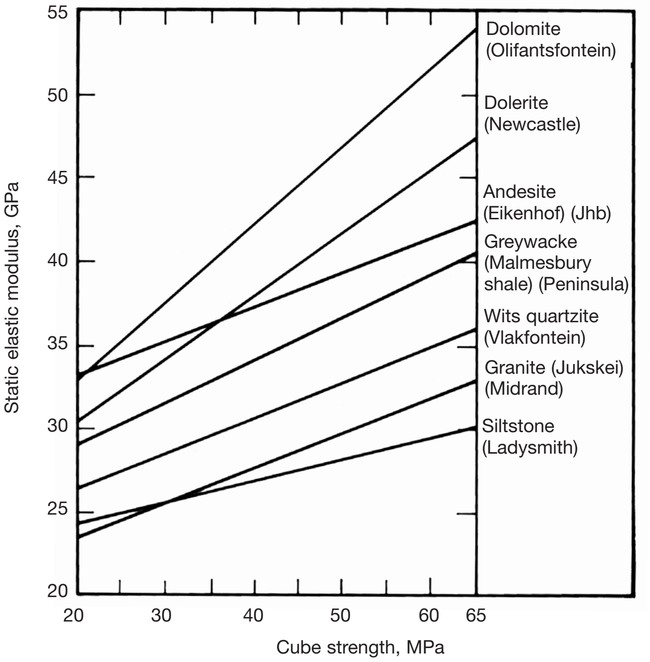


Figure 8.4: Relationship between static elastic modulus and cube strength for ages from three days to 28 days[8.22]

An indication of how concretes made with local aggregates vary widely in respect of stiffness is given in Figure 8.4, where seven aggregate types are represented. These examples show the wide variation of elastic behaviour patterns, the powerful influence of aggregate type on the value of E, and the effect of different aggregates on the slope of the lines.

At opposite extremes, the dolomite concrete has an E value highly sensitive to concrete strength, while the siltstone concrete does not display such a trend. One practical consequence of this might be, for example, that dolomite concrete would be more suitable for high-strength concrete columns.

Figure 8.5 shows the increase in elastic modulus during the period from 28 days to six months (assuming continued curing) for a particular concrete made with dolomite. Concretes made with other aggregates also exhibited similar large gains. This increase in E with maturity cannot be accounted for solely by strength gains in the matrix. The explanation lies in some form of on-going interfacial bond development or densification of the ITZ between aggregate and paste. There is evidence of marked variations in the interactions of different aggregates with the surrounding matrix.

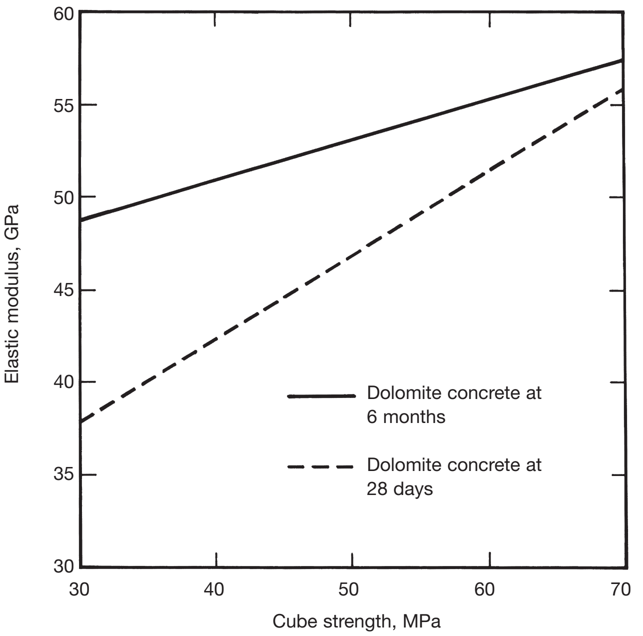


Figure 8.5: Increase in elastic modulus of dolomite concrete from 28 days to six months[8.22]

With reference to equation (8), ranges of possible K_o and α values for design are given in Table 8.2, in terms of generic aggregate types. For information relating to specific rock types from particular quarries, reference 8.18 should be consulted. Single values rather than a range of values have been given in Table 8.2 where an aggregate from only one source was tested in concrete.

8.2.4 Structural implications of elastic modulus

The importance of E_c depends very much on the sensitivity of the structure to deformation. The majority of structures are not very sensitive to changes in E_c, and sophistication in E_c prediction is unjustified. However, where deflections will be critical, or where secondary cracking and distress is unacceptable, accurate E_c predictions are important, and often stiff concretes will be required. In other cases, low E_c may be desirable, for instance where cracking due to restrained movement is to be avoided.

8.2.5 Elastic modulus in tension

Certain authors[8.23, 8.24, 8.25] suggest that E_c in tension and E_c in compression are equal, while others[8.26, 8.27] have concluded that the modulus in tension is either greater or less than

Table 8.2: Design values for estimating elastic modulus of concrete for ages (a) from three to 28 days (b) at six months or older[8.19-8.22]

Aggregate type	Range of design values			
	(a) three to 28 days		(b) six months or older	
	K_o GPa	α GPa/MPa	K_o GPa	α GPa/MPa
Western Cape				
Granite	21	0,25	34	0,10
Greywacke (Malmesbury shale)	24	0,25	31	0,20
TM quartzite (Mossel Bay area)	23	0,25	34	0,15
KwaZulu-Natal				
Dolerite	15 - 22	0,40	29 - 39	0,15
TM quartzite	17 - 21	0,25	30	0,15
Tillite	20	0,35	29	0,20
Siltstone (KwaZulu-Natal Midlands)	21	0,15	27	0,10
Gauteng and surrounding areas				
Andesite	25 - 26	0,30	35 - 36	0,20
Dolomite	24 - 25	0,45	43 - 49	0,20
Felsite	18 - 21	0,35	29 - 31	0,20
Granite	17 - 18	0,25	25 - 31	0,10
Quartzite: Ferro	17	0,40	31	0,20
Daspoort	14	0,30	28	0,15
Reef quartzite	18 - 20	0,25	33 - 35	0,10

that in compression. As in compression, the modulus of elasticity of concrete in tension is generally a secant value because of the curvilinear stress-strain relationship. The modulus therefore depends on the stress level at which it is measured and it would appear that the differences between the moduli in tension and compression, reported by some investigators, are mainly due to this factor.

Alexander[8.28] found that elastic moduli measured in flexure on beams of 100-mm square section are generally higher than those measured on 100-mm square prisms in compression, and has suggested that the difference may be qualitatively explained by taking into account the volumes of stressed material.

For all practical purposes the Young's moduli of concrete in tension and in compression are similar provided that both are measured at stress levels that are the same percentage of the respective ultimate strengths, and with the further proviso that the stress levels are below that at which rapid crack propagation occurs in the test specimen.

8.2.6 Elastic modulus of high-strength concrete (HSC)

The modulus of elasticity of HSC is highly dependent on the type of coarse aggregate rather than the compressive strength.[8.29] In addition, elastic modulus develops very rapidly within the first 24 hours, and thereafter elastic modulus increases are moderate.[8.30] Using equation (8) given previously, typical α values for HSC are between 0,05 and 0,1, while K_0 will be highly dependent on the type of aggregate. For instance, Addis's[8.31] results for andesite aggregate concretes with compressive strengths between 55 and 110 MPa yielded $\alpha = 0{,}11$ and $K_0 = 42$ GPa.

8.2.7 Tests to determine elastic modulus

Most reported values of E_c derive from static tests, where loads are increased over periods of minutes rather than microseconds as with dynamic tests.

Static tests

The method outlined in BS 1881-121:1983[8.32] involves loading a standard cylinder (150 mm diameter x 300 mm) and measuring load and deformation. Several cycles of stressing to about one third of ultimate are carried out to reduce the effects of creep and small shrinkage cracks on the results. The rate of loading is controlled (time of loading two to five minutes), and a secant modulus is normally measured corresponding to a stress of one third of ultimate.

In the ASTM method C 469-02,[8.33] a similar procedure is adopted except that the strain at 40% of the ultimate stress is recorded. The "secant" modulus of elasticity at this stress is taken to be the static modulus. When the ultimate compressive strength of the concrete is not known, the secant modulus is determined at a stress corresponding to a specified strain, the stress generally being between 30 and 50% of the ultimate stress.

The Standard RILEM method is CPC 8 (1975): *Modulus of elasticity of concrete in compression*.[8.34]

Dynamic tests

In these tests the applied loads are very small and the rates of application and release of the loads are rapid; the effects of creep are therefore negligible.

The first method consists of determining the fundamental transverse or longitudinal resonant frequency of vibration of concrete prisms or cylinders. Standard procedures for measuring the fundamental resonant frequencies are given in BS 1881-209:1990[8.35] and ASTM C 215-02[8.36] and consist of exciting the test specimen by means of an exciter unit driven by a variable-frequency oscillator. The oscillations are received by a pick-up unit, amplified, and applied to an amplitude indicator. The frequency of excitation is varied until resonance is obtained in the fundamental mode of vibration.

The dynamic Young's modulus of elasticity may be calculated from the formula

$$E = Kwn^2 \quad \text{.......} \quad (9)$$

where

E = dynamic modulus of elasticity

w = mass of the specimen

n = fundamental resonant frequency

K = a factor depending on the size and shape of specimen, the mode of vibration and, for transverse resonance, Poisson's ratio of the concrete; K is determined from classical wave theory.

A second method of determining the dynamic modulus is to measure the velocity of a pulse of mechanical vibrations through the concrete: the ultrasonic pulse velocity (UPV) method. The dynamic modulus may be calculated from the formula

$$E = \rho V^2 \frac{(1+\mu)(1-2\mu)}{1-\mu} \quad \text{.......} \quad (10)$$

where

E = dynamic modulus of elasticity
V = longitudinal pulse velocity
ρ = density of the concrete
μ = Poisson's ratio

It is not certain that pulse velocity is independent of variations in the shape and size of the concrete test specimen.[8.37] An advantage of this method is that it may be used on concrete in structures. Poisson's ratio and the density of the concrete must however be known accurately and, in general, this method is not recommended for the determination of the dynamic modulus of elasticity.

8.2.8 Comparison of static and dynamic moduli

The loads applied in the dynamic method are extremely small and the effects of creep are negligible. For this reason the dynamic modulus is considered to resemble the initial tangent modulus determined in the static test. Powers[8.38] obtained fairly good agreement between the static initial tangent modulus and the dynamic resonance modulus. Generally, however, the static modulus of concretes is less than the dynamic modulus, the ratio between them increasing with concrete strength and age.[8.39, 8.40]

Various relationships between static and dynamic moduli have been suggested, but the one proposed by SANS 10100-1:2000[8.4] is probably the most suitable.

The relationship is

$E_c = 1{,}25\ E_d - 19$ (GPa)..(11)

where E_c and E_d are the static and dynamic moduli respectively.

Such an estimated value for the static modulus will generally be correct within ± 5 GPa. Jones[8.41] has pointed out that the relationship between static and dynamic moduli may vary when different types of aggregate are used.

8.2.9 Micro-rheological models for estimating concrete elastic modulus

Concrete may be viewed as a composite material consisting of different phases which are mechanically and chemically distinct and are separated by interfaces. Regarding concrete's mechanical properties, the important phases will be a cement-paste matrix, aggregate particles which are embedded in the matrix, and an ITZ phase. This structure can be modelled mathematically by assuming that the paste phase and the aggregate phase are each homogeneous and isotropic, which is reasonable at the macroscopic level. The modelling of the ITZ is, however, more complex. Two-phase models have been proposed based on simplifying assumptions, which provide reasonably good approximations to the mechanical behaviour of concrete. Further details are available in references 8.42 to 8.47, or in the 7th edition of *Fulton's concrete technology.* For application to two local aggregate concretes, see reference 8.48.

8.3 Poisson's ratio

This is defined as the ratio of lateral strain to axial strain, where the strains are caused by uniaxial stress only.

8.3.1 Estimating Poisson's ratio

Many investigators[8.22, 8.42, 8.49, 8.50] have measured static Poisson's ratio from uniaxial compression tests and have found that values of static Poisson's ratio for mortars and concretes lie between 0,1 and 0,2. The type of aggregate affects Poisson's ratio and an increase in aggregate:cement ratio produces a significant decrease in the static value[8.51] (Figure 8.6).

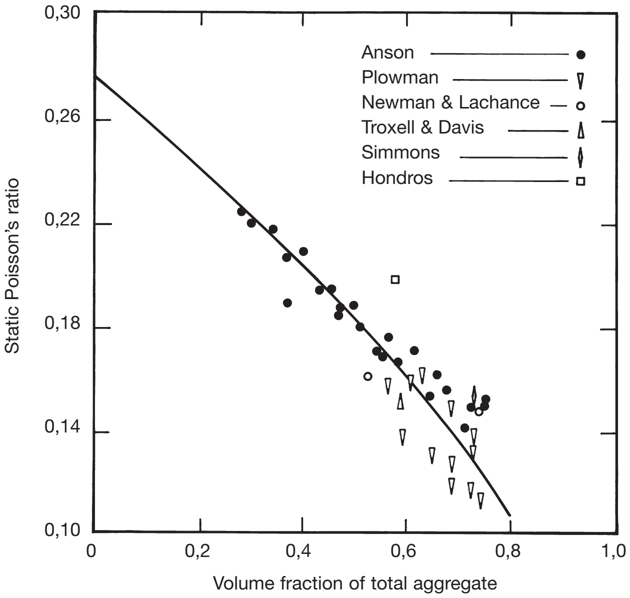


Figure 8.6: Relationship between static Poisson's ratio and total aggregate volume fraction[8.51]

Results for dynamic Poisson's ratio for mortars and concretes lie between 0,18 and 0,30, and show that it decreases with an increase in aggregate content and age.[8.49,8.51] Increase in water:cement ratio increases the Poisson's ratio. The type of aggregate used also has a considerable effect on dynamic Poisson's ratio.[8.9, 8.15] Kaplan[8.9] for example, has shown that concrete of the same mix proportions but made with a quartzite gravel coarse aggregate had a dynamic Poisson's ratio of 0,25 as compared with 0,31 when made with a limestone gravel.

In the CEB-FIP Model Code[8.7] and in SANS 10100-1:2000[8.4] the value of Poisson's ratio is given as between 0,1 and 0,2, or as 0,2 respectively.

8.4 Creep of concrete

Creep is defined as the time-dependent increase in strain of a solid body under constant or controlled stress (Figure 8.7). Creep may also be manifested as a relaxation of stress under a constant strain (Figure 8.8). Creep in concrete occurs at all stress levels, and the creep strain can be several times larger than the initial strain on loading. Consequently, creep imparts to concrete a degree of ductility, which is desirable from the point of view of structural behaviour. Other beneficial

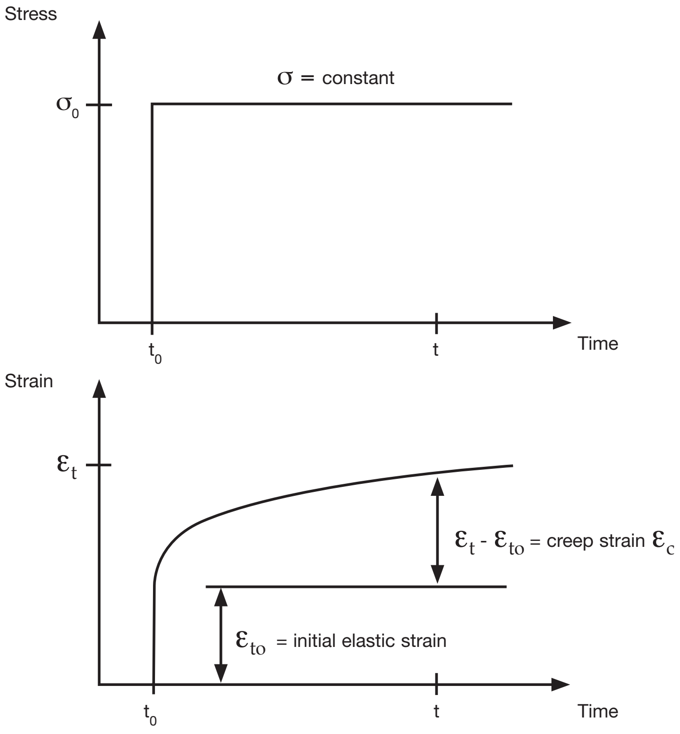


Figure 8.7: Characteristics of creep: time-dependent increase in strain under constant stress

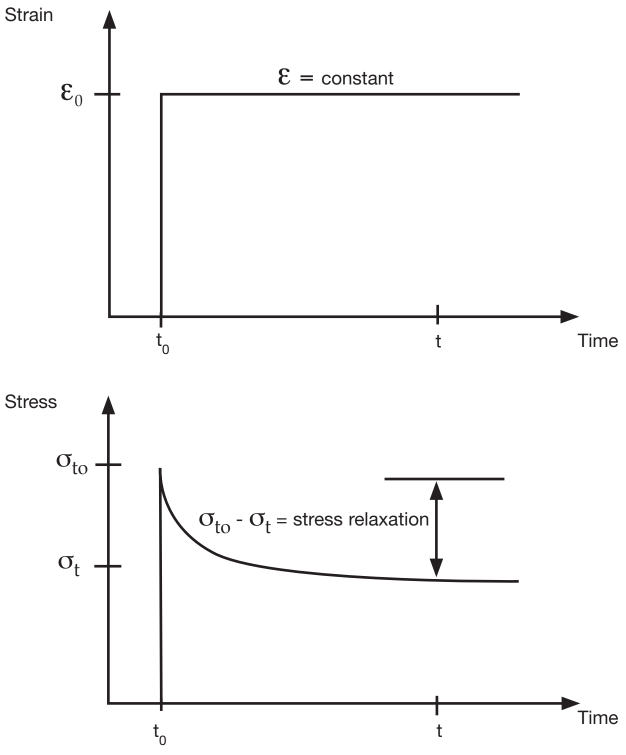


Figure 8.8: Characteristics of relaxation: time-dependent decrease in stress under constant imposed strain

effects of creep include the relief of concrete stresses due to differential structural movement and restrained shrinkage. However, creep also has detrimental effects on structures, such as increased deflections which can result in cracking, loss of prestress and creep buckling of long columns. These detrimental effects can usually be minimised provided the designer is acquainted with the properties of the material and takes the necessary steps to allow for them in design.

The phenomenon of creep has been recognised ever since concrete was tested for its properties. However, this aspect of concrete's behaviour has become the focus of greater attention in recent times due to the tendency of structures to become ever more highly stressed and slender. Strength is no longer necessarily the overriding factor in design. The modern designer must frequently check structural deflections and stability in conjunction with strength calculations, and revise member sizes to conform with these other structural criteria. While a reduced elastic-modulus approach has sufficed in the past, and may still suffice for the simpler problems, more explicit procedures are required to handle the complex problems of creep deformations in critical members or under varying stress.

8.4.1 Creep expressions

A linear relationship exists between applied stress and creep of concrete (for relatively low stress levels of up to approximately 40% of ultimate strength). This allows a parameter called the specific creep C_c to be defined, where C_c is the creep strain ε_c per unit stress

$$C_c = \frac{\varepsilon_c}{\sigma} \quad \text{.......} (12)$$

This parameter provides a basis for comparing the creep behaviour of concretes loaded to different stress levels. Alternatively, concretes may be compared on the basis of equal stress:strength ratio, which takes account of the factors which affect the strength of concrete at the time of loading.

Another parameter frequently used for comparative and computational purposes is the creep factor or creep coefficient ψ, defined as creep strain ε_c divided by initial elastic strain ε_o:

$$\psi = \frac{\varepsilon_c}{\varepsilon_o} \quad \text{.......} (13)$$

Note: initial elastic strain defines ε_{to} in Figure 8.7.

Since factors such as volume concentration and elastic modulus of the aggregate affect both the elastic and the creep deformations of concrete, it is possible that two concretes with different aggregate properties may nevertheless exhibit similar creep factors. From $\varepsilon_o = \sigma/E_t$ where E_t is the elastic modulus of the concrete on loading, it follows that:

$\psi = C_cE_t$ ....(14)

A further approach is to define a creep compliance function $\phi(t,\tau)$, as the sum of the instantaneous and creep strains at time t produced by a sustained *unit stress* applied at τ ,

$$\phi(t,\tau) = \frac{1}{E_c(\tau)} + C_c(t,\tau) = \frac{1}{E_c(\tau)}\,[1 + \psi(t,\tau)] \quad (15)$$

Therefore, for stress σ, the total (instantaneous + creep) load-induced strain at time t for loading at time τ is:

$$\varepsilon(t,\tau) = \sigma\phi(t,\tau) \quad (16)$$

The advantage of using a creep compliance function is that it represents total load-induced strain at any time.

8.4.2 Characteristics of creep

Creep occurs in concrete at all stress levels, but the mechanisms are different at higher stress levels of above 40 to 50% of short-term strength. Below this level, significant load-induced microcracking has not occurred. The heterogenous nature of concrete leads to the presence of substantial stress concentrations in the matrix, and microcracks between aggregate and matrix and within the matrix initiate and grow with increasing external load, leading to an additional component of creep at high stress levels. This factor is allowed for in design where working stresses are generally limited to values not greater than 40% of the short-term strength of concrete.

A typical creep curve for concrete under uniform sustained low-level compressive stress is shown in Figure 8.7. Under sustained loading the total measured strain increases at a decreasing rate and at extended ages may approach a value several times larger than the elastic strain. If this time-dependent deformation occurs under drying conditions, a portion of the deformation will consist of shrinkage. The remaining portion is termed the creep of the concrete.

For computational purposes the creep is often assumed to be independent of and algebraically additive to the shrinkage so as to produce the total time-dependent deformation. This is not strictly true, as has been shown by Ruetz[8.52] and others.

The interdependence of creep and shrinkage is shown by the fact that an additional component of creep is introduced if the concrete is allowed to dry while under sustained load. In this case, creep will consist of:

- *basic* creep which is the creep experienced by a concrete which has previously been brought into hygral equilibrium with its environment; and
- *drying* creep, which is the additional creep experienced when simultaneous drying of the concrete is involved.

For practical purposes, basic creep may be considered to occur in large structural members (d_{min} > 600 mm) where drying is a very slow process, or in sealed or submerged members. In most cases, however, consideration of creep usually refers only to the total creep, ie the sum of basic and drying creep.

8.4.3 Structural effects of creep

It is beyond the scope of this chapter to include a detailed discussion of the effects of concrete creep on structural performance. Several references[8.3, 8.53-8.57] are available in which methods of taking creep into account in design, and case histories of structural damage or defects caused by creep, are presented. The beneficial effects of creep on structures are the relief of tensile stresses induced by shrinkage or thermal strains, or reduction of stress concentrations caused by concentrated loads. However, the effects of creep are generally considered to be detrimental inasmuch as serviceability is affected, and substantial loss of prestress may occur. In general, creep has little effect on the ultimate strength of concrete members since deformations associated with the ultimate state are usually very much larger than those induced by creep. At service loads, creep will cause a redistribution of stress in reinforced concrete members, and lead to an increase in deflections. The following is a brief review of creep effects.

Columns undergo a redistribution of stress due to the combined effects of creep and shrinkage. The steel stress increases with time and may become very large, even approaching yield. Conversely, the concrete stress reduces with time. In the event that one element reaches its ultimate strength, the other will attract any additional loads under continuing deformation, and thus the ultimate strength is sensibly unaffected by the magnitude or distribution of internal stresses resulting from creep at service loads.

The possibility of buckling of the reinforcing steel in columns due to the effects of creep and shrinkage may also arise. This is well covered in reference 8.54. An adequate number of ties is essential, and attention should be paid to ensuring an adequate cover to the steel. Creep deflection may also give rise to instability in arch or shell structures.

The long-term deflections of bending elements due to the combined effects of additional cracking, creep and shrinkage may lead to structural inadequacy. Most important are elements that are truly simply supported, and long-span elements, in which excessive sagging must be controlled. Flat slabs used for roofs should be designed so as to avoid ponding of water. Australian and local experience with flat slabs suggests that these elements may suffer long-term deflections of the order of five to eight times the initial deflections depending on the degree of initial cracking and exposure conditions.[8.58] Specialist literature[8.59] should be consulted for selecting limiting span:depth ratios for different types of construction.

The most noticeable effect of creep and shrinkage is probably the damage caused to secondary building elements which are often non-loadbearing.[8.60] Claddings attached to creeping columns may suffer spalling or bowing, while partition walls of brick or other materials may crack as the supporting concrete beam or slab continues to deflect. This latter problem is accentuated if brick walls are built before supporting props to the concrete beam or slab are removed. Attention to detail by the designer may often help to avoid such problems. In general, the stiffness of adjoining elements should be matched as far as possible, and where creep movements appear to be a problem, suitable precautions should be taken at the design stage.

In summary, the effects of creep seldom lead to structural collapse or damage to actual members themselves. Secondary effects such as excessive deflections, unsightly surfaces, cracking and spalling, and damage to non-structural partitions and other building elements, are the main results of creep. Expensive repair and maintenance may follow such general unserviceability.

Relaxation in concrete members subjected to restrained hygral volume changes

For certain types of concrete structures or elements, the effects of restrained hygral and thermal deformations need to be considered. A typical example of such members are bonded overlays which are frequently used in concrete repair works and in hybrid construction (where precast elements are combined with in-situ concrete). The restraint of hygral and thermal deformations is the main factor influencing the serviceability and durability of bonded overlays, the associated failure mechanisms being cracking and debonding. For stress development in overlays subjected to restrained deformations, tensile stress relaxation has been recognised as one of the most important factors.[8.61-8.67] In absence of detailed information, the estimation of such relaxation is often based on creep strain characteristics.

An important aspect for the performance of bonded overlays is the time-dependence of visco-elastic mechanisms. In experiments discussed in reference 8.65, it took only 2 to 3 hours to reach the ultimate values of tensile relaxation in specimens subjected to constant strain, irrespective of the stress-strength ratio which ranged from 0% to 60%. Specimens were tested at different ages between 1 and 21 days, with the observation that ultimate relaxation decreases with the age of the member. Similarly, experiments discussed in reference 8.64 showed that the initial stress-strength ratio has no significant influence on tensile relaxation. In the study referred to, relaxation after 100 hours was found to be 35 to 50% of the initial stress. Kordina et al[8.68] tested different concrete mixes for tensile relaxation and found that ultimate relaxation values were reached after approximately 2 days. In their tests, relaxation was found to decrease the tensile stresses by about 50 to 60%. Beushausen and Alexander[8.69] found relaxation in bonded overlays subjected to restrained shrinkage to occur rapidly and to release approximately 40 to 50% of tensile stress. Based on these studies, it can be assumed that relaxation mechanisms reduce tensile stresses related to restrained hygral deformations in concrete members to about 50% of their initial value, which needs to be considered when the potential serviceability of such members is analysed. For thermal stresses due to ongoing temperature cycles during the lifetime of a structure, however, different relaxation values may be found as concrete age possibly plays a role. The abovementioned studies relate to relaxation assessments on relatively young concretes and are indicative for the case of restrained hygral deformations, which are most important during the first weeks after casting.

8.4.4 Factors affecting creep of concrete

Table 8.1 shows that creep of concrete is affected by many variables, some of them intrinsic properties of the mix, others associated with extrinsic environmental or specimen factors. These factors are discussed below, together with relevant South African data where available. Creep of concrete is dealt with directly wherever possible, but behaviour of the paste component is referred to where this is appropriate.

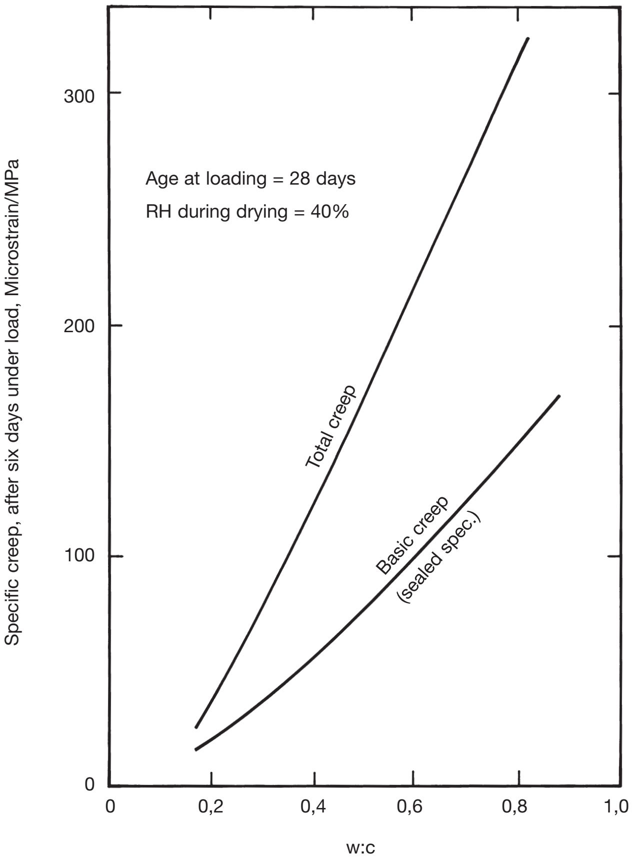


Figure 8.9: Effect of w:c on creep of cement paste[8.52]

Water:cement ratio (w:c)

This parameter controls the strength, stiffness and permeability of cement paste, and therefore indirectly affects the creep of paste. Since stronger pastes are also stiffer it is found that a decrease in w:c causes a decrease in creep. This is shown in Figure 8.9, which is adapted from Ruetz.[8.52] It can also be seen that the creep of the specimens allowed to dry while under load, is about twice the basic creep, under the test conditions shown in the figure.

Moisture content

Since creep is associated with the presence of mobile water in the paste, the greater the moisture content, the greater the creep. Two effects should be noted here. Firstly, concretes that are allowed to dry before loading will contain less moisture and will consequently creep less. This is illustrated in Figure 8.10, in which the very low creep of the paste dried at zero relative humidity before loading may be noted. Secondly, pastes and concretes that are allowed to dry for the first time under compressive load will experience greater creep than had they remained saturated, due to the additional component of drying creep.

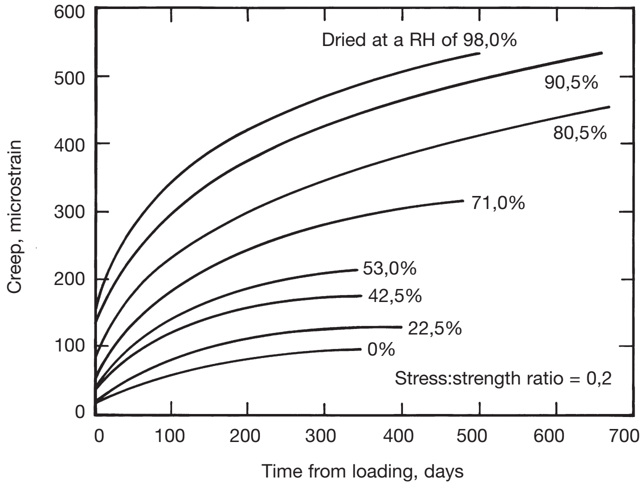


Figure 8.10: Effect of previous drying on basic creep of cement paste[8.70]

Cement type

Creep of cement paste is affected by cement composition, with higher creep resulting from higher C_3A content or lower C_3S content.[8.47] However, at least a portion of the difference in creep between different cements can be accounted for by the variable rates of strength gain which in turn are affected by composition and fineness of grinding. Consequently, ordinary portland cement will tend to creep more than a rapid-hardening cement, but less than a low-heat cement. Fineness of cement appears to influence creep only through its effect on strength development at early ages.

Cement extenders

Several investigators have noted that the use of binders containing GGBS may increase creep (and shrinkage) of concrete,[8.54, 8.71-8.73] but this effect is by no means universally noted.[8.74] There are many factors which influence the relative performance of GGBS concretes, including GGBS and cement characteristics, GGBS proportion, ages of loading and / or exposure, conditions of drying, etc. Two aspects that are particularly important are the strength of the concrete at the age of loading, and whether samples are sealed or left exposed to a drying environment during the test.

South African work[8.75] concluded that the presence of GGBS in portland-cement concretes had the effect of causing a possible increase in creep (about 20%) at early ages in exposed specimens, but this effect was usually reversed at later ages. Creep strains were markedly reduced in sealed specimens at all ages, often by as much as 40%. The use of GGBS was found to produce concretes with more consistent properties in the case of variable clinkers or clinkers from different sources (assuming the GGBS was from a single source).

Jaufeerally[8.76] measured creep characteristics of concrete made with Corex slag (GGCS) from the Western Cape, and concluded that the use of GGCS results in somewhat lower creep factors, compared to PC concrete. When considering basic creep (ie creep under constant moisture conditions), GGBS concrete was found to have significantly lower creep potential than PC concrete.

Grieve's work[8.77] on concrete incorporating Matla fly ash (FA) found that specific creep was reduced in FA mixes relative to plain mixes with similar 28-day strengths, particularly if the specimens were sealed. This confirms the results of work done elsewhere on FA mixes.[8.78-8.81] The reduction of creep appears to be greater than simply the effect of reduced water contents in FA mixes. If FA is used to simply replace portland cement in a mix, however, then higher creep strains may be experienced.[8.82]

Limited results for the influence of condensed silica fume (CSF) on creep of concrete seem to indicate that CSF generally has the effect of reducing concrete creep when compared with otherwise similar plain mixes.[8.83-8.86]

Admixtures

The effect of admixtures on creep appears to be highly variable and depends, inter alia, on the specific cement and admixture chemistry, age at loading, relative humidity of the environment and time under load.[8.87] In a South African investigation[8.88] of concretes made with a plasticiser (water reducer) and three superplasticisers, it was found that creep increased despite reductions in paste content achieved by the use of the admixtures. However a further study[8.89] as well as UK results[8.90] have shown that results for different admixtures can vary widely, and therefore any effect of admixtures on the creep of concrete will depend on the specific admixture and test conditions. Where creep may detrimentally affect structural performance, admixtures proposed for use should be evaluated by laboratory tests. In these cases, care should be taken in the interpretation of results so as to separate the intrinsic effect of the admixture from effects introduced by concomitant changes in mix proportions.

Aggregate properties and content

Normal-density aggregates of hard gravel or crushed rock usually do not exhibit creep at the stress levels to which they are subjected in normal concrete. Consequently aggregates reduce the creep of concrete by "diluting" the paste and by restraining its movement. Concrete creep is therefore affected by aggregate volume concentration and aggregate stiffness. Aggregate properties such as grading, maximum particle size and shape exert an indirect influence on concrete creep through their effect on volume concentration. Neville[8.91] and Hobbs[8.46] have suggested relationships for the creep of concrete as a function of aggregate volume concentration and creep of the paste. Hobbs' relationship is:

$$\varepsilon_c = \frac{1 - V_a}{1 + V_a} \cdot \varepsilon_p \tag{17}$$

where ε_c and ε_p are the creep of the concrete and the paste respectively, and V_a is the volume fraction of the aggregate. According to the above relationship, creep for the normal range of concretes and mortars should range between about 15% and 30% of the creep of their corresponding pastes, depending on the quantity of the aggregate. The same relationship predicts that for paste contents in the range of 30% to 40% by volume, concrete creep should increase by about 5% for each 1% increase in paste content. This is also quoted by Evans and Kong.[8.92]

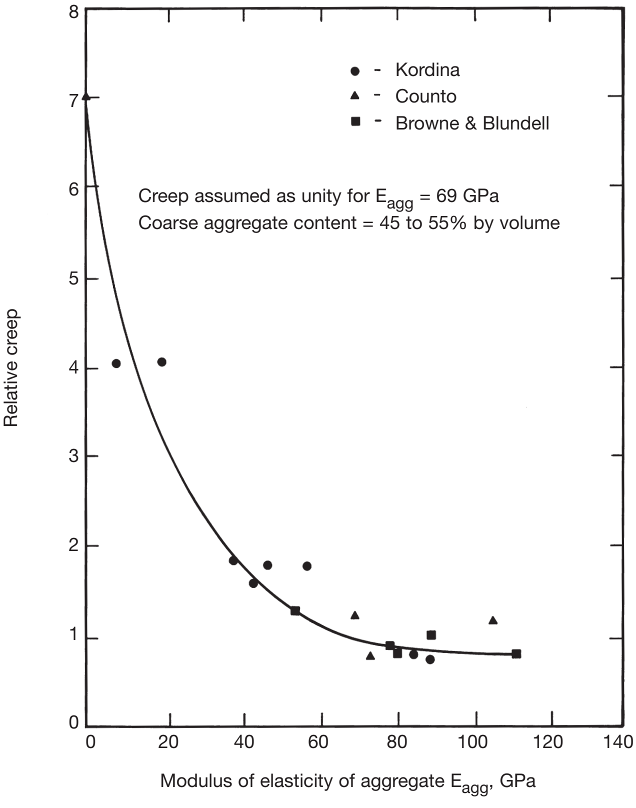


Figure 8.11: The effect of aggregate stiffness on creep of concrete[8.93]

The higher the elastic modulus of the aggregate, the greater will be the restraint offered by the aggregate to the creep of the paste. This is shown in Figure 8.11, where the creep of concretes containing low-modulus aggregates may be up to four times that of concretes with stiffer aggregates. Creep appears to be unaffected once the aggregate modulus exceeds about 70 GPa. This has been confirmed in recent South African work, where there was little or no influence of aggregate stiffness on creep for E_{agg} greater than about 70 GPa.[8.94]

Work in the USA by Troxell, Raphael and Davis[8.95] showed that the creep of concrete made with sandstone aggregate was more than twice that of concrete made with limestone aggregate, the various aggregates in increasing order of concrete creep being limestone, quartz, granite, gravel, basalt and sandstone. Rüsch et al[8.96] also found that sandstone aggregate concrete produced higher creep than concrete using other aggregate types, but the order of increasing creep of concrete using their aggregates differed from the previous list, presumably due to mineralogical and petrological variations.

Other South African work shows a substantial influence of aggregates on concrete creep.[8.18, 8.19] In evaluating the creep of concretes with 23 different aggregates, in which the influence of water demand of the aggregate was included, it was found that creep coefficients varied by up to ± 40 to 50% from the mean. In general, dolomites and unweathered granites showed low relative creeps, with basic igneous rocks falling nearer the upper range. Quartzites were distributed throughout the range. It is important to stress that creep is a variable phenomenon, and in critical cases it is best to undertake laboratory creep testing with the chosen aggregates.

Referring to the creep of low-density-aggregate concrete, the lower modulus of the aggregate is reflected in higher concrete creep, as would be expected. However, for the same strength, low-density-aggregate concrete usually requires paste with lower w:c than normal-density concrete, and the effect is that the creep properties of the two concrete types are essentially the same when compared on the basis of an equal ratio of stress to strength.[8.97]

Specimen geometry and size

For drying under load, the shape and size of the member affect the rate and magnitude of creep. A suitable parameter reflecting the specimen geometry and size is the volume: surface ratio.[8.98] Members with large values of this ratio

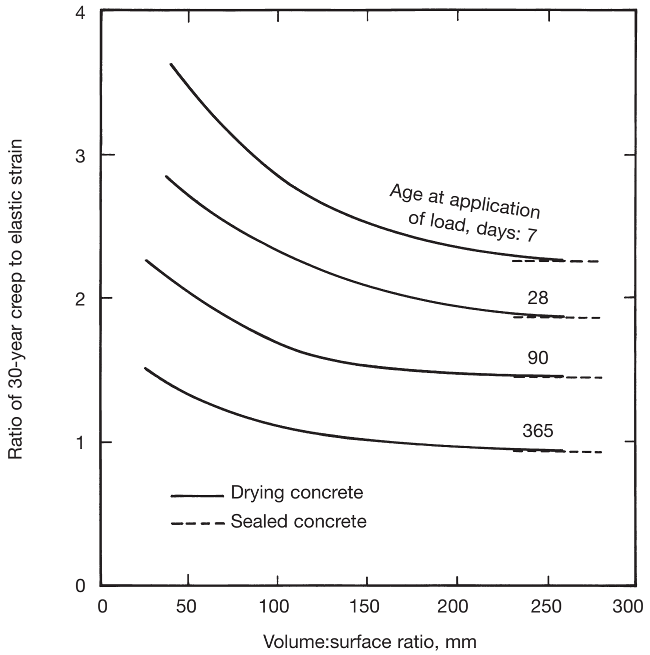


Figure 8.12: Influence of volume:surface ratio on the ratio of creep to elastic strain for sealed concrete and for drying concrete stored at a relative humidity of 60%[8.101]

will have longer diffusion paths for moisture loss and consequently the early rate of drying creep will be lower, as will be the magnitude of the ultimate creep.[8.99] This is illustrated in Figure 8.12. Current methods of prediction may underestimate creep of small members.[8.100]

Drying conditions (relative humidity and temperature)

In a normal creep test under drying conditions, the total creep comprising basic and drying creep will increase with a decrease of ambient relative humidity or an increase in temperature. This effect of the intensity of drying has been confirmed in many tests.[8.95, 8.100, 8.102]

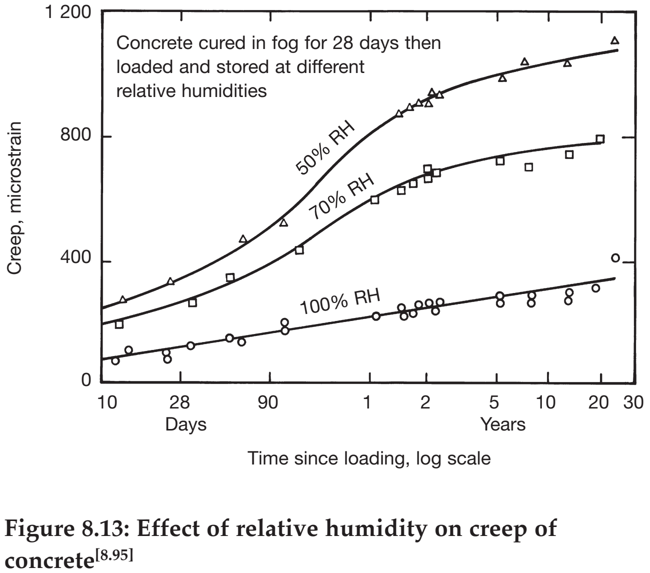


Figure 8.13: Effect of relative humidity on creep of concrete[8.95]

Lower relative humidities considerably increase the creep of concretes that are allowed to dry for the first time under load. Figure 8.13 illustrates the higher creep resulting from lower relative humidities for a concrete cured at 100% relative humidity before being loaded and thereafter exposed to different humidities. The enhanced creep seems to be due to higher creep rates during the earlier stages of drying. The relative humidity effect is much smaller when concretes have previously reached hygral equilibrium with the environment before the application of load. In these cases, concretes previously dried to lower relative humidities can be expected to exhibit somewhat lower creep than concretes conditioned at higher relative humidities. In practical terms this implies that it is advantageous to allow a structure to achieve some drying prior to loading.

The influence of elevated temperatures on creep depends on the time at which the temperature of concrete rises relative to the time of load application.[8.103] Figure 8.14 shows that for saturated concrete, curing at the test temperature reduces creep in comparison with heating and loading at the same time, due to improved hydration. Relationships for unsealed concrete are given in Figure 8.15, showing three distinct regions: creep increase up to about 50ºC; creep decrease between 50ºC and about 120ºC; further creep increase between 120ºC and about 400ºC. The initial increase in creep results from expulsion of evaporable water. After removal of this water, creep is reduced and becomes equal to that of pre-dried (desiccated) concrete. The increase in creep strain at temperatures higher than approximately 150ºC, as shown in the figure, is a result of the alteration of hydration products.

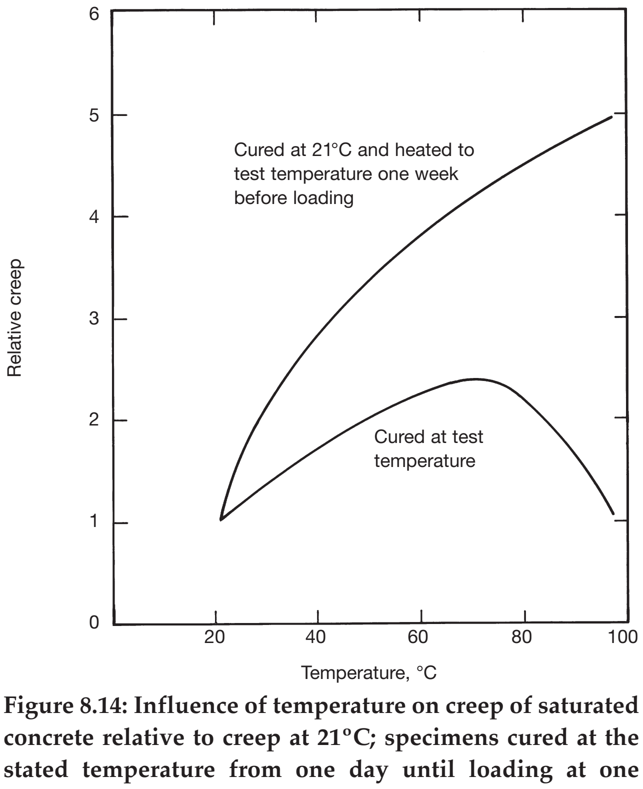


Figure 8.14: Influence of temperature on creep of saturated concrete relative to creep at 21°C; specimens cured at the stated temperature from one day until loading at one year[8.101]

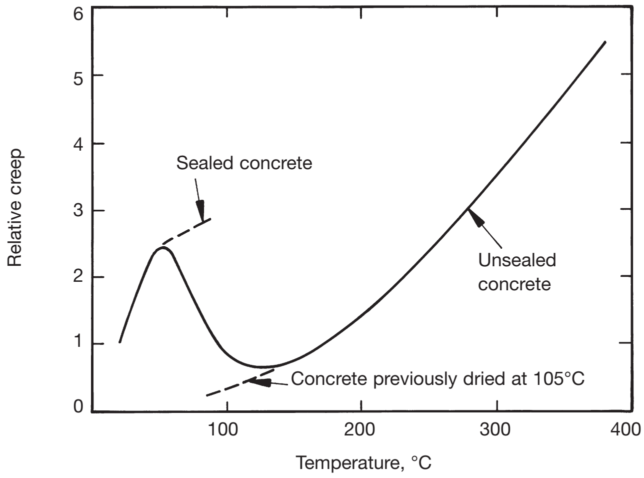


Figure 8.15: Influence of temperature on creep of unsealed concrete relative to creep at 20°C; specimens moist-cured for one year and then heated to the test temperature 15 days before loading.[8.101]

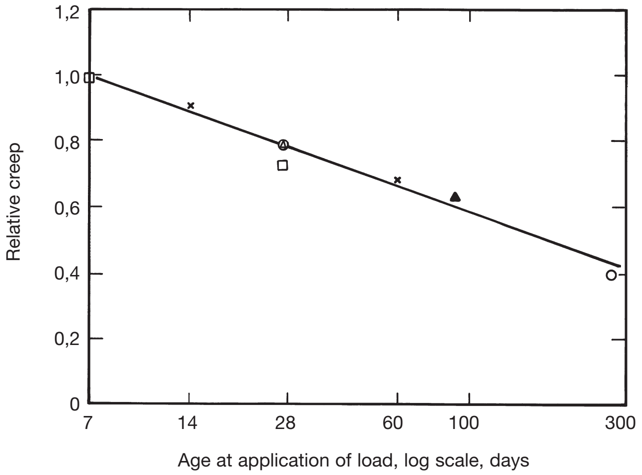


Figure 8.16: Influence of age at application of load on creep of concrete relative to creep of concrete loaded at seven days, for tests of different investigators; concrete stored at a relative humidity of approximately 75%[8.107]

For creep at low temperature, consult references 8.104 to 8.106.

Stress:strength ratio, curing and age at loading

The stress:strength ratio accounts for a number of factors that affect concrete creep, such as magnitude of stress, age at loading, duration and type of cure, and w:c. Creep increases with a higher stress and decreases with increasing strength. A linear relationship between concrete creep and stress:strength ratio is normally assumed, and this is reasonable for stress levels less than about 40% of short-term strength. Furthermore, for the same stress:strength ratio, creep is sensibly independent of w:c. Different concretes can therefore be compared on the basis of stress:strength ratio, provided the same aggregates are used and different volumetric paste contents are accounted for.

Strength increases with age provided adequate curing is achieved, and concretes loaded at later ages will creep less (see Figure 8.16). It is noticeable that the age effect continues for a very long time, and this has been attributed to an ageing process which increases the bonding within calcium silicate hydrate (CSH). Hence any factor that will increase the concrete strength before loading will generally reduce the creep.

Time under load

Creep occurs for a very long time, if not indefinitely. Figure 8.13 indicates that even at ages approaching 30 years, measurable creep rates still occur. However, depending on the size of the member, 50% of the 20-year creep will occur after about two to six months under load, while 80% will occur after about one to two years.

State of stress

Although far less work has been done on creep of concrete under stress states other than uniaxial compression, the following trends have emerged:

- Creep in tension at a given tensile stress:strength ratio is of the same order of magnitude as compressive creep at a corresponding compressive stress:strength ratio. However, the initial rate of creep may be higher in tension.
- The torsional creep curve is of the same form as the compression curve, and torsional creep is affected qualitatively by the same variables as compressive creep.
- Flexural creep is complicated by the presence of compression and tension zones, and by cracking in the tension zone. In reinforced concrete beams, very little creep occurs in the tension zone, while creep in the compression zone will be restrained by compression reinforcement if present.
- Creep under dynamic loading may be complicated by fatigue microcracking, and for a given maximum stress is greater than static creep.
- A Poisson's ratio creep component appears to occur for uniaxial compressive loading, and is approximately the same as for short-term loading provided corrections for lateral shrinkage are made.

8.4.5 Estimation of creep

Many publications are available in which empirical methods for the estimation of creep are presented.[8.3, 8.6-8.8, 8.93] The complexity of the factors influencing creep makes it very difficult to prepare a unified approach that will be applicable to all situations and incorporate all relevant factors. Hence estimation is at best imperfect, and will depend on the amount and quality of initial data available to the designer, and on the importance of the estimation to the structural problem in hand. At the lowest level of estimation, a single creep value may be estimated, based on normal conditions and possibly a knowledge of concrete strength, environmental humidity and member size. For the majority of concrete structures, this approach is probably adequate. At a higher level, the designer will require knowledge of concrete materials and mixes, and ages of loading, and will usually have the means to determine strains at any time, using superposition if necessary. The CEB-FIP document[8.7] and the RILEM Model B3[8.8] represent this level. At the most advanced level, relevant to structures in which time-dependent movements are critical, a considerable amount of data based on a synthesis of available information and results of specific laboratory tests is required, and special predictive models have to be developed.

BS 8110[8.2] and SANS 10100-1:2000[8.4]

The BS 8110 method,[8.2] taken up in the SANS Code,[8.4] is presented here due to its simplicity. According to this method, the final (30 year) creep strain can be estimated from the equation

Final creep strain $\varepsilon_c = \varepsilon_e$ x creep factor

$$= \frac{\text{stress}}{E_t} \times \psi \qquad (18)$$

where ε_e is the elastic strain dependent on the stress and modulus of elasticity E_t at the age at loading, t. The creep factor ψ can be obtained from Figure 8.17 with a stated coefficient of variation of about 15%. The effective section thickness is calculated as twice the cross-sectional area divided by the exposed perimeter. For a slab drying from both faces, this reduces to the slab thickness.

Further details for the estimation of creep are:

- For members that are fully sealed or immersed in water, in which case only basic creep occurs, the creep factor should be determined using an effective section thickness of 600 mm.
- The final creep recovery one year after unloading may be estimated as approximately 30% of the elastic recovery on unloading (ie 0,3 x stress reduction/E_t).
- The development of creep with time may be assumed to be 40%, 60% and 80% of the final creep during the first month, six months and 30 months respectively under load when concrete is exposed to conditions of constant relative humidity.
- The modulus of elasticity can be estimated from a knowledge of the characteristic cube strength at the age of loading and some estimate of the quality of the aggregates. Details are provided in section 8.2.3.
- Allowances for different aggregates can be made where necessary using references 8.18 and 8.19.

The above method is based on limited initial data (exposure conditions, age at loading, size of member) and is suitable only for approximate calculations. Secondary variables such as temperature, aggregate stiffness and cement type are omitted. Constant relative humidity is assumed, and this may introduce error when considering the variations in relative humidity. Table 8.3 shows relative humidity figures for selected centres in South Africa, and these can be used in conjunction with Figure 8.17.

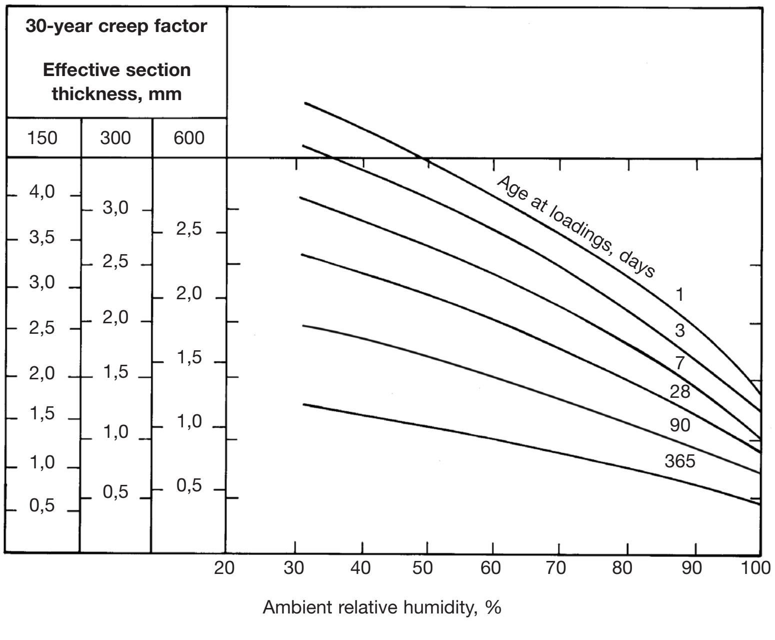


Figure 8.17: Effects of relative humidity, age of concrete at loading and section thickness upon creep factor[8.2]

Table 8.3: Average relative humidities in South Africa (based on mean hourly values for the period generally between 1932 and 1950)

Location	Average relative humidity, %
Bloemfontein	53
Cape Town	78
Durban	78
Johannesburg	59
Kimberley	46
Port Elizabeth	79
Pretoria	60
Upington	39

CEB-FIP[8.7, 8.99]

Two documents are available from the CEB-FIP[8.7, 8.99] in which methods of estimating creep are given. More initial information is required and fuller answers are provided than in the method presented above. The 1978 document[8.99] adopts the two-component creep formulation, in which the recoverable and irrecoverable portions are separately assessed (see section 8.4.6). The limiting recoverable creep is taken as 0,4 x the elastic strain. The irrecoverable creep can be estimated from a knowledge of the humidity of storage, size of member and mix proportions. The development with time of both components can be estimated, with suitable corrections for cement type and variable temperature.

In the 1990 document,[8.7] concrete is considered as an aging linear visco-elastic material. A product formulation for the prediction of creep has been used, ie creep after a given duration of loading can be predicted from the product of a basic creep coefficient which depends on the age of concrete at loading and a function describing the development of creep with time. (The alternative, such as in the 1978 code, is that creep may be described by a summation formulation as the sum of delayed elastic and viscous strains.) The principle of superposition may be used with both the above methods for computing strains under variable stress.

RILEM Model B3[8.8]

A further approach is available, the so-called "RILEM Model B3", deriving from the work of RILEM Technical Committee 107-GCS.[8.8] The model is based on the work of Bazant and co-workers.[8.108-8.117] This complex model incorporates both extrinsic and intrinsic variables that affect creep, such as relative humidity, temperature, concrete age at first loading, section size, w:c ratio, cement type, cement content and aggregate:cement ratio. The model requires detailed information on the actual concrete to be used, and is therefore not suitable at the preliminary structural design stage. For creep prediction, the model separates basic and drying creep components, and determines the creep compliance function rather than the creep coefficient. This allows the creep prediction to be separated from the estimate of the elastic modulus, thereby avoiding a separate component of error.

BS EN 1992:2004[8.6]

The code provides a graphical method for determination of the creep coefficient, based on a computerised data bank with a stated coefficient of variation in the order of 20%. Figure 8.18 shows the graphical method for concrete situated in outside conditions (in the UK) (RH = 80%). The document provides a second graph for inside conditions (RH = 50%). The values obtained from the figure are considered a suitable prediction for the final creep coefficient for structures where great accuracy in creep prediction is not required. The figure uses the following symbols:

$\psi(\infty, t_0)$ = final creep coefficient
t_0 = age of the concrete at first loading (days)
h_0 = notional size = $2A_c/u$, with A_c = cross-sectional area and u = perimeter exposed to drying
S = applies to slow hardening cements
N = applies to normal cements
R = applies to rapid hardening cements
C20/25 = strength classes according to EN 206-1: C $f_{ck}/f_{ck,cube}$, f_{ck} = characteristic cylinder strength
$f_{ck,cube}$ = characteristic cube strength

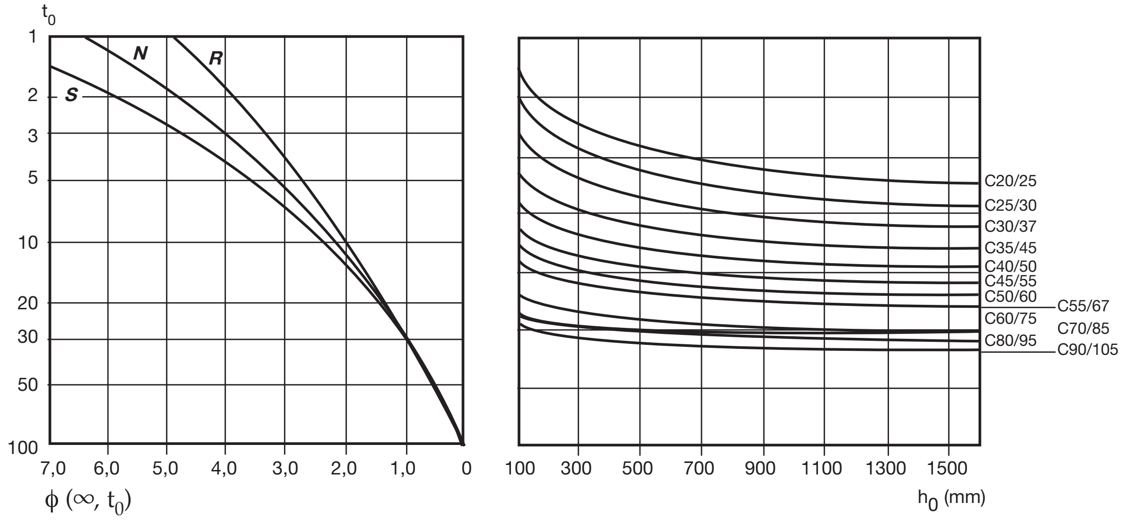
In cases where the concrete is subjected to compressive stresses greater than 45% of the characteristic compressive strength at first loading, for example as a result of prestressing, the code specifies that creep non-linearity should be considered, and presents a non-linear notional creep coefficient. For further details, the EN document should be consulted.

ACI 209R-92[8.3]

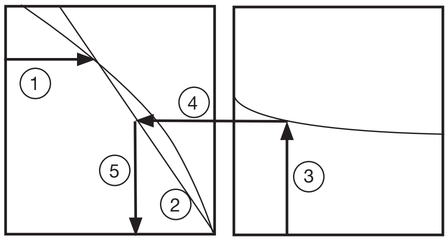
According to the ACI method of creep prediction,[8.3] the creep coefficient is given by the product of several partial coefficients which cover the major factors likely to influence creep. This method applies to normal-density and low-density concretes, and ASTM type I and III cements (ordinary and rapid-hardening portland cements).

Time development of creep

Based on statistical analysis of results of laboratory tests to relatively long ages, Brooks and Neville[8.118] suggest power and logarithmic-power expressions for estimating the specific creep $C_{c,t}$ at any time t after application of load (t > 28 days) from a measure of the specific creep $C_{c,28}$ after 28 days under load as follows: for periods up to five years under load, the expressions for the majority of concretes, regardless of the w:c ratio or the type of aggregate, are

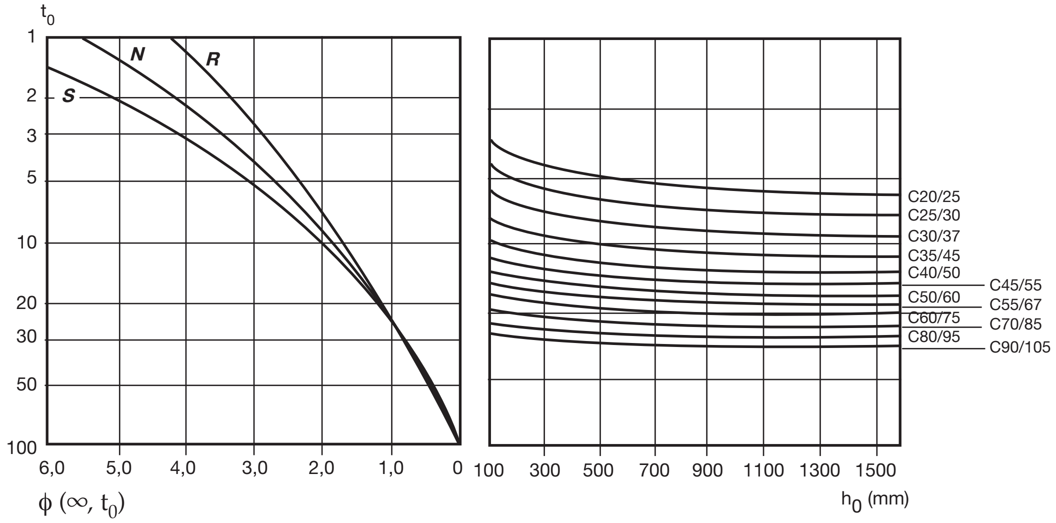


a) inside conditions - RH = 50%



Note:

- intersection point between lines 4 and 5 can also be above point 1
- for $t_0 > 100$ it is sufficiently accurate to assume $t_0 = 100$ (and use the tangent line)



b) outside conditions - RH = 80%

Figure 8.18: BS EN 1992:2004 method for determining the creep coefficient for concrete under normal environmental conditions. [8.6]

basic creep $C_{c,t} = C_{c,28} \times 0{,}5t^{0{,}21}$ .... (19)

total creep $C_{c,t} = C_{c,28} \times [-6{,}19 + 2{,}15 \log_e t]^{0{,}38}$ ....(20)

The above expressions are generally valid for ages of loading greater than seven days. The earlier the age of creep measurement, the more difficult becomes the estimate of ultimate creep, and great care must be taken in extrapolating from short-term tests.

Accuracy of creep prediction

Some of the prediction methods discussed above were compared with creep results for South African concretes in the 6th edition of this book (1986). The analyses indicated that the methods differed widely in their ability to produce accurate results for a wide range of concretes loaded at different ages. The ACI method tended to underestimate creep strain. The BS 8110 method appeared to be accurate at earlier ages, but overestimated at later ages. The CEB-FIP

(1978) method was accurate to within 15% of plotted values for two ages of prediction.

Recent South African research by Ballim[8.119, 8.120] and Fanourakis[8.121] shows the following:

- The CEB-FIP MC90 and BS 8110 models show good prediction for concretes in the range of 30 MPa to 50 MPa. On the other hand, RILEM Model B3 significantly underpredicted the creep strains of 30-MPa concretes and showed only fair prediction for the 50-MPa concretes at later ages.
- The consideration of aggregate type in the BS 8110 model, using modification factors proposed in references 8.18 and 8.19, resulted in less accurate prediction of creep strain.
- The models are generally unable to account for aggregate effects in concrete.
- The accuracy of predictions does not increase with the complexity of the models used, or with an increasing number of variables accounted for in the models.

In view of the above findings, Fanourakis and Ballim[8.121] recommend that the BS 8110 (1985) model be used for the prediction of creep strains under South African conditions.

Further guidance concerning the effects of creep to be allowed for in prestressed concrete design is given in the South African code of practice for the structural use of concrete.[8.4] It cannot be stressed too strongly that the recommendations discussed above are necessarily general, and specialist literature should be consulted for critical cases. The problem of creep prediction is essentially a statistical one, and the scatter of actual data may vary significantly from the mean values proposed. Hence, where creep-sensitive structures are being analysed, upper- and lower-bound solutions should be obtained.

8.4.6 Other aspects including theoretical considerations

Creep recovery

Referring to Figure 8.1, a recovery of strain is experienced by the concrete on unloading. This recovery consists of an immediate elastic recovery followed by a time-dependent creep recovery which is smaller than the preceding creep and reaches a limiting value in due course. Hence, a portion of the creep is irrecoverable, leading to residual deformation in the concrete. This phenomenon of creep recovery has led to creep being viewed as the sum of two components, namely irrecoverable creep or flow, and recoverable creep or delayed elastic strain. The irrecoverable creep portion of the total creep increases with time under load, as shown in Figure 8.19, so that at medium to later ages this component dominates creep behaviour. The variation of creep with extrinsic factors such as aging (time and temperature), drying conditions and stress appears to be due to variation of the irreversible component of creep. Although the creep recovery may depend on mix parameters such as w:c and aggregate content, it can be taken as sensibly constant for the extrinsic factors mentioned above.

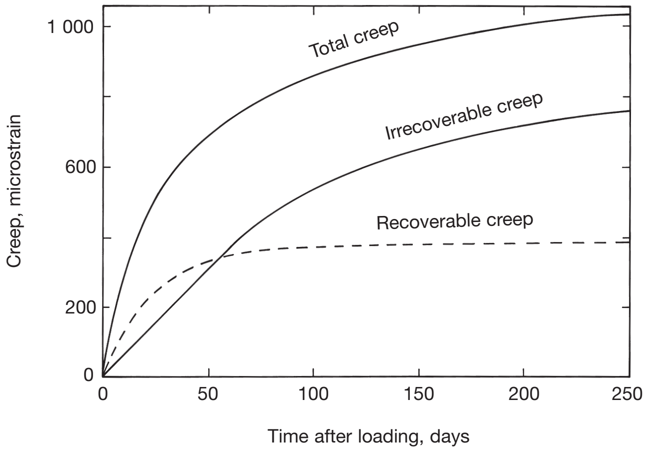


Figure 8.19: Variation of creep and creep recovery with time

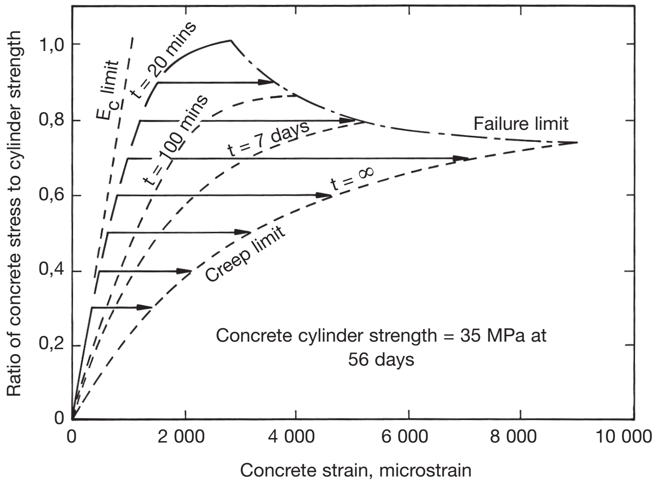


Figure 8.20: Creep and failure limits for a typical concrete[8.122]

Creep at high stress levels

Referring to the additional component of creep experienced at high stress levels above about 70 to 80% of short-term compressive strength, the microcracks which were originally initiated at the coarse aggregate-mortar interface spread extensively into the mortar phase. If the sustained compressive stress is high enough, the microcracks can propagate with time and eventually coalesce to cause complete breakdown of the material. In this way time-dependent failure may occur, as shown in Figure 8.20, taken from Rüsch.[8.122] There is evidence[8.123] that the critical stress level is higher in high-strength concrete. Figure 8.20 also shows that the degree of non-linearity of the stress-strain relation increases with increasing stress level. For the normal range of working stresses less than about 40% of short-term strength; however, the assumption of linearity is reasonable.

Creep mechanisms

Despite considerable work over the years, the mechanisms of creep are not yet fully understood.[8.124] Reviews are given in references 8.54, 8.125 and 8.126. Possible hypotheses are:

Recoverable or reversible creep (or delayed elastic strain)

- Diffusion of water from areas of hindered to unhindered adsorption which reduces swelling pressure leading to a reduction in inter-particle spacing.[8.127]
- Diffusion of water from high- to low-pressure areas causing gradual load transfer from liquid to solid phase (hence a delayed elastic-type strain).[8.128, 8.129]
- Removal of interlayer and intralayer water, under the action of external load, causing reduction in layer thickness and spacing.[8.130, 8.131]

Irrecoverable or irreversible creep

- Weakening of interparticle bonds by disjoining pressure facilitating relative sliding of the layers.[8.132, 8.133]
- Displacement of gel layers relative to each other, introducing a process of breaking and remaking of interparticle bonds.[8.130]
- Formation of new bonds as particles are brought into close contact for the first time.

The above remarks apply to stresses in the concrete not exceeding about 40% of ultimate strength (compressive). At higher stress levels, progressive microcracking occurs, resulting in additional components of strain which are very much larger than the normal moisture-related creep strains at lower stresses (see Figure 8.20).

Creep-time relationships

Two time variables have an important influence on the creep of concrete:

τ = age of the concrete at the time of loading
t = total age of the concrete

$(t - \tau)$ is therefore the time under load.

The interrelationship of these two variables is illustrated in Figure 8.21, showing a typical "creep surface" for concrete, which itself can be represented by an expression:

$$\varepsilon_c = \sigma f_1(\tau) f_2(t - \tau) \qquad (21)$$

(i) $f_1(\tau)$ is the age multiplier, and is generally represented as a power of the age of loading, ie τ^{-m} or as an inverse function of strength (since the greater the strength, the lower the creep)

(ii) $f_2(t - \tau)$ is an expression for development of creep with time and can be represented by either limiting value or non-limiting value expressions:

Limiting value expression:

$$\text{hyperbolic: } \varepsilon_c = \frac{a(t-\tau)^n}{b + (t-\tau)^n}$$

where n usually equals 1 ....(22)

Non-limiting value expression:

$$\text{logarithmic: } \varepsilon_c = A + B \log(1 + t - \tau) \qquad (23)$$

a, b, A and B are constants

The constants a and b for the hyperbolic relationship can be found by plotting the linear relationship between $(t - \tau)/\varepsilon_c$ and $(t - \tau)$. The logarithmic formula gives a continuously increasing value for creep, and may overestimate long-term creep. The constants A and B can be found by plotting ε_c against $\log(1 + t - \tau)$.

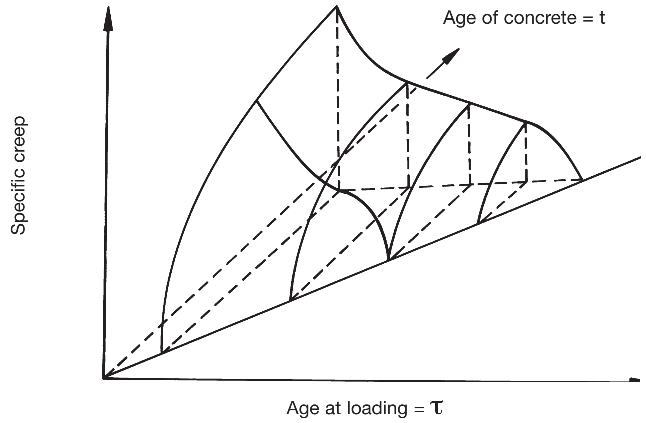


Figure 8.21: Creep surface for concrete

Creep under varying stress

On occasions a designer is faced with having to estimate the strain response of a structure under conditions of variable stress or vice versa. The linear relationship between creep and stress allows the Boltzman superposition principle to be applied to concrete for the solution of such problems. According to this linear viscoelastic law, the total creep $\varepsilon_c(t)$ at age t can be found by summing the creeps of the individual stress increments applied at different ages during the preceding stress history. Mathematically we can write:

$$\varepsilon_c(t) = \sum_{i=1}^{n} \Delta \sigma_i . c_{ci} \quad \text{for n increments of stress } \Delta\sigma \qquad (24)$$

where c_{ci} represents the series of specific creeps corresponding to the differing times under load. A numerical solution to the above expression is possible, assuming that the stress record is divided into stepped increments. While concrete does not perfectly obey the superposition principle, the principle nevertheless remains a convenient working assumption.

Simplifications of the general superposition principle have resulted in various techniques for the numerical solution of practical problems. These are covered in various references[8.93, 8.134] and two are briefly mentioned here. The first is the effective modulus approach, in which the creep strain ε_c is added to the elastic strain ε_e, to give an effective, or reduced, modulus of elasticity E'_c.

$$E'_c = \frac{\text{stress}}{\text{total strain}}$$

$$= \frac{\sigma}{\varepsilon_e + \varepsilon_c}$$

$$= \frac{\sigma}{\varepsilon_c + \psi\varepsilon_c}$$

$$= \frac{E_c}{(1 + \psi)} \quad \text{.........} \quad (25)$$

where E_c is the short-term elastic modulus, and ψ is the creep factor. The advantage of this simple technique is that the full range of elastic solutions to structural problems is available, and provided the stress history does not have large fluctuations, reasonable solutions can be obtained with the minimum of difficulty.

A more complex solution capable of providing adequate answers for a wide range of variable stress histories is the *rate of flow* technique. The creep is considered to be made up of two components, the one recoverable and the other irrecoverable, each of which is proportional to the applied stress. The recoverable component has a limiting value not greatly affected by factors such as age at loading, conditions of drying or temperature, and can simply be taken as 0,4 times the elastic strain. This strain is observed as the creep recovery after the stress is removed. The irrecoverable component, or the flow, is taken as independent of the age at loading, and depends for a given mix merely on the age of the concrete. Hence the irrecoverable creep can be found for various ages of loading from a single curve. This method forms the basis of the 1978 Model Code of the CEB-FIP.[8.99] Its advantage lies in the fact that data on only one creep curve and one creep recovery curve are required.

Computational and mathematical approaches

Particularly for the creep analysis of complex structures such as concrete pressure vessels, long-span bridges, structures built sequentially and very tall buildings, the use of computers together with inelastic finite element methods is necessary. Such methods can incorporate non-homogeneous behaviour due to reinforcement, concrete cracking, and moisture and temperature effects.[8.135-8.137] With the development of computational approaches has been the advance of sophisticated mathematical models to describe the physical effects of creep and shrinkage and relate these to structural effects.[8.136, 8.138] The sophistication and complexity of these approaches has rendered application by the designer difficult, and there is no guarantee of vastly improved accuracy of prediction in many cases. Ultimately, probabilistic models based on the stochastic nature of the phenomena may be the best approach.[8.136]

8.4.7 Creep testing of concrete

There is no South African or British Standard for creep testing. However, numerous types of creep apparatus have been designed, usually based on mechanical or hydraulic principles.[8.54, 8.139-8.141]

ASTM C 512-02[8.142] is the American standard. Creep testing involves applying the stress over a short time period (two to five minutes) and then measuring long-term total strain. Companion unloaded specimens are used for measuring shrinkage/swelling strains.

RILEM CPC-12, 1983 *Measurement of creep in compression*[8.143] is a European document on creep testing. Various environments of storage are permitted.

8.4.8 Creep literature and further reading

A great deal of literature has appeared over the years dealing with concrete creep, and bibliographies of the American Concrete Institute are listed in the references.[8.144, 8.145] The greater part of work on creep is reported in journals such as *Cement and Concrete Research, Magazine of Concrete Research, Materials and Structures,* and *Proceedings of the American Concrete Institute.* Further reading is given in references 8.146 to 8.149, as well as in the proceedings of a series of international conferences on creep and shrinkage of concrete.[8.150-8.153]

8.5 Shrinkage of concrete

Concrete experiences volume changes in both the fresh and hardened states throughout its existence. This section is mainly concerned with volume changes related to movement of moisture into and out of normal-strength concrete. Loss of moisture leads to the phenomenon of shrinkage, and the aspect of drying shrinkage in hardened concrete will receive most attention. However, it should be noted that measured shrinkage of concrete under drying conditions will include an autogenous component, and a drying component. The autogenous shrinkage develops very early (immediately after setting) due to internal consumption of water in hydration reactions, and the rate then reduces rapidly. In normal-strength concrete (w:c > 0,40) which experiences little autogenous shrinkage, the total measured shrinkage is usually taken to be the drying shrinkage. Strictly, drying shrinkage is the additional shrinkage caused by moisture loss from the concrete. For high-strength concrete, the drying component is of the same order as the autogenous shrinkage (see section 8.6). There are also other types of volume changes that will be briefly discussed later.

The type of volume change that is of most concern in structural performance of normal-strength concretes is that associated with an interchange of moisture between the hardened concrete and the environment. If the net flow of moisture is from the environment to the concrete, the result is an increase in volume, called swelling. A net outflow of moisture from the concrete to the environment results in volume decrease, termed drying shrinkage. For practical purposes, it is the linear shrinkage (longitudinal, one-dimensional shrinkage) that is usually of most concern. Conventional concrete will usually contain more water than can chemically be combined with the cement, with the consequence that in normal drying environments there is the tendency for moisture to be lost from the concrete, resulting in shrinkage. The rate at which moisture is lost is fairly slow and the strain responses are time-dependent. These strains are important and the designer must make reasonable allowance for them in order to ensure satisfactory structural performance.

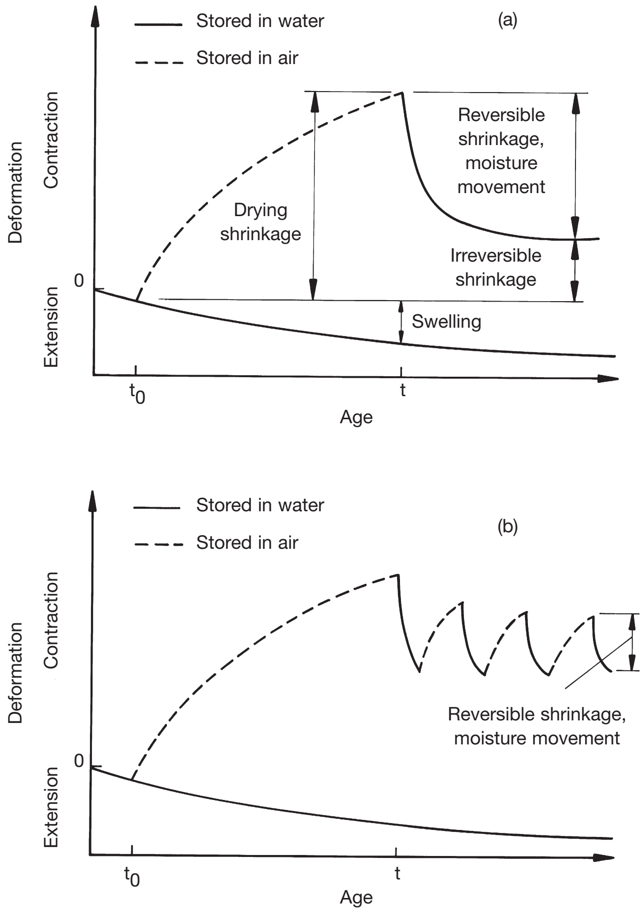


Figure 8.22: Moisture movement in concrete: (a) concrete that dried from age t_0 until age t and was then re-saturated, and (b) concrete that dried from age t_0 until age t and was then subjected to cycles of drying and wetting.[8.101]

Shrinkage makes up a portion of the total deformation in a concrete member. Figure 8.1 shows the various components of deformation (excluding thermal movements), and shrinkage is regarded as that portion of the total strain that is time-dependent and non-load-induced. Shrinkage and creep movements are closely related in that both are moisture-dependent and the source of both normally lies in the cement paste. The addition of aggregate modifies the paste properties, and it is useful to understand the behaviour of the paste itself as well as examining the composite behaviour of concrete.

8.5.1 Characteristics of shrinkage

The greatest shrinkage movement occurs on first drying, when a large part of the shrinkage cannot be recovered if the concrete is later re-wetted. On subsequent wetting and drying cycles, a portion of the original shrinkage movement is recovered with each wetting, until the process becomes more or less completely reversible. Shrinkage therefore consists of reversible and irreversible components, and Figure 8.22 shows the basic characteristics. Note that drying is very much slower than re-wetting. Shrinkage strains also depend on the environmental conditions, the composition of the concrete and the degree of hydration at the onset of initial drying.

8.5.2 Structural effects of shrinkage

A number of references provide useful information on the effects that drying shrinkage can have on structural performance.[8.3, 8.56, 8.154-8.156] In general, these can be divided into effects on cracking and effects on deflections.

Cracking

The free shrinkage strain of concrete usually exceeds its tensile strain capacity and concrete that is restrained will crack. Unrestrained concrete will not crack, but a condition of zero restraint is seldom achieved in practice. Cracking of reinforced concrete members due to loading stresses is an accepted aspect of design, but these cracks in normal service are small and limited to areas of high load intensity. Cracking due to shrinkage may, however, result in cracks of appreciable size, frequently at unexpected or random locations. Such cracking may be aesthetically unacceptable and is undesirable from the point of view of durability. In the case of liquid-retaining structures, cracks allowing leakage are unacceptable, and special design and construction precautions should be taken to control cracking in these structures. Design codes provide guidance on acceptable crack-widths for different exposure conditions and on control of cracking.[8.157]

Internal and external restraints may operate in a concrete member. Internal restraint is offered by aggregate and reinforcement, resulting in micro-cracking at the aggregate-paste interfaces and in tension being induced in the concrete with corresponding compression in the steel. At normal

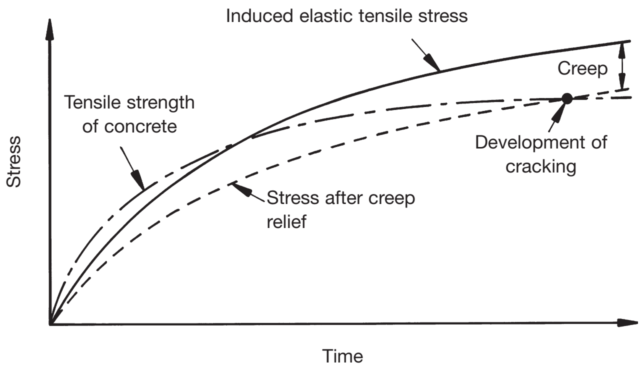


Figure 8.23: Schematic pattern of crack development when tensile stress due to restrained shrinkage is relieved by creep[8.159]

steel ratios, such concrete tensile stresses are unlikely to result in cracking of the concrete.[8.158] A further internal type of restraint is created by uneven drying of the concrete. The inner core dries more slowly than the outer zones and thus restrains the surface shrinkage, and this may lead to surface cracking.

External restraint is caused when new concrete is bonded to an older substrate, or when friction operates between a member and its base, for example in concrete pavement slabs. Such external restraint frequently leads to cracking. Particular attention should be paid to the problem of controlling shrinkage cracking in thin reservoir walls which are cast onto existing large bases. The function of shrinkage steel in such cases should be to distribute the cracks and limit their widths.

Tensile stresses due to restrained shrinkage are relieved to some extent by creep. This might result in less cracking, or alternatively cracking might occur later. This is shown in Figure 8.23 which is a representation of the development of cracking due to shrinkage-induced stresses. The mechanisms of relaxation in restrained concrete members were also discussed in section 8.4.3.

Adequate moist-curing of concrete delays the onset of shrinkage and increases the tensile strength of the concrete. On the other hand the elastic modulus of the concrete also increases with the period of curing, and stresses resulting from restrained shrinkage strains are therefore likely to be higher. The likelihood of cracking will depend mainly on potential shrinkage, relaxation properties, elastic modulus and tensile strength of the material, as well as the degree of restraint.

Measures to reduce shrinkage-induced cracking include:

- Mix design – the lowest possible water and cement contents consistent with strength and durability requirements should be used
- Use of strategically placed joints to accommodate shrinkage movement and control cracking
- Correct construction practice – for example to minimise differential shrinkage between adjacent members
- Application of proper curing methods

Furthermore, when large members are being placed, or when constructing during hot weather, precautions should be taken to limit the heat build-up in the concrete since, if final set occurs when the concrete is in an expanded condition due to increased temperature, an additional component of "thermal" shrinkage may result in a very large potential shrinkage for the member.

Expansive cements have been used overseas to offset the effects of shrinkage. Some success in controlling cracking has been achieved in the USA and Europe in this way.[8.160-8.163]

Deflections

In flexural members, shrinkage may affect deflections in two distinct ways. Firstly, in unsymmetrically-reinforced members, the concrete experiences a greater degree of free shrinkage in the compression zone than in the more heavily-reinforced tension zone where restraint is greater. Consequently, additional shrinkage-induced curvature is introduced which leads to deflections that are additional to normal load-induced deflections.[8.155, 8.156] Time-dependent shrinkage may also lead to additional tensile stresses over and above the load-induced stresses in the tension zone, causing existing cracks to increase in size, and possibly inducing additional cracking.[8.154] Hence, a time-dependent reduction in flexural rigidity may occur, leading to additional deflections. In design calculations, it is only the former effect that is generally allowed for. While shrinkage deflection is usually a small proportion of total deflection, it may account for up to 25% of deflection of beams that are singly reinforced. Methods of calculating shrinkage-induced deflections are outlined in reference 8.155 and in SANS 10100-1:2000: Part 1, Annex A.[8.4]

With reference to compression members, it should be noted that while column shortening is primarily due to creep, shrinkage will compound the creep effects and therefore cannot be ignored. Prestress losses due to shrinkage are also not negligible.

8.5.3 Factors affecting shrinkage

The paste is the source of shrinkage in concrete. The discussion that follows will deal with aspects of the shrinkage of both paste and concrete. The factors given in Table 8.1 will be used as a basis.

Structure of the paste

Hardened cement paste may be thought of as comprising solid gel particles or platelets, generally layered in form, with two types of pores present: very small gel pores which are formed by spaces between solid gel layers, and much larger capillary pores which are formed by the excess water above that required for hydration of the cement. This structure is primarily affected by w:c and degree of hydration. The lower the w:c and the greater the degree of hydration, the more will be the volume of hydration products (gel) formed, and the greater will be the ratio of gel pore to capillary pore volume. Water is held in the paste at different bonding energies, the interlayer and gel pore water being very much more tightly held than the free capillary water. Consequently, as paste dries it loses capillary water first, then adsorbed and gel-pore water, and finally interlayer water. The removal of free water in the capillaries leads to small shrinkages, while the removal of tightly bound water causes a larger component of shrinkage as forces of contraction come into play.

This is illustrated in Figure 8.24 which shows the effect of w:c on shrinkage of pastes. A more marked influence is seen at later ages than at early ages. The lower the w:c the lower the ultimate shrinkage, which is then also more quickly approached. The early ages of drying represent the removal of free water, and shrinkage is similar for all the pastes since only capillary water is being lost. The high-w:c pastes actually lose more water than the low-w:c pastes, but shrinkages are not very different. Once the free water has been lost, gel water begins to be removed, and the paste is subjected to contraction forces. The stronger pastes (lower w:c) will also be stiffer and will experience less contraction strain than the weaker pastes, despite the fact that they will contain a greater gel volume. Lower w:c pastes (and concretes) are also more impermeable, hindering the free movement of moisture from the paste microstructure.

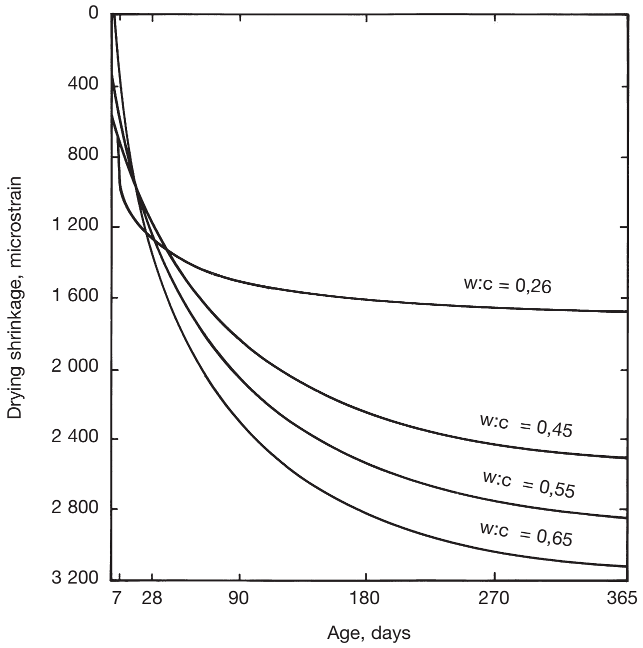


Figure 8.24: Effect of w:c ratio on shrinkage of cement pastes[8.164]

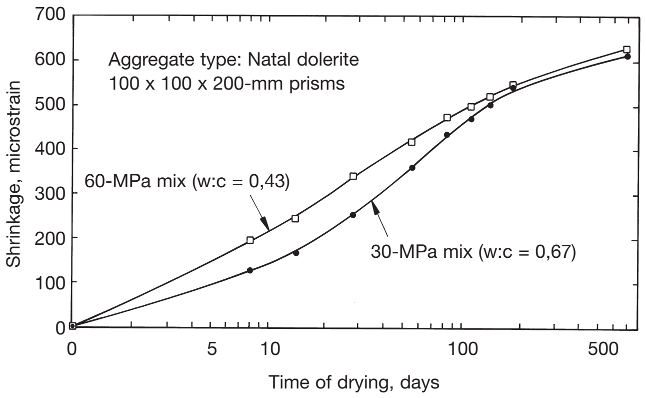


Figure 8.25: Rates of shrinkage of two different concretes made with the same local aggregates[8.19]

Turning to the effect of degree of hydration, it can be expected that pastes matured longer will exhibit greater proportionate volumes of gel pores, which will tend to increase the shrinkage assuming that drying from gel pores causes greater shrinkage than that from capillary pores.

The effects of w:c and degree of hydration can be combined in the single parameter of porosity. Porosity decreases with a decrease in w:c and an increase in the degree of hydration. From the information available, irreversible shrinkage decreases linearly with porosity.[8.165]

For concretes within the normal range of w:c (0,40 < w:c < 0,70), the influence of w:c on the ultimate shrinkage is much less pronounced than shown in Figure 8.24, primarily because the large volumetric portion of the aggregate masks the paste behaviour. However, lower w:c concretes do shrink faster at earlier ages (see Figure 8.25).

Cement composition and fineness

The tricalcium aluminate phase appears to have an effect on shrinkage, with gypsum content having a secondary effect.[8.166] It appears that an optimum sulphate content exists for minimum shrinkage, this content being the same as for minimum creep.[8.167] The effect of fineness of grinding is insignificant, except when the cement is extremely coarse (less than about 200 m²/kg) or extremely fine (greater than about 600 m²/kg).[8.168] There also seems to be some evidence that higher alkali contents in cements give higher shrinkages.[8.168] Thus there are differences in shrinkage between different types of portland cement, and between cements of the same type from different sources.[8.169]

Effect of cement extenders

An aspect of particular importance in regard to the effect of cement extenders is the state of exposure of the concrete, for example whether this state approaches sealed (large member) or rapid drying (small member) conditions. For instance, Mills[8.72] showed that specimens of GGBS concrete had significantly higher shrinkage and creep in exposed conditions when compared with plain portland cement concrete, but this effect was not apparent in sealed specimens. Neville[8.54] on the other hand quotes contradictory results drawing on Japanese data.

Work on South African materials[8.75] has shown that in general terms the incorporation of GGBS in portland cement concretes at normal replacement levels (50%) has the effect of causing a possible increase (20%) in shrinkage at early ages in small exposed laboratory specimens. This effect is usually reversed at later ages. Shrinkage strains are reduced in sealed specimens at all ages.

Jaufeerally[8.76] measured the shrinkage characteristics of concrete made with Corex slag (GGCS) and concluded that in higher strength concrete (water:binder ratio of 0,5 and below) the use of GGCS resulted in slightly lower drying shrinkage compared to plain PC concrete (10 to 15% less). For concretes with water:binder ratios of 0,6 and above no significant difference in drying shrinkage was observed between plain PC and GGCS concrete.

Work by Grieve[8.77] on concrete made with Matla fly ash showed that, for similar exposure conditions, the drying shrinkage of such concretes is very similar to plain portland cement concrete mixes at comparable compressive strengths, over a range of FA contents up to 30%. Mukheibir[8.170] similarly found that the influence of Lethabo fly ash on concrete shrinkage was minor. These South African findings confirm the reported findings of other investigators.

Referring to shrinkage of concrete containing CSF, a review[8.171] of several authors[8.172-8.176] indicated that concrete shrinkage is little influenced by CSF contents, at least up to 10% by mass of cement. CSF has the effect of densifying the microstructure of concrete, thus reducing the rate of moisture loss from concrete. Shrinkage thus takes place at a slower rate in CSF concretes, although the final shrinkage will be similar to other comparable concretes.[8.86] For concrete exposed to early drying, shrinkage may increase for lean CSF concrete (W/(C+S) > 0,60 where W, C and S are masses of water, cement and CSF respectively) and for high CSF contents (10% by mass of cement), because early drying inhibits the pozzolanic reaction. Concrete shrinkages are similar despite the fact that at equal W/(C+S), CSF pastes have a higher shrinkage potential than plain pastes, particularly for CSF contents above 10% by mass of the cement. The explanation appears to lie in the fact that, due to finer pore structure, CSF concrete dries more slowly than control concrete of equal strength; it is known that slow drying (or large specimen size) gives decreased shrinkage for a given concrete.

Admixtures

The effect of admixtures on shrinkage is variable and depends on the specific admixture and cement, and the exposure conditions. It is advisable that where admixtures are contemplated for use in movement-sensitive structures, specific tests using the actual proposed materials should be carried out. Where this is not practicable, allowances for possible increased creep and shrinkage strains should be made at the design stage. In all cases, material properties must be related to the specific structure in hand.

Aggregate and mix proportions

Aggregates have two effects on paste shrinkage: dilution and restraint. Dilution refers to the fact that shrinkage of the concrete will decrease with increasing aggregate concentration, while restraint refers to the fact that concrete shrinkage will reduce with increasing aggregate stiffness. The effect of aggregate concentration is shown in Figure 8.26. For normal concretes with aggregate volume concentrations of 65 to 70%, the concrete shrinkage may be only about 20% of the paste shrinkage.

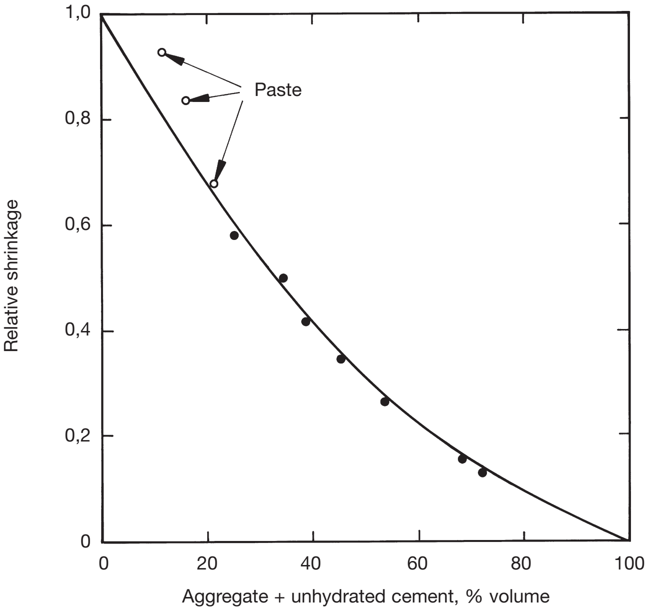


Figure 8.26: Effect of aggregate concentration on shrinkage of concrete[8.177]

The effect of aggregate stiffness is indirectly shown in Figure 8.27. The elastic modulus of concrete is closely related to the modulus of the aggregate, and provided w:c and aggregate concentration are constant, it can be expected that concrete shrinkage will also depend on the elastic modulus of the aggregate. However, Hobbs and Parrott[8.178] state that for high-quality dense non-shrinking aggregates employed in concrete with w:c greater than 0,4, the concrete shrinkage is effectively independent of aggregate stiffness. It would appear therefore that the effect of aggregate stiffness is important only in concretes with low-stiffness aggregates such as low-density aggregates or certain weathered materials. There is some disagreement on this point since compressible aggregates are also generally those with a high surface area, and earlier studies at the NBRI in Pretoria suggest that a confusion of these two factors may have influenced the published findings.[8.179]

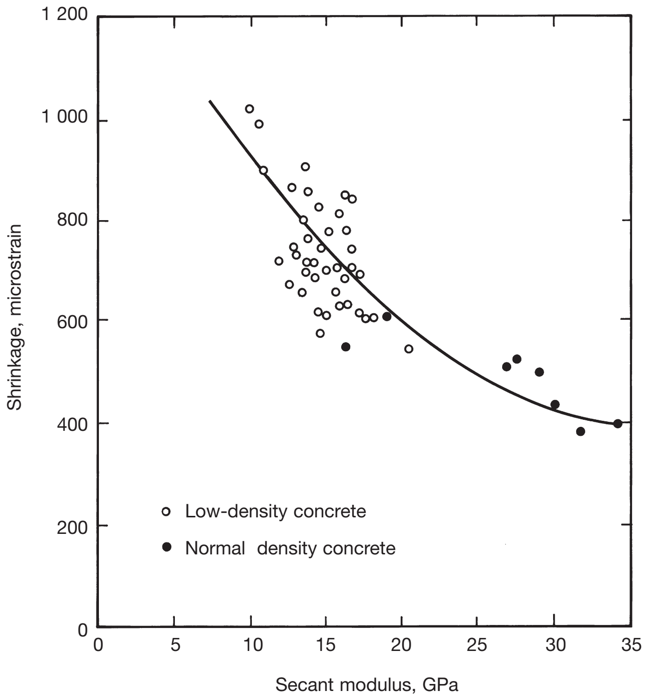


Figure 8.27: Relation between drying shrinkage after two years and secant modulus of elasticity of concrete (at a stress:strength ratio of 0,4) at 28 days[8.180]

The NBRI investigations in fact show that "a relatively good correlation exists between the shrinkage of the mortar and the specific surface of the mortar, and between the shrinkage of the mortar and the specific surface of the aggregates."

The effect of different aggregates on concrete shrinkage was also studied by Troxell et al.[8.95] The aggregates in increasing order of concrete shrinkage were quartz, limestone, granite, basalt, gravel and sandstone. It is possible that aggregate porosity as well as stiffness were factors in these tests.

South African work[8.18, 8.19] on the influence of aggregates on concrete shrinkage (also referred to in section 8.4.4) indicates that, for concretes of similar 28-day strength and similar slump (50 to 75 mm), concrete shrinkage can vary by up to about ± 30 to 40%. The results include the effects of different water demands of the various aggregates. Dolomites gave low concrete shrinkage, while a siltstone aggregate from the KwaZulu Natal midlands gave abnormally high concrete shrinkage. Other rock types gave variable shrinkage depending on their particular source.

Further work on use of Reef quartzite containing shale as concrete aggregate shows that shale causes increases in concrete shrinkage that are almost linearly proportional to the amount of shale in the aggregate. This is regardless of whether the shale is in the form of sand or stone – see Figure 8.28.[8.120]

Pickett studied the effect of aggregate stiffness and concentration on shrinkage,[8.181] and derived the expression:

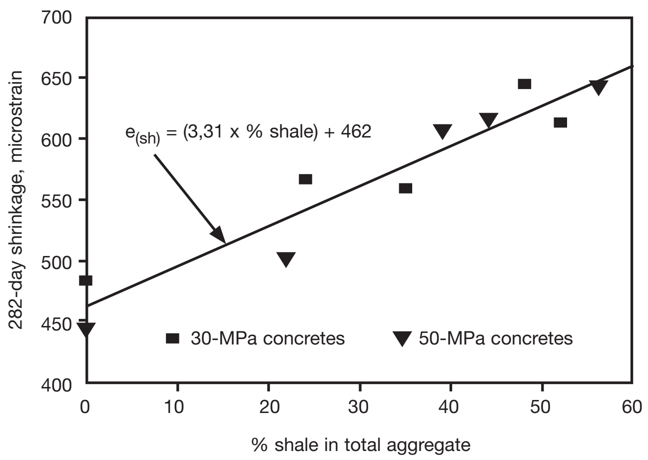


Figure 8.28: Effect of the proportion of shale in the total aggregate on 282-day shrinkage of concretes.[8.120]

$S_c = S_p (1 - V_a)^n$ ....(26)

where S_c and S_p are the shrinkage of the concrete and paste respectively, V_a is the volume fraction of the aggregate, and n is a complex function of the elastic moduli and Poisson's ratios of the concrete and aggregate. Pickett's model was that of small spherical elastic particles embedded in a homogeneous matrix, and his test results yielded a value for n of 1,7 although values from 1,2 to 1,7 have been measured elsewhere.[8.182]

Equation (26) predicts that for a normal concrete with an aggregate content of about 70%, the concrete shrinkage will be only about 10 to 15% of the corresponding paste shrinkage. Furthermore, for paste contents varying from 30 to 40%, the equation predicts that shrinkage will increase by about 6% for each 1% increase in paste content. Hence, even minor changes in mix proportions may have large effects on shrinkage.

Aggregate properties such as size and grading affect shrinkage of concrete indirectly through their effect on the water requirement of a mix. An advantage can be gained by using larger maximum size aggregate with a corresponding increase in total aggregate content.

The above discussion has generally referred to normal density non-shrinking aggregates; the situation is quite different for aggregates that are themselves dimensionally unstable. Shrinking aggregates can increase concrete shrinkage enormously, and since large areas of South Africa are underlain by rock types that exhibit drying shrinkage, this subject is discussed in Chapter 3.

Although w:c was shown to have a measurable influence on paste shrinkage, this effect is masked by the considerably lower shrinkages in concrete. Figure 8.29 shows that for typical concrete aggregate contents in the region of 70%, and w:c between about 0,45 and 0,65, w:c has very little influence.

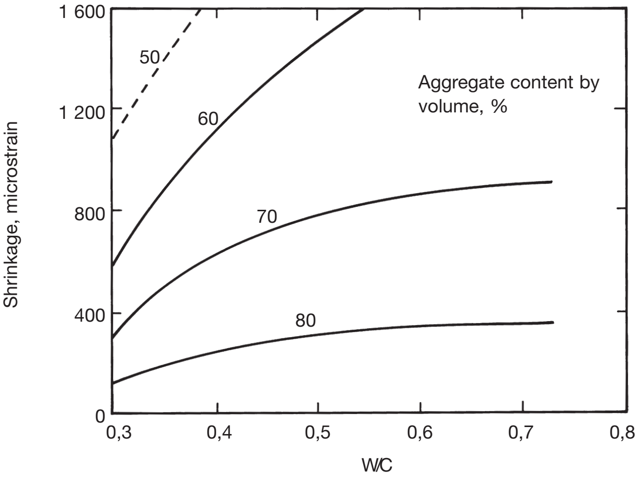


Figure 8.29: Influence of w:c ratio and aggregate content on shrinkage[8.183]

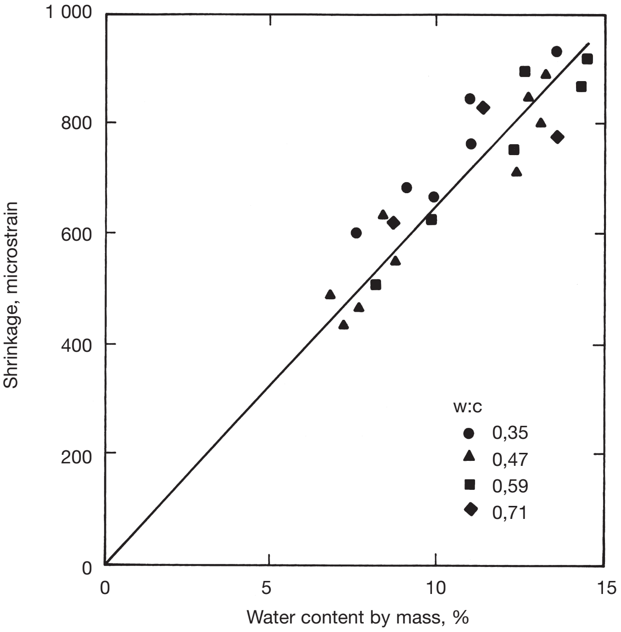


Figure 8.30: The linear relation between shrinkage and water content for a variety of mix proportions[8.184]

Water content has an important influence insofar as it affects the volume of the restraining aggregate. Hobbs and Parrott[8.178] state that the shrinkage of concrete is approximately proportional to the original water content by mass. This is illustrated in Figure 8.30, which shows that for a wide range of aggregate:cement and water:cement ratios, the shrinkage is related to original water content.

Member size and shape

Drying of concrete takes place from the exposed surfaces. This generates moisture-content differentials within the sample, with associated restrained strains and internal strain gradients. Thus, although potential drying shrinkage is, at least conceptually, an intrinsic property of a concrete, actual observed shrinkage will depend upon member geometry and dimensions.

Also, large sections are less affected by carbonation and have longer effective curing times, hence reducing the carbonation shrinkage component. The observed shrinkage decreases with increase of member size, and is a function of the volume:surface area ratio of the member. Generally a power relationship of the form

$$S_c = f\left(\frac{A}{V}\right)^n t \quad \ldots\ldots (28)$$

is suitable, where A is the exposed surface area, V is the member volume, n ranges from 1 to 2, and t is the time of drying. The effect of member shape is secondary and not significant for design purposes.

The ability of large concrete members to retain internal moisture over very long periods has been noted by Tremper and Spellman[8.185] who measured the relative humidity at depths of 50 to 75 mm and 100 to 125 mm from the surface of concrete in eight bridges built between 1929 and 1959 in California. They reported that, despite the arid conditions prevailing at some sites, the average relative humidity measured was 67% at the 75 mm depth, and 70% at the 125 mm depth. In normal laboratory shrinkage testing, only small specimens are likely to come into moisture equilibrium with the surrounding atmosphere, and long-term investigations (in excess of six months) are usually necessary to obtain a reasonable estimate of shrinkage.

Environment and time

The concrete's environment comprises the type and extent of curing and the subsequent drying conditions, ie the ambient relative humidity and temperature. While moist curing delays the onset of shrinkage, the effect of curing on shrinkage of normal strength concrete is small. Prolonged moist curing (greater than one month) seems to somewhat reduce shrinkage of concrete, suggesting that this is one reason why the shrinkage of large structural members in which effective curing time is increased by slow drying is less than that of small members. An opposite effect has been noted for pastes[8.186] presumably since the gel volume is increasing at the expense of unhydrated cement grains which restrain shrinkage. In general however the length of the curing period is not an important factor in normal-strength concrete shrinkage.

The influence of ambient relative humidity and time on shrinkage is shown in Figure 8.31. The rate of shrinkage decreases with time, although it is still measurable after 20 years. The shape of the curves for the various humidities is similar and this allows a curve of common form to be used for shrinkage determinations. In a study conducted by Baroghel-Bouny and Godin[8.187] a linear relationship between shrinkage strains and relative humidity was found for a large range of concrete mixes.

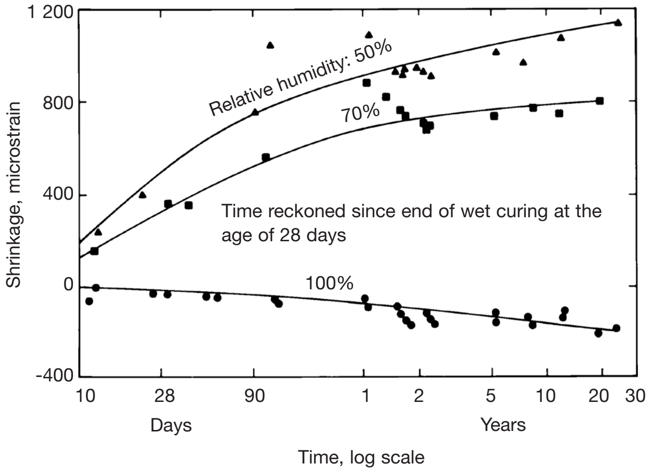


Figure 8.31: Relation between shrinkage and time for concretes stored in different relative humidities[8.95]

The effect of temperature on drying shrinkage will depend on the stage at which increased temperatures are experienced by the concrete, and on the surrounding moisture condition. Hence, while steam and autoclave curing reduce shrinkage, elevated temperatures during drying will increase shrinkage. The effect of normal steam curing is to increase the strength and the elastic modulus of the concrete, and so on drying the concrete exhibits lower shrinkage. The degree of shrinkage reduction depends mainly on the cement content of the mix, leaner mixes exhibiting lower shrinkages than richer mixes, and reductions of between 10 to 30% compared with normal concrete have been measured.[8.188] In the case of steam curing under pressure (autoclave treatment), changes in the morphology and composition of the cement paste occur. This leads to considerable reduction in shrinkage, with various researchers reporting reductions of between 50 to 75% when compared with normal concrete.[8.189, 8.190]

8.5.4 Estimation of shrinkage

Linear shrinkage can be estimated from a knowledge of mix proportions, service environment and member size. The number of variables influencing shrinkage is considerable, and estimation is, at best, imperfect. Assumptions concerning the mix or the environment will frequently have to be made, and the inaccuracy of these assumptions may lead to a significantly different shrinkage being observed in practice. Examples of where design assumptions may be in error are: actual relative humidities and temperatures will fluctuate rather than remain constant; cement and aggregate properties may differ significantly from those assumed; erratic curing may occur leading to moisture losses at times other than those allowed for. The conclusion is that shrinkage values should be viewed as statistical variables, and while a mean expected value may be estimated, actual values may range about the mean. For these reasons, the designer must choose an appropriate level of estimation (see section 8.1.5), and view the results in a critical light.

SANS 10100-1:2000[8.4]

Guidance on the shrinkage strains to be used in the calculation of prestressing losses are set out in SANS 10100-1:2000 *The structural use of concrete*.[8.4] Values are given for pre-tensioning at between three and five days after concreting and for post-tensioning at between seven and 14 days after concreting. Values for South African conditions are given in Table 8.4.

For reinforced concrete design, the minimum information normally required would be a knowledge of the environmental humidity and member size.

BS 8110[8.2] and SANS 10100-1:2000[8.4]

A graphical method for the estimation of shrinkage is given in BS 8110,[8.2] reproduced in SANS 10100-1:2000[8.4] and shown in Figure 8.32. This represents somewhat above a first-level approach, by incorporating a rough guide to development of strain with time. The required information is:

- Exposure condition (as average relative humidity)
- Effective section thickness (ie twice the volume divided by exposed surface area)
- Relative area of reinforcing steel

Table 8.4: Shrinkage of concrete for prestressing operations (from SANS 10100-1:2000)[8.4]

	Shrinkage per unit length, Microstrain		
	Relative humidity		
	80%	60%	35%
System	Coastal Towns	Most inland areas	Environments of unusually low relative humidity such as Windhoek and Upington
Pre-tensioning Transfer at 3 to 5 days after concreting	180	310	420
Post-tensioning Transfer at 7 to 14 days after concreting	140	250	350

Figure 8.32 relates to concrete with an original water content of 8% by mass and made with high-quality dense aggregate. The use of aggregates of lower stiffness (ie those which cause a low elastic modulus of the concrete) or of shrinking aggregates will increase the concrete shrinkage. Shrinkage is broadly proportional to the original water content so that estimates may be adjusted for concretes with a water content other than 8%.

Concrete stored outside under conditions of fluctuating relative humidity will exhibit seasonal, cyclic strains superimposed upon the average strain estimated from the graph shown; the maximum shrinkage strain will occur at the end of each winter for most inland areas and coastal areas other than the Western and Southern Cape coastal belts, where the maximum strain will occur at the end of summer.

The reduction in shrinkage caused by symmetrically placed reinforcing steel can be allowed for by the equation:

Shrinkage of symmetrically reinforced concrete =
$S/(1 + K\rho)$..(28)

where S is the potential shrinkage of the plain concrete, ρ is the area of steel relative to that of concrete, and K is a coefficient = 20 for South African conditions. Potential movements due to concrete shrinkage may be reduced by external or structural restraint. Relative humidities for various centres in South Africa are given in Table 8.3.

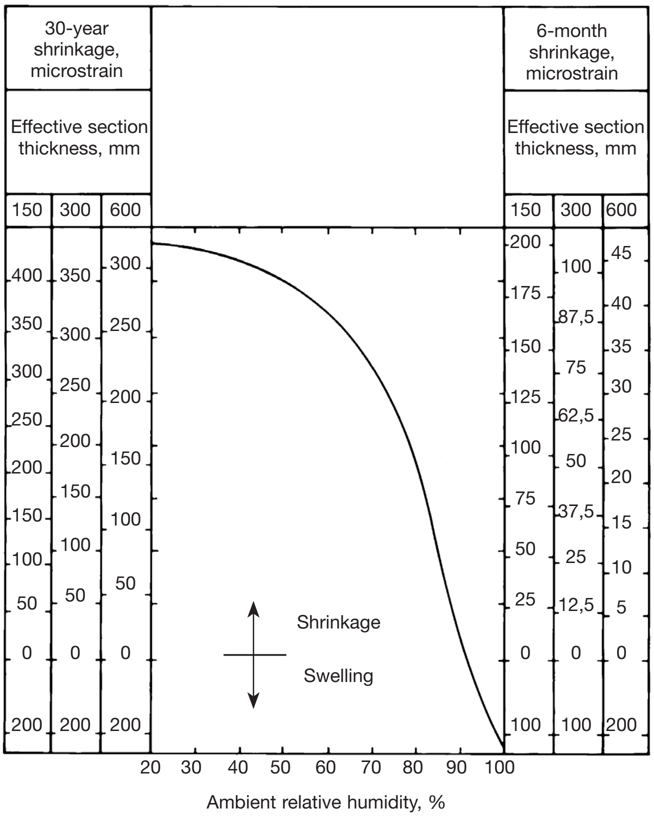


Figure 8.32: Drying shrinkage of normal-density concrete[8.2]

Simplifications are that secondary variables such as temperature, aggregate stiffness, the effects of different cements and admixtures, and the detailed development of shrinkage with drying time are omitted. The main advantage of the method is its simplicity.

CEB-FIP[8.7, 8.99]

The CEB-FIP[8.7, 8.99] have produced a series of documents for shrinkage prediction. They represent empirical methods at a higher level of estimation than BS 8110 and require more initial data, such as relative humidity of storage, member thickness, mix proportions and age of drying

BS EN 1992:2004[8.6]

The EN shrinkage prediction model distinguishes between drying and autogeneous shrinkage components and the total shrinkage strain is calculated by addition of the two components. The final drying shrinkage strain $\varepsilon_{cd,\infty}$ is estimated from

$\varepsilon_{cd,\infty} = k_h \cdot \varepsilon_{cd,0}$... (29)

where $\varepsilon_{cd,0}$ is the nominal unrestrained drying shrinkage value, which can be read from a table in relation to compressive strength and relative humidity of the ambient environment, and k_h is a coefficient that depends on the notional member size. An equation for the expected development of drying shrinkage strain with time is also presented in the document. The autogenous shrinkage strain component ε_{ca} is a function of the characteristic concrete strength f_{ck} and follows from:

$\varepsilon_{ca}(t) = \beta_{as}(t) \cdot \varepsilon_{ca}(\infty)$... (30)

where $\varepsilon_{ca}(\infty) = 2{,}5\,(f_{ck} - 10).10^{-6}$.................................... (31)

and $\beta_{as}(t) = 1 - \exp(-0{,}2t^{0{,}5})$, where t is given in days..(32)

RILEM Model B3[8.8]

RILEM Technical Committee 107-GCS has produced a sophisticated shrinkage model – the so-called RILEM Model B3.[8.8] The model derives from the work of Bazant and co-workers.[8.108-8.117] Factors such as w:c, cement type, cement content and aggregate:cement ratio are included. The complexity of the model does not make it suitable for use at the preliminary mix design stage, and it is therefore mainly recommended for structures which are deflection-sensitive.

In many cases, it is more useful to use algebraic expressions to account for the various factors, rather than graphical estimation (see references 8.3, 8.108 to 8.117, 8.136). While this may give an appearance of accuracy, the final values may in fact be no better than those estimated using simpler approaches.

8.5.5 Other aspects: shrinkage mechanisms and shrinkage tests

The following aspects regarding concrete shrinkage are not dealt with in great detail here. Further reading can be undertaken using the references provided.

Microstructure of hardened cement paste (HCP)

A fundamental understanding of concrete shrinkage is not possible without an appreciation of the microstructure of HCP. This material is the usual source of shrinkage in concrete, and the magnitude and rate of shrinkage is governed by the nature of the calcium silicate hydrate, the pore structure (size and distribution) and the nature and quantity of water associated with the paste.[8.191] Shrinkage is controlled by the different degrees of mobility of water in the paste, and its interaction with the environment.[8.126] Micro-structure of HCP is discussed in Chapter 1.

Shrinkage mechanisms

Several hypothetical mechanisms have been advanced to explain paste shrinkage. Briefly, these are:

Reversible shrinkage mechanisms

Capillary tension

On drying, water is lost from the paste, first from the larger pores such as capillaries and then from the smaller gel pores. Water in partially saturated pores is in a state of capillary tension which must be balanced by compressive stresses in the surrounding gel, causing a volume reduction. The consensus of opinion is that this mechanism is operative at relative humidities above about 40%.[8.127, 8.129, 8.130] Below this value, other mechanisms operate.

Surface tension

A solid surface is subjected to surface tension forces which act tangentially to the surface and have the effect of inducing compressive stresses inside the material. For colloidal-size gel particles, these stresses can be very large. Adsorption of water onto these surfaces reduces the surface tension forces, while drying has the opposite effect. This mechanism will be valid only at humidities lower than about 40% where variations in the water content of the paste are mainly due to variations in the amount of adsorbed water.[8.132, 8.192]

Swelling (disjoining) pressure

The thickness of the adsorbed water layer on gel particles depends on the relative humidity. If the distance separating two particles is less than twice the free-water layer thickness, an area of "hindered" adsorption will occur. This area will result in a swelling or disjoining pressure and corresponding deformation being developed. As the relative humidity decreases, the free-water layer thickness will reduce causing a corresponding reduction in disjoining pressure. Wittmann[8.193, 8.194] considers this to be important above 50% relative humidity.

Movement of interlayer water

According to the Feldman and Sereda model,[8.130] movement of water in and out of the layered hydrate particles will cause volume changes by affecting the spacing of the layers. This mechanism is considered to be effective below about 35% relative humidity.

Irreversible shrinkage mechanisms

- Reduction of particle spacing with time leads to a decrease in the total energy of the system (Ishai).[8.129]
- Gel surfaces approaching each other for the first time form new bonds on first drying and reorientation of the hydrate layers (Feldman and Sereda).[8.130]

Other non-load-induced volume changes

Changes due to cement solution, water absorption and adsorption

These volume changes occur while the concrete is still sufficiently plastic to accommodate such changes without stress, and are therefore of little practical importance.

Changes due to bleeding and plastic shrinkage

Since these occur while concrete is in the plastic state, they will not be covered here. However, they may be very important, and Chapters 6 and 12 should be consulted.

Autogenous volume changes due to cement hydration

Autogenous shrinkage, also called basic shrinkage, is a volume reduction caused by cement hydration and the internal consumption of water. It explains the shrinkage of concrete in sealed conditions, without moisture exchange between concrete and environment. The formation of hydration products that have lesser volumes than the sum of the volumes of the original components causes the cement-plus-water system to contract as hydration proceeds. In addition, the internal consumption of water during hydration, called self-desiccation, can lead to autogenous shrinkage. For normal strength concrete, autogenous volume changes are usually small compared with carbonation and drying shrinkage. However, concrete with a w:c of less than 0,4 has a much higher internal consumption of mixing water and therefore a high autogenous shrinkage component. Baroghel-Bouny and Godin[8.187]measured autogenous shrinkage for a large set of portland cement (PC) pastes and concluded that the magnitude of autogenous shrinkage increases linearly with decreasing w:c ratio. Recent developments in high performance concrete have shown that autogenous shrinkage in such materials cannot be modelled accurately with older prediction models which were designed for normal strength concretes.[8.195, 8.196] The new European standard BS EN 1992:2004[8.6] therefore incorporates an autogenous shrinkage component in its shrinkage prediction models, as discussed in section 8.5.4.

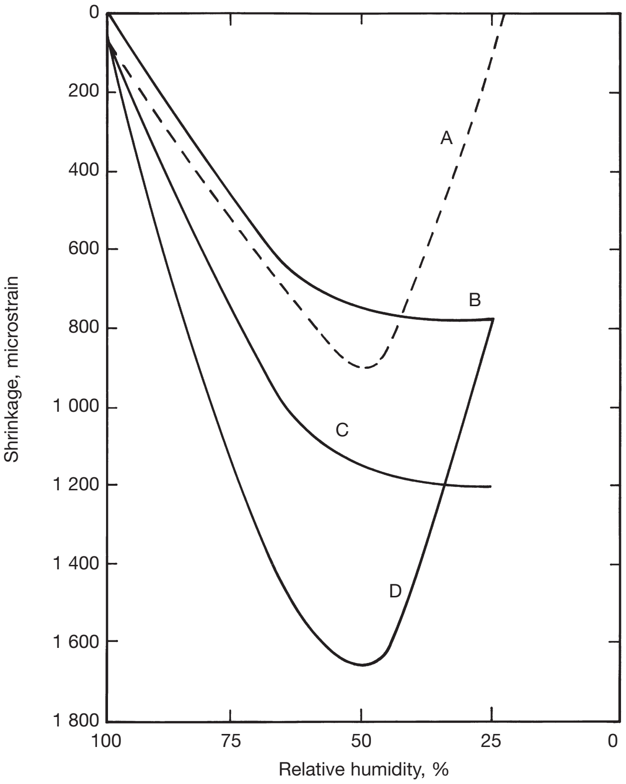
Approximately 40% of autogenous shrinkage occurs within the first 24 hours, during which the tensile strain capacity is low, resulting in a high risk of early cracking. Being an internal process, autogenous shrinkage cannot be controlled by curing procedures, although active moist curing can mitigate the effects. It develops rapidly after setting and tends to reach its limiting value after approximately 10 weeks.[8.197] Pane and Hansen[8.198] reported lower autogenous shrinkage in concretes made with FA and GGBS, compared to PC concrete, while the use of CSF resulted in a slight increase in autogenous shrinkage.

Carbonation shrinkage

The reaction between the carbon dioxide of the atmosphere and the constituents of hardened cement paste results in a reduction in volume called carbonation shrinkage, which occurs over a long period and may in some cases exceed the drying shrinkage in magnitude. Figure 8.33 shows that carbonation shrinkage is a function of relative humidity and is greatest at intermediate humidities. Carbonation is reduced at either higher or lower humidities, in the former case since carbon dioxide cannot penetrate the water-filled pore spaces easily, and in the latter case due to an absence of water films (water is needed in the chemical reaction). In general, carbonation shrinkage is greatest when carbonation occurs subsequent to drying.

Some practical implications of carbonation shrinkage are:

- Carbonation under normal conditions is slow, but over a long period it may cause irreversible shrinkage, additional to drying shrinkage, which can have serious effects. Warping of structural components, and crazing of concrete surfaces have been attributed to this cause. This is particularly true for concretes exposed to atmospheres of intermediate relative humidities[8.200, 8.201] and possessing small cross-sectional dimensions.
- Walls of concrete masonry may show severe cracking as a result of carbonation, accelerated by the relatively porous type of concrete used for block manufacture.[8.201] For this reason manufacturers of concrete units sometimes use carbon dioxide curing to accelerate carbonation shrinkage. The effect not only reduces the shrinkage of the units after building in, but also reduces volume changes due to subsequent wetting and drying.
- An understanding of the underlying mechanisms is basic to the interpretation of shrinkage tests to determine the drying shrinkage of concretes, since these are seldom carried out in CO_2-free atmospheres. Frequently such tests are carried out by drying small specimens in air to equilibrium at 50% relative humidity. In such circumstances as much as one third of the observed volume change may be attributable to carbonation and not to drying shrinkage. Consequently any theories relating to drying shrinkage built on such observations must be viewed with caution.



A: Carbonation shrinkage at humidity shown
B: Shrinkage on drying to humidity shown
C: Simultaneous carbonation and drying to humidity shown
D: Drying to humidity shown followed by carbonation at that humidity

Figure 8.33: The effect of relative humidity on drying and carbonation shrinkage of mortars[8.199]

8.5.6 Tests for shrinkage

Shrinkage tests involve measuring the linear (one-dimensional) contraction of a prismatic specimen with time. The specimen is usually relatively long in the direction orthogonal to the moisture movement and therefore the potential free shrinkage of the cross-section (represented by the moisture gradient) is not realised due to the continuity of the section. This implies that plane sections remain plane, and therefore what is actually measured is a composite strain comprising shrinkage, elastic and creep strains. Self-equilibrating stresses in the section cause compression at the centre, and tension on the surfaces. L'Hermite[8.182] also showed that axial versus surface shrinkage measurements could vary quite substantially depending on specimen size.

Some standard tests for shrinkage involve accelerated drying regimes, usually in an oven or an environment of low relative humidity. Examples are:

- BS EN 1367-4:1998[8.202] 105°C; the test is used to classify the drying shrinkage of aggregates in concrete
- BS 1881-209:1990:[8.35]50 to 55°C; relatively moist air
- SANS Method 5836:[8.203] 50 to 55°C, 15-25% RH; for classifying the drying shrinkage of aggregates in concrete

- SANS Method 6085:[8.204] 50-55ºC, 15-25% RH; for measuring the shrinkage of concrete

Other tests employ a milder environment typical of laboratory conditions, for example:

- ASTM C 157-06:[8.205] 23°C; 50% RH
- RILEM: CPC 9 (1975):[8.206] 20°C; 50% RH

Goodman,[8.207] in a series of local comparative tests between accelerated and long-term methods, concluded that the criterion for terminating the SANS test is arbitrary and therefore it is not possible to predict long-term shrinkages from results of this test.

For further reading on concrete shrinkage, refer to the articles recommended in section 8.4.8.

8.6 Creep and shrinkage of high-strength concrete (HSC)

High-strength concrete (HSC) will be defined here as concrete for which the w:c ratio is less than 0,4.

Shrinkage and creep behaviour of HSC is significantly different from that of normal-strength concrete (NSC).[8.197] HSC has higher basic (ie sealed) creep and shrinkage due to:

- A "self-sealing" effect whereby the pore structure of the concrete densifies rapidly.
- A far larger component of internal consumption of water, thus leaving a smaller proportion of water for movement within the concrete. This internal consumption of water results in a high autogenous shrinkage (or basic shrinkage) component.
- Lower initial water contents due to high dosage of superplasticisers.

8.6.1 Creep of HSC

Compared with NSC, HSC has lower creep.[8.30] The major portion of creep is basic creep, and drying creep is very low for HSC with silica fume.[8.29] However, for loading at early ages (≤ three days), creep tests by Dilger et al on the mixes detailed in section 8.6.2 below, showed that the basic (ie sealed) creep was high (30 to 40 x 10^{-6}/MPa) and developed very fast (within two to three weeks).[8.197] For later loading ages (28 days), the basic (and total) creep was substantially reduced as shown in Figure 8.34. Drying creep also substantially reduced with later loading ages, and in general was about one third of the total creep.

Work by Addis[8.31] on South African high-strength concretes has shown that drying creep reduces substantially with decreasing w:c (see Figure 8.35). Below w:c = 0,33, how-ever, creep is virtually constant. Addis ascribes this to the fact that in this range paste porosity is virtually constant.

8.6.2 Shrinkage of HSC

Dilger et al[8.197] carried out shrinkage tests on two HSCs incorporating CSF: an 80 MPa mix (w:c = 0,31) and a 100 MPa mix (w:c = 0,21). The results, shown in Figure 8.36, indicate that autogenous shrinkage under sealed conditions is very high, approaching 500 x 10^{-6}, and develops rapidly after final set. Approximately 40% of this shrinkage occurs within the first 24 hours, during which time the tensile strain capacity of the concrete is low. As a result, the risk of cracking due to autogenous shrinkage is high at this stage.

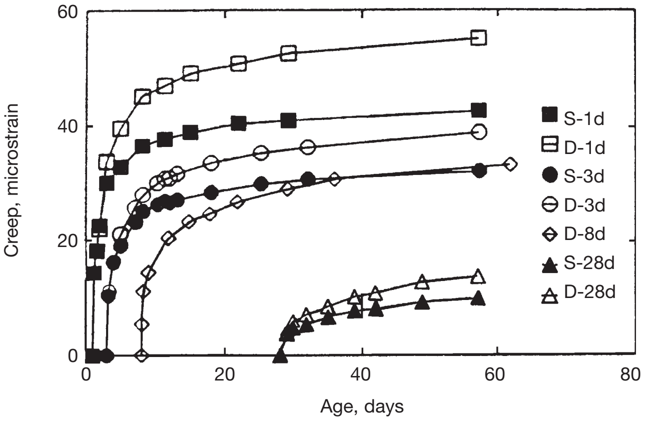


Figure 8.34: Creep of 100 MPa mix loaded at different ages under sealed (S) and unsealed (D) conditions[8.197]

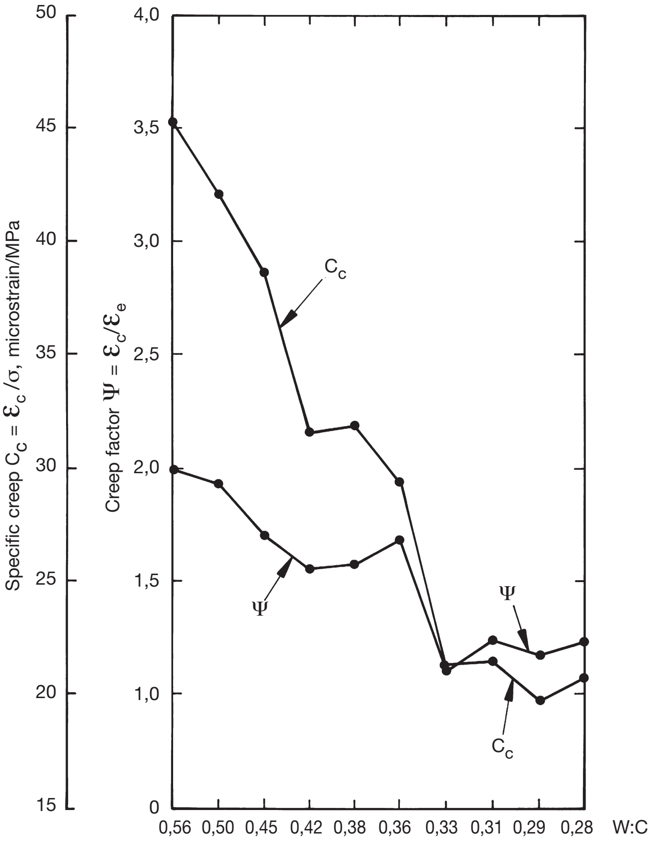


Figure 8.35: Specific creep and creep factor versus water:cement ratio for concretes made with andesite aggregates[8.31]

Measured shrinkage of concrete under drying conditions includes the autogenous (basic) and drying components. In NSC which experiences little autogenous shrinkage, the total measured shrinkage has often been taken to be the drying shrinkage. Strictly, drying shrinkage is the additional shrinkage caused by moisture loss from the concrete, and for HSC, the drying component is of the same order as the autogenous shrinkage. However, moist curing for up to seven days can substantially reduce the drying shrinkage component. Test results show that total shrinkage after two years at 50% RH is in the range 800 x 10^{-6} when drying was started at one to seven days.[8.29]

Drying conditions also have a large influence on total shrinkage, as shown in Figure 8.37. Even under full moist curing, the high early autogenous shrinkage results in overall shrinkage of the specimen, implying that in thick HSC members, early shrinkage will occur even under wet curing.

Addis[8.31] investigated drying shrinkage of high-strength concretes (cube strength > 60 MPa) made with South African materials. He found that at low w:c (< 0,35), shrinkage reduced considerably, even though water content in this range remained constant (see Figure 8.38). He attributed this primarily to reduced permeability of the high-strength mixes. Other investigators[8.208, 8.209] found little difference between shrinkage of normal and high-strength concretes, except that shrinkage of HSC occurs at a more rapid rate than shrinkage of normal-strength concrete. However, as pointed out above, shrinkage measurements that exclude early autogenous shrinkage in HSC will underestimate the true shrinkage.

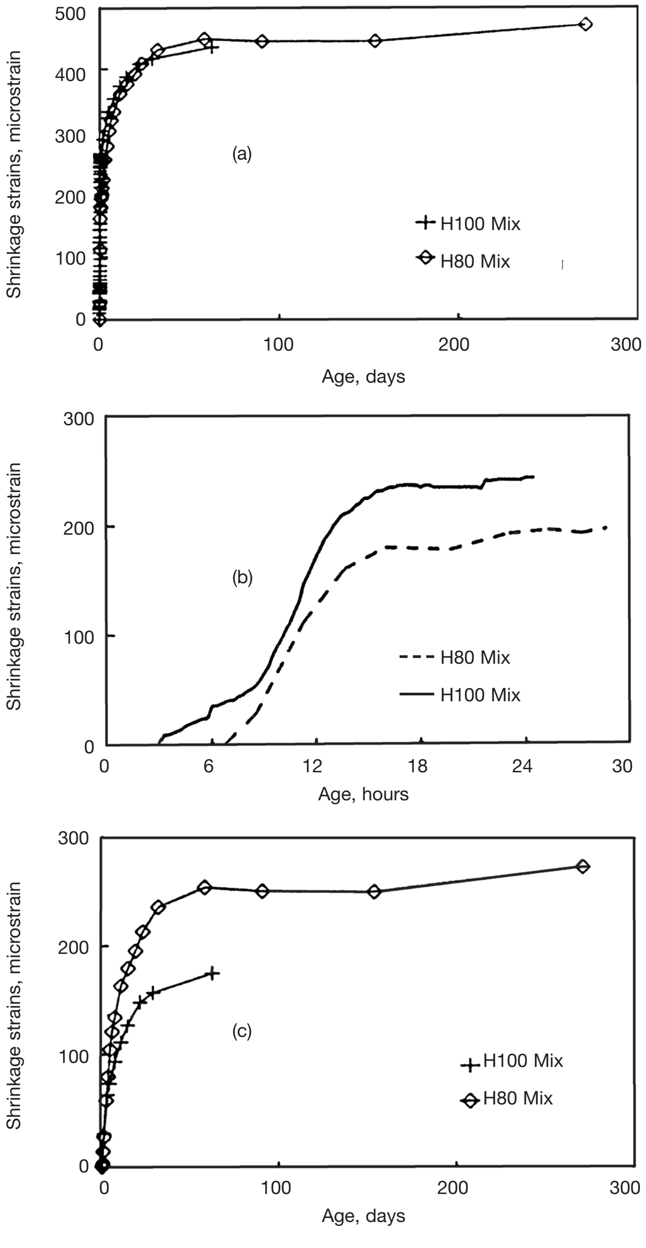


Figure 8.36: Development of autogenous shrinkage: (a) total shrinkage (b) during the first 24 hours (c) after the first 24 hours[8.197]

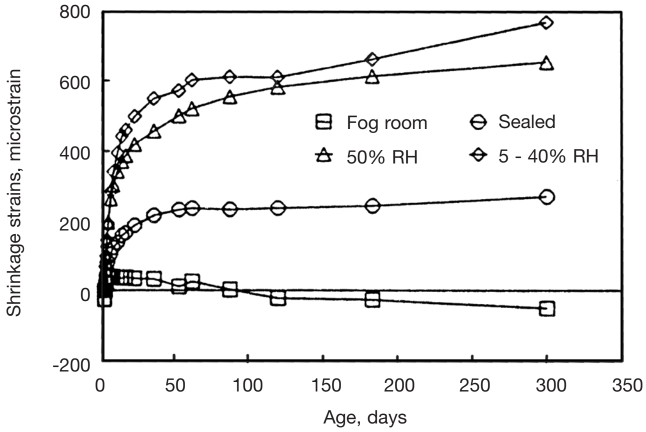


Figure 8.37: Shrinkage of 80 MPa mix in different ambient conditions, after 24 hours in mould curing[8.197]

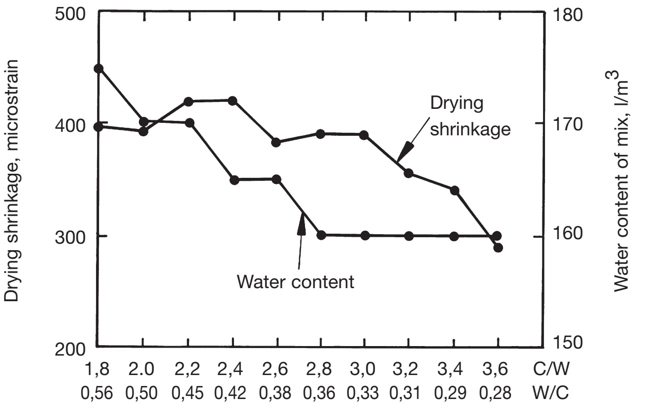


Figure 8.38: Drying shrinkage (at 180 d) and water content of mix versus W/C ratio for concretes made with andesite aggregate[8.31]

8.6.3 Creep and shrinkage predictions for HSC

Due to differences in shrinkage and creep behaviour of HSC, predictive approaches developed for NSC become highly inaccurate. For HSC exposed at very early ages (< 24 hours) to drying, CEB-FIP Model Code 90[8.7] severely underestimates shrinkage, due to the autogenous (basic) shrinkage. Comparison with test data shows that this code consistently underestimates HSC shrinkage by about 50%. On the other hand, the ACI Code[8.3] tended to overestimate shrinkage when drying and measurements were started at three days. The accuracy of creep predictions depends on the age of loading of the concrete. For later ages of loading (>14 days) it will generally be found that current prediction approaches overestimate creep deformation of HSC (since HSC has a lower specific creep than NSC). For very early ages (<24 hours), current predictions approaches may seriously underestimate creep (due to creep components linked to on-going early hydration reactions, and a relatively open pore structure at very early ages).

In view of these inadequacies, new creep and shrinkage prediction models have been proposed for HSC, which are superior to most of the current codes.[8.210, 8.211] The European standard BS EN 1992:2004[8.6] incorporates an autogenous shrinkage component in its shrinkage prediction models, as discussed in section 8.5.4.

8.7 Thermal movement

Most materials exhibit dimensional changes with changes in temperature. This section concerns the effect of temperature variations on the expansion and contraction of concrete. Thermal movement should be considered in conjunction with other deformations of concrete and does, in some cases, dominate the other dimensional changes. The importance of thermal properties of concrete depends on the type of structure and on the degree of exposure to which it is subjected. Thermal movements are of critical importance, for instance, in casting large masses of concrete in dam construction or in massive structural blocks of high-strength concrete where internal temperature rises of 45°C or more can be expected in the concrete. The advent of nuclear reactors has greatly increased research into thermal behaviour of concrete at very high temperatures. At the opposite extreme, South Africa has several major cryogenic storage facilities in which concrete behaviour at temperatures well below minus 100ºC is of special importance.

For the more normal applications of concrete the major factors influencing thermal movement are: firstly, the type of aggregate; secondly, the moisture content of the concrete; and thirdly, the volume concentration of aggregate in the mix. The effects of other factors such as type of cement, strength and age of concrete and curing methods are far less significant.

The importance of moisture content should be mentioned before discussing the effect of aggregates. Partially moist concretes have coefficients of expansion higher than those in the completely dry or saturated state and the cause of this lies in the thermal behaviour of cement paste which has its maximum coefficient at some intermediate moisture content, and lower coefficients in both the saturated and oven-dry conditions. Some failures of concrete have been attributed to continued cycles of temperature changes and wetting and drying causing internal stresses between aggregate and paste. On the other hand a great many examples can be cited of concretes successfully resisting severe exposures to such cyclic variations. It is possible that porosity of aggregates, creep and other internal mechanisms help to relieve the stresses.

8.7.1 Effect of aggregate on thermal movement

The thermal coefficient of expansion of normal weight aggregate ranges widely from about 5 to 15 microstrain/ºC. The coefficient for siliceous minerals such as quartz (about 12 microstrain/ºC) is higher than that for most other minerals, so that expansion coefficients of rocks increase with their silica content. In general, aggregates with high quartz contents have the highest coefficients while calcareous aggregates have the lowest. The coefficients of thermal expansion of some South African and other aggregates are given in Table 8.5. It will be seen that andesite and dolerite are in the region of 6 to 7,5 microstrain/ºC while quartzite ranges from 8,5 to 12,5 microstrain/ºC.

As would be expected, the thermal movement of concrete is strongly influenced by that of the aggregate from which it is made, so that quartzite concretes generally exhibit the highest coefficients of expansion. As mentioned earlier, however, the volume concentration of aggregate in the mix also plays some part, albeit a minor one, and this is shown in Figure 8.39.

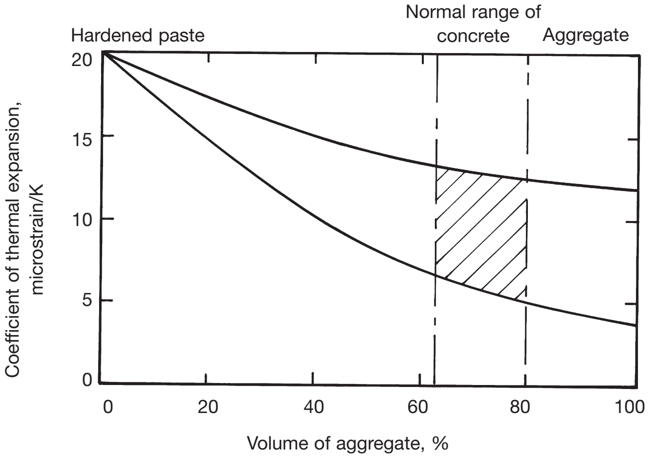


Figure 8.39: Thermal expansion of concrete having different aggregate types and contents[8.212]

Table 8.6 shows data for coefficients of linear thermal expansion of concrete, where it can be seen that aggregates play a crucial role.

Table 8.5: Range of coefficients of thermal expansion of rocks

Type of rock	Average coefficient of thermal expansion, microstrain/°C	
	Temperatures up to 65°C Reference 8.213	Temperatures up to 45°C Reference 8.11
Quartzite, silica shale, chert	11,0 - 12,5	8,5 - 12,5
Reef quartzite	-	9,5 - 12,0
Sandstone	10,5 - 12,0	-
Quartz sand and pebbles	10,0 - 12,5	-
Clay and mica shale	9,5 - 11,0	-
Granites and gneiss	6,5 - 8,5	6,5 - 9,5
Syenite, feldspathic porphyry, diorite, andesite, phonolite, gabbro, diabase, basalt	5,5 - 8,0	-
Andesite	-	7,5
Dolerite	-	6,0 - 7,0
Limestone	3,5 - 6,0	-
Pure calcite	4,0 - 6,5	-
Marble	4,0 - 7,0	-
Dolomite, magnesite	7,0 - 10,0	7,5 - 9,0
Greywacke	-	11,0
Tillite	-	6,5
Felsite	-	7,0

Table 8.6: Effect of aggregate type on coefficient of thermal expansion of concrete

Aggregate	Coefficient of thermal expansion of concrete, microstrain/°C	Reference
Granite and rhyolite	6,8 - 9,5	8.214
South African granite	8,0 - 9,5	8.19
Sandstone	11,7	8.214
Quartzite	12,8	8.214
South African quartzite	9,5 - 11,5	8.19
Limestone	6,1 - 9,9	8.213, 8.214
Marble	4,1	8.213
Dolerite	9,5	8.214
South African dolerite	7,5 - 8,0	8.19
Quartz	10,4	8.213
Blastfurnace slag	10,6	8.214
Reef quartzite	9,5 - 12,0	8.15, 8.19
Granite from north of Johannesburg (Jukskei)	8,0 - 9,5	8.15, 8.19
Dolomite from Olifantsfontein	8,5 - 9,0	8.15, 8.19
Malmesbury hornfels from Cape Peninsula	10,0 - 11,0	8.15, 8.19
Limestone (50/50 by mass) Lichtenburg/Ulco	9,5	8.15
Namaqualand onyx	10,5	8.15
Dolerite from the Orange-Fish Tunnel	7,5	8.15
Felsite from Witbank	9,0	8.15, 8.19

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Chapter 9

Durability of concrete

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9.1 Introduction

The oldest concrete structures, made with pozzolanic binders, date back to Roman times,[9.1] attesting to their endurance. There are many other examples of concrete structures from the previous century and earlier that have performed successfully over many decades or longer, underscoring the inherent durability of concrete. A common feature of these early concrete structures is that they either incorporated pozzolanic binders, as in the case of the Roman structures, or were made with cements and mix proportions that gave them inherent durability.

It is this perception of inherent durability that has given rise to the still generally accepted view among many structural engineers that durability of concrete is not really a major problem. Indeed in many cases this is correct, but engineers now need to take a more sophisticated view of concrete durability, and design for adequate service life. Examination of newer structures made from modern portland-based cements reveals that even the best of concretes is not immune to weathering and corrosion from exposure to natural environments. This can affect the concrete as a material directly, as in the case of soft water or sulphate attack, or it can affect embedded reinforcing steel in reinforced concrete members, where corrosion is a major issue in modern as well as aging structures.

In the context of structural concrete, durability can be defined as "the ability of a structure or component to withstand the design environment over the design life, without undue loss of serviceability or need for major repair".[9.2] Durability therefore relates to the concept of material performance, and cannot be regarded as an intrinsic material property. A concrete that is durable in one environment may not be durable in another. Durability is thus associated with the deterioration of the material over the intended service life of the structure in a given environment.

This concept is demonstrated in Figure 9.1, which shows the progress of deterioration over time of two structures, A and B. Structure A deteriorates at such a rate that it reaches the level of minimum required quality after its expected service life and is, therefore, of acceptable durability. On the contrary, structure B displays inadequate durability and deteriorates more rapidly, requiring unplanned rehabilitation during its expected service life. This deterioration may be unexpected, which illustrates the need to be able to accurately predict the likely performance of concrete structures during their service lives. This is becoming a requirement for modern concrete design. The increasing frequency of inadequate durability and the associated high costs of repair are leading owners of structures to demand that designers and contractors provide assurance of a pre-defined, repair-free service life of concrete structures.

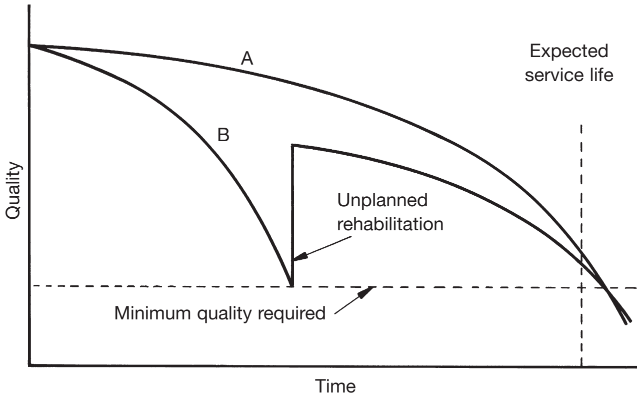


Figure 9.1: Two scenarios of deterioration and rehabilitation of a structure

There is a growing movement in modern concrete practice away from prescriptive approaches and towards performance approaches to durability. This implies that designers will need to consider what acceptable performance of a structure is likely to be in a given environment, and then specify and design according to a set of performance criteria. The constructor's responsibility will be to ensure that the structure is built in a manner consistent with the designer's intentions, and to show proof that the as-built structure in fact meets durability design criteria. To do this, the following aspects must be considered during design and construction:

- A proper understanding of deterioration mechanisms on the part of designers
- Practical Service Life Models that predict with sufficient accuracy the likely durability performance of the structure over its design life
- The means of characterising the potential durability of the concrete to be used in the structure (ie an assessment of the quality of materials and the nature of their proportions)
- The means of assessing the actual quality of the construction, ie the as-built structure, so as to give assurance that the structure has been built to the specification

The elements given above are all part of a performance approach to design, specification, and construction. Merely using concrete strength to "guarantee" durability, as has mostly been the case in the past, is not adequate. Modern developments in durability design and specification take the above aspects into account, as discussed in sections 9.6 and 9.7.

9.2 Factors affecting concrete durability

Concrete durability must be seen as the interaction between concrete as a system and its environment, and both need to be considered in durability assessments. Factors associated with the concrete system influence the ability of the concrete to resist deterioration, while environmental factors influence the degree of aggressiveness that the concrete has to withstand. Using these two categories, the factors that affect durability are shown in Figure 9.2.

Figure 9.2 shows a number of influencing factors, not all of which may be important at all times. The factors most likely to dominate in any given durability assessment are binder type and water:binder ratio (for intrinsic factors), and curing and early age temperature history (for extrinsic factors). The most critical environmental factor is the aggressiveness of agents attacking the concrete. The effects of cracking can speed up environmental deterioration as cracks may accelerate ingress of harmful substances into the concrete.

This chapter takes the approach that, as with other engineering properties such as strength and stiffness, concrete needs to be designed to achieve a particular durability. Such "durability design" must encompass appropriate structural design (eg attention to details such as drainage), materials selection, deterioration modelling, and construction practice. Section 9.4 examines the various influences to which concrete structures are subjected and explains the mechanisms that operate to reduce or destroy the integrity of such structures. Practical guidelines are presented for making durable concrete and the chapter shows how to utilise existing specifications and codes of practice to best advantage in order to maximise durability. This chapter also shows the steps that can be taken to minimise the rate at which deterioration occurs and discusses what additional measures can be implemented to enhance durability.

9.3 Concrete properties in relation to durability

9.3.1 Transport properties

Transport properties of cementitious materials are key to predicting their durability, since deterioration mechanisms such as chemical attack, leaching, chloride ingress or carbonation all relate to the ease with which a fluid or ion can move through the concrete microstructure. The passage of potentially aggressive species (ions or molecules in the form of liquids and gases) is primarily influenced by the penetrability of the concrete. Penetrability is broadly defined as the degree to which the concrete permits gases, liquids, or ionic species to move through its pore structure. It embraces the concepts of permeation, sorption, diffusion and migration and is quantified in terms of the transport parameters.[9.3]

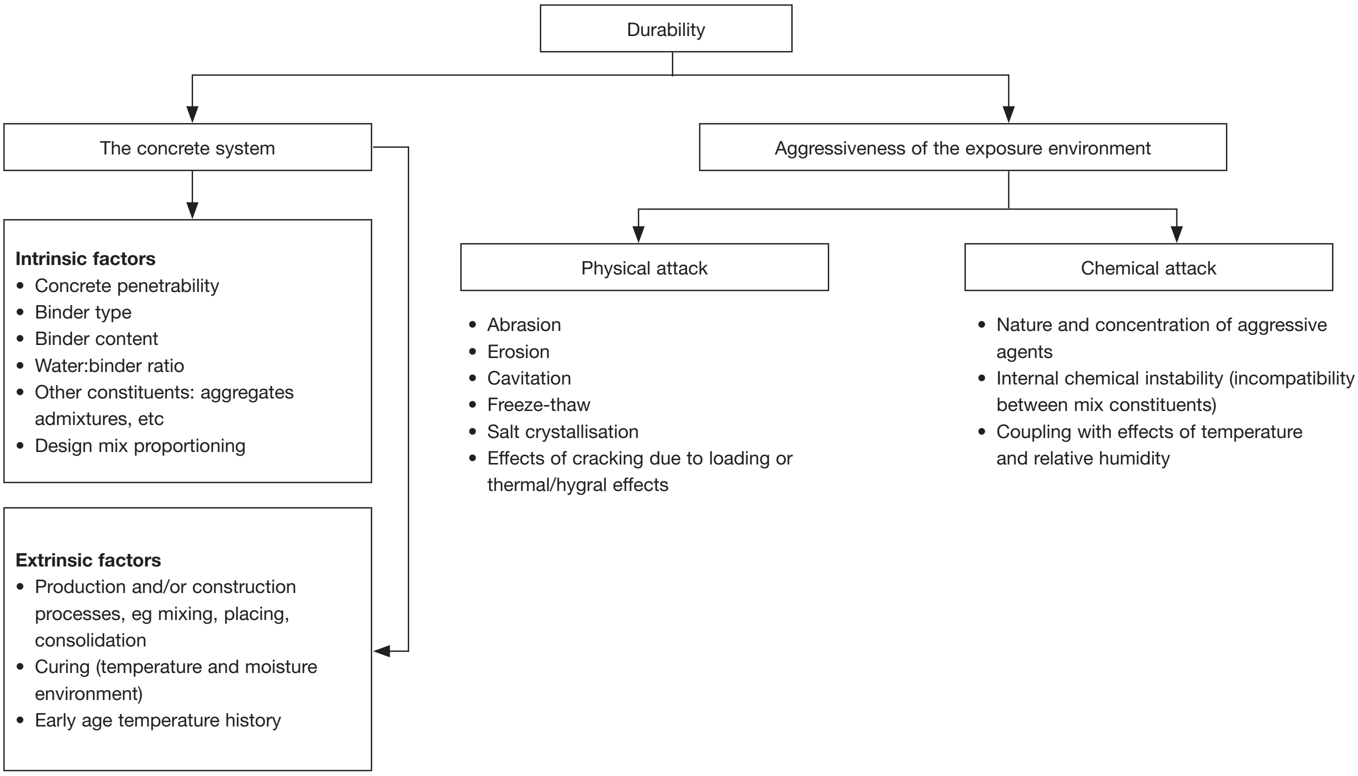


Figure 9.2: Concrete and environment: factors influencing the durability of concrete

The processes involved in fluid and ion movement include the distinct mechanisms of capillary action, flow under pressure and flow under a concentration gradient. These mechanisms are characterised by the material properties of sorptivity, permeability and diffusivity respectively.[9.4] The permeability of concrete will predominantly be influenced by the permeability of the cement paste, especially at the interface with aggregate particles. Transport mechanisms such as permeation, absorption, diffusion and migration, as well as the influence of the interfacial transition zone (ITZ) between aggregate and cement paste, are important for concrete deterioration processes. Some general considerations are discussed below.

Permeation

Permeation describes the process of movement of fluids through the pore structure under an externally applied pressure while the pores are saturated with the particular fluid. Permeability is therefore a measure of the capacity for concrete to transfer fluids by permeation. The permeability of concrete is dependent on the concrete microstructure, the moisture condition of the material and the characteristics of the permeating fluid.

Permeation plays an important role in water-retaining structures where water transport through the structure is detrimental. Permeability characteristics may also be used in predicting carbonation of concrete, as discussed in section 9.7.2.

Absorption

Absorption is the process whereby fluid is drawn into a porous, unsaturated material under the action of capillary forces. The capillary suction is dependent on the pore geometry and the degree of saturation of concrete. Water absorption caused by wetting and drying at the concrete surface is an important transport mechanism near the surface but becomes less significant with depth. The rate of movement of a wetting front through a porous material under the action of capillary forces is defined as sorptivity.

Sorptivity is influenced by the larger capillaries and their degree of interconnection, and is very sensitive to hydration of the outer concrete surface, and hence curing. It is also influenced by compaction and aggregate orientation and distribution, and by mix composition.[9.5]

Diffusion

Diffusion is the process by which liquid, gas or ions move through a porous material under the action of a concentration gradient. Diffusion occurs in partially or fully saturated concrete and is an important internal transport mechanism for most concrete structures exposed to salts. High surface salt concentrations are initially developed by absorption, and the salt migrates by diffusion towards the low concentrations of the internal material. Diffusion rates are dependent on temperature, moisture content of concrete, type of diffusant and the inherent diffusibility of the material. Diffusion into concrete is complicated by chemical interactions with the cement hydration products, and by partially saturated conditions, defects such as cracks and voids, and electrochemical effects due to steel corrosion and stray currents. In the marine environment, diffusion of chloride ions is of particular importance due to the depassivating effect of chlorides on embedded steel, which ultimately may lead to corrosion.

The modelling of gaseous and ionic diffusion in concrete is commonly done using Fick's first law of diffusion (for steady state diffusion). This law may be used to describe the rate of diffusion of a fluid or ion into a uniformly permeable material:[9.4]

$$J = -D_{eff} \frac{dC}{dx} \quad \ldots\ldots (1)$$

where: J = mass transport rate (g/m^{2}s)
D_{eff} = effective diffusion coefficient (m^2/s)
dC/dx = concentration gradient (g/m^3/m)
C = concentration of fluid (g/m^3)
x = distance (m)

The negative prefix denotes that the flux occurs along a negative concentration gradient. The modelling of ionic diffusion in concrete is commonly done using Fick's second law of diffusion:[9.4]

$$\frac{\partial C}{\partial t} = D \frac{\partial^2 C}{\partial x^2} \quad \ldots\ldots (2)$$

where: D = Diffusion coefficient (m^2/s)
t = time parameter (s)

C and x are the same as in equation 1. The boundary conditions for equation 2 are as follows:

$C_x = 0$ at $t = 0$ and $0 < x < \infty$
$C_x = C_s$ at $x = 0$ and $0 < t < \infty$

where: C_x = chloride concentration at depth x at time t (g/m^3)
C_s = surface chloride concentration (s/m^3)

Crank's error function solution (also called Gauss error function) of Fick's second law can be stated as:[9.4]

$$C_{x,t} = C_s \left[1 - erf\left(\frac{x}{2\sqrt{D_a t}} \right) \right] \quad \ldots\ldots (3)$$

where:
$C_{x,t}$ = chloride concentration at depth x at a given time t
C_s = surface chloride concentration (g/m^3)

D_a = apparent chloride diffusion coefficient (m^2/s)
t = time of exposure (s)
erf = mathematical error function

Chloride concentrations are most commonly expressed as percentage chlorides by mass of cement.

Diffusion is the dominant mechanism for concrete structures fully submerged in sea water or damp salt-contaminated soil. In combination with other mechanisms, diffusion contributes to chloride transport in concrete under most exposure conditions. The transport of oxygen in concrete to the steel surface is also governed mainly by a diffusion process.

Diffusion characteristics are especially important in the service life modelling of reinforced concrete structures exposed to the marine environment. However, practical observations have shown that the application of Fick's second law of diffusion for the prediction of chloride penetration into concrete requires careful consideration in application. Close to the surface, the concrete is exposed to a continuous cycle of wetting and subsequent evaporation (drying). Here the water carrying dissolved chlorides moves in and out, which prevents the development of clear diffusion mechanisms[9.6] (see Figures 9.3 and 9.4). The depth in which this occurs is termed the convection zone. Beyond the convection zone, diffusion is considered the decisive transport mechanism.

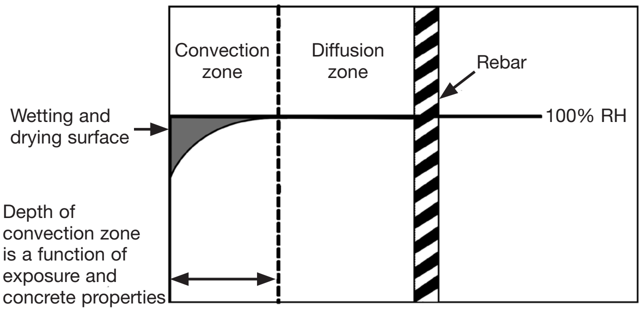


Figure 9.3: Convection zone in concrete [9.7]

Figure 9.4 shows a typical chloride profile in concrete exposed to the marine splash zone.[9.6] Based on measured chloride profile and chloride surface concentration in a concrete member, the diffusion coefficient can be estimated using Fick's second law of diffusion. Applying the reverse relationship in durability predictions, the time-dependent chloride profile in concrete can be predicted when diffusion coefficient and expected chloride surface concentration are known.

Migration

Migration (also referred to as accelerated diffusion, electro-diffusion, or conduction) is the movement of ions in a solution under an electrical field. It is the transport mechanism most often used in laboratory accelerated chloride tests and is described by the Nernst-Planck equation. Details may be found in reference 9.8.

Combined transport processes

The selection in isolation of a single transport mechanism for the ingress of a particular substance may represent an over-simplification of the real transport process since more than one transport mechanism may be active at a given time, either in parallel or in different sections along the flow paths.[9.5, 9.9]

Effect of concrete aging on transport mechanisms

Within its service life, the transport properties of concrete are most likely to be modified. This is mainly due to the ongoing hydration of the cementitious material, but may also be due to concrete deterioration, eg cracking or leaching. As a result, the porosity as well as the continuity of the capillary porosity may change with time. Other processes such as chloride binding also continue during this time.[9.10, 9.11] This is of considerable importance when transport parameters are assessed in laboratory experiments on young concrete specimens to provide input data for a long term assessment of structures, eg service life prediction.

The time-dependent diffusion coefficient (D(t)) takes into account the decrease in chloride diffusivity with time due

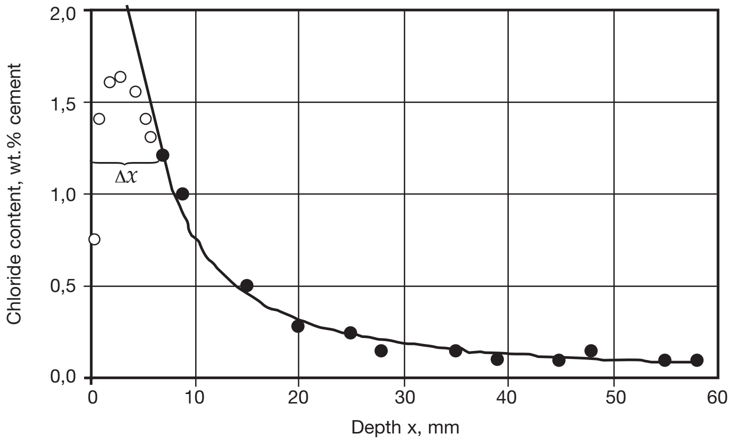


Figure 9.4: Typical measured chloride profile for concrete situated in the marine splash zone (the profile has been fitted to Fick's second law of diffusion error function neglecting data points in the convection zone Δx)[9.6]

to effects such as continued hydration and chloride binding. This is described by the relation:[9.10]

$$D(t) = D_o\left(\frac{t_o}{t}\right)^n \text{..}(4)$$

where D_o is the diffusion coefficient at reference time to (eg 28 days) and n is the aging coefficient (reduction factor). The value of the aging factor depends on the rate of hydration and is critically dependent on type of binder.

Penetrability and percolation

As mentioned earlier, the term penetrability encompasses the concepts of permeation, sorption, diffusion and conduction. On the other hand, percolation refers to whether a material is penetrable or not; thus, percolation is a "pass/fail" criterion.[9.3]

All concrete matrixes are inherently porous and penetrable. Even very dense, hydrated matrixes show penetrability due to the internal pore system and microcracking. Well-hydrated cement paste however has very low permeability (eg 10^{-12} to 10^{-14} m/s), similar to many dense sound rocks used as aggregates. The nature of concrete as a composite material, comprising a paste (or matrix) phase with an aggregate phase, results in the formation of a further phase - the interfacial transition zone (ITZ). This phase, typically 20 to 50 µm thick, generally has higher porosity and penetrability than the bulk paste, providing preferential paths for fluid transport. Sometimes, the ITZ is not clearly evident as a distinct phase, such as in dense heavily compacted dry mixes, in CSF concretes, or in some very mature concretes.

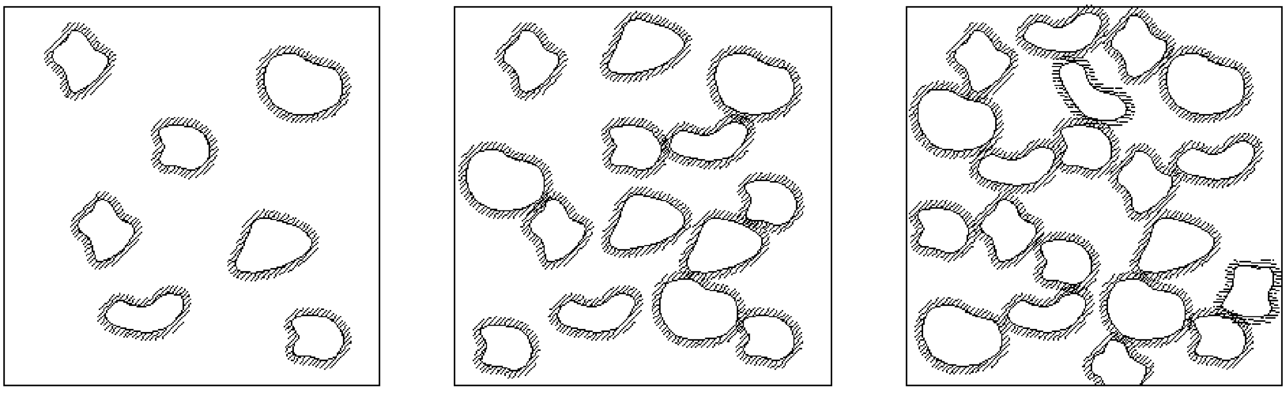
Figure 9.5 illustrates the concept of percolation through the ITZ.[9.3] The three situations are: (a) aggregate particles are widely dispersed and therefore "discrete" and ITZ zones do not overlap or coalesce; thus penetrable paths through the ITZ are not available; (b) with increasing aggregate concentration, particles approach each other more closely and ITZs begin to overlap and create possible transport paths; and (c) at a critical aggregate concentration, a point is reached when fully connected paths through the material are created, represented by an increase in penetrability. At this point, the ITZ becomes percolated, and further increase of aggregates should result in additional penetrability.

In practice, this idealised concept is modified by several factors, so that penetrability rises gradually rather than changing suddenly. The role of aggregates is also important, with very fine aggregates (< 100 µm) having less effect as they do not show the same ITZ phenomenon. Extremely fine particles actually participate in the "fine filler" effect which improves ITZ properties and therefore also durability.

In practice, most concretes and mortars are percolated. Experimental evidence and modelling of these effects suggest that, at total aggregate volume fractions of 55 % or higher – typical of most concretes – percolated pathways will be present due to overlapping ITZ regions.[9.12] The results also suggest that pore and ITZ refinement achieved by, for example, use of ultra-fine mineral additives such as CSF, will control or even eliminate percolation phenomena from real concretes or mortars, giving improved durability of these materials. Intelligent modification of the ITZ microstructure, for example through use of fine fillers or high shear mixing of concrete with a low water content, can therefore improve the durability of concrete in relation to deterioration by transport-mediated processes.

Transport properties of cracked concrete

Knowledge of the transport properties of cracked concrete is essential for predicting its durability since most deterioration mechanisms depend on the flow of aggressive agents through the cracked concrete. The presence of cracks substantially modifies transport properties of intact concrete. Microcracks that are discrete and well distributed will influence transport in a very different manner compared to visible, connected, localised macrocracks. Transport in a cracked concrete is


(a)
Unpercolated, discrete isolated particles

(b)
Unpercolated, some continuous ITZ regions

(c)
Percolated, continuous ITZ regions

Figure 9.5: Schematic of penetrability and percolation related to the ITZ[9.3]

therefore a coupled phenomenon between the matrix and the crack.[9.13]

Since the kinetics of different transport processes vary, changes resulting from cracking depend on the predominant transport mechanism.[9.14, 9.15] Moreover, it has been shown that the main parameters for describing flow in damaged and sound material are different; for example in uncracked concrete permeability is related to its porosity, while in cracked concrete it is related to crack properties. Regardless of the transport mechanism, properties of the cracks can become more important in cracked concrete than the properties of the concrete itself. Parameters such as crack width and shape, crack density/frequency and degree of connectivity, as well as crack origin, govern transport in cracked concrete.[9.16] For the above reasons, predictions of the behaviour of cracked concrete cannot be made accurately based on data for uncracked concrete.

Transport processes in cracked concrete can be studied using methods such as numerical simulation/modelling,[9.17] feedback-controlled splitting tests[9.18] and experimental imitation of various cracking mechanisms that RC structures encounter under actual service conditions.

Model (simulated) or artificially induced cracks can be used in studying transport mechanisms in cracked concrete, as they are much easier to characterise and control than real cracks. However, any models derived from experimental artificial cracks should always be calibrated on real concrete structures to ensure their reliability.

9.3.2 Mechanical and physical properties

Structural design engineers are most familiar with the mechanical and physical properties of concrete, such as strength (compressive and tensile, shear, etc), stiffness (elastic modulus), creep, shrinkage, thermal expansion coefficient and density. This is because design generally concentrates on not exceeding various limit states of strength, stability, and deformations which relate to these properties.

It is the predominance of concrete strength and stiffness in structural design that has led to the use of these properties, traditionally, as the means for specifying and controlling concrete durability. While there are broad correlations between concrete strength and its other properties including durability, strength should not be used as a proxy for durability. There are numerous examples of concrete structures of adequate strength, yet deteriorating prematurely. Partly this is because considerably less cement is needed in modern structures to obtain the required strength due to improved cement manufacture, giving rise to higher water:cement ratios. This has resulted in structures becoming more "penetrable" and thus more durability-sensitive, which, when coupled with the increasingly severe environments to which concrete structures are being subjected (eg due to higher CO_2 levels in the atmosphere, increasing industrialisation and pollution, etc) has resulted in the present phenomenon of a large stock of deteriorating concrete infrastructure.

Modern approaches to durability for concrete structures involve explicit and rational formulations that permit durability to be controlled directly through ensuring that the concrete properties meet criteria that give reasonable assurance of adequate service life. These properties are no longer simply those related to strength.

9.3.3 Chemical properties

Concrete resists aggressive environments by a combination of its physical resistance and its chemical resistance. The former derives from its dense microstructure and complex pore system that inhibits the movement of aggressive agents. Its chemical resistance relates to the thermodynamic stability of its hydrates and other phases in the presence of aggressive or transported agents, and to the nature of the interaction of these phases with the agents. Thus, in some cases, aggressive agents will attack the concrete phases directly and cause deterioration, such as in sulphate attack (external) or alkali silica reaction (internal), whereas in other cases certain cement phases will immobilise agents such as chloride ions, thereby slowing their ingress into the concrete. This emphasises the importance of a thorough understanding of transport properties and deterioration mechanisms in concrete.

The fact that concrete generally performs well in even quite aggressive environments is testimony to the stability of its phases in the hardened and mature state. For example, concrete generally resists normal waters and dilute solutions of many aggressive chemicals for long periods of time; it is stable in non-aggressive water as well as almost indefinitely in dry air; it provides an ideal environment for protecting reinforcing steel from corrosion and is able to withstand considerable physical abrasion. For these reasons, concrete is likely to continue to be a significantly durable material into the future.

Notwithstanding the above, concrete does deteriorate at measurable rates in certain cases, and this has to be allowed for in design. The task of the designer is to maximise the ability of concrete to resist deterioration in its design environment for an adequately long period of time. This implies making best use of its inherent chemical stability together with its physical robustness. In modern concretes, this is largely achieved through the use of blended cements which produce greater quantities of hydrates in the concrete, and also generally improve the pore and micro-structure. Blended cement concretes are now almost essential to durability in many cases, while at the same time they address environmental concerns such as carbon footprints. Such materials also generally allow durability to be achieved without resorting to excessively low water:binder ratios, very high strengths or significantly more costly concretes. This is because of their ability to better resist aggressive environments and immobilise species such as chlorides. Nevertheless, such concretes are often more sensitive to early-age effects related to temperature and curing, and this has an impact on construction practices.

9.4 Mechanisms of deterioration

9.4.1 General aspects

There are a number of mechanisms that can cause deterioration of concrete and a comprehensive discussion of all these processes is unnecessary in this context. Instead, the deterioration mechanisms are discussed in broad categories: deterioration primarily caused by physical mechanisms, and deterioration primarily caused by chemical mechanisms. Some of the more common forms of deterioration are discussed in each category and strategies for mitigating the effects of such deterioration are also presented. Furthermore, because of its importance as a mechanism of deterioration, corrosion of reinforcement is discussed under a separate heading, as being both a physical and chemical mechanism of deterioration.

The deterioration processes that affect concrete must be seen as an interaction between the concrete and its environment. The likelihood of a particular form of deterioration occurring, as well as the rate at which it will take place, depends on the nature and degree of aggressiveness of the environment as well as the physical and chemical resistance of the concrete. This also applies to the so-called internal mechanisms of deterioration such as alkali-silica reaction (discussed in Chapter 10). Environmental exposure conditions will have a significant influence on the rate of deterioration of the concrete.

It is possible to state general principles in regard to mechanisms of deterioration of concrete and strategies for limiting the rate of deterioration:

- Deterioration usually involves the movement of substances and fluids (liquids, gases, ions, etc) through the pore structure of the concrete. The fluid-transport properties of the concrete are therefore important parameters in determining the rate of deterioration.
- Almost all forms of deterioration involve the presence of water in the surrounding environment and in the internal pore structure of the concrete. Concrete in a dry environment therefore generally shows a lower rate and extent of deterioration than concrete in a wet or humid environment.
- The nature of the cement and cement extenders used in the concrete will have a primary influence in determining the resistance of the concrete to deterioration. In general, the use of cement extenders tends to enhance the durability of concrete.
- For most mechanisms of deterioration, the rate and extent of deterioration is much reduced by the provision of good quality dense and well-compacted concrete, particularly in the external cover zone of the concrete element.

9.4.2 Mechanical and physical processes affecting durability

Abrasion

Abrasion refers to wearing of the surface of concrete caused by repeated rubbing or frictional action. This problem manifests itself mainly in pavements or industrial floors carrying wheeled or pedestrian traffic.

Hardened cement paste has abrasion resistance that is lower than most aggregates. Hence, the abrasion resistance of concrete is largely determined by the hardness and quality of the aggregates. This is not to imply that the w:c of the concrete is unimportant in determining its abrasion resistance. It has been shown[9.19] that concrete abrasion resistance increases linearly with decreasing w:c. It is possible that this derives from the improvement in the resistance of the cement paste and the improved bond strength between aggregate and cement paste.

Erosion

Erosion refers to wearing of the concrete surface by the abrasive action of fluids and suspended solids as well as by wind-borne sand particles. Erosion is therefore a special case of abrasion. Nevertheless, the possibility of directed and focused application of stress on the cement paste phase of the exposed concrete surface, causing aggregate particles to be dislodged from the surface, makes erosion a potentially hazardous form of physical deterioration.

Cavitation

Cavitation occurs when a high velocity flow of fluid is subject to a sudden change in direction or velocity. This results in a zone of severe sub-atmospheric pressure immediately downstream of the obstruction or abrupt change, allowing the formation of pockets of vapour. As these vapour pockets leave the low pressure zone, they collapse by implosion, resulting in high, localised and repeated impact stresses on the surface. These stresses are known to indent hard metals and can cause fairly rapid deterioration of even good-quality concrete.

Freezing

Problems caused by freezing of concrete are seldom encountered in Southern Africa because of the relatively mild climate in this region. But there are some applications, such as cold rooms or cold liquid storage facilities, in which concrete is exposed to extremely low temperatures and could be damaged by the freezing of water in the pore structure.

Damage by freezing derives from the fact that water undergoes a volume increase of about 9% when it turns to ice. Hence, damage occurs when the water in the pore structure expands upon freezing and there is not enough empty space in the pore structure to accommodate this expansion. Tensile stresses are then set up and microcracking of the cement paste occurs. In freeze-thaw deterioration, the cracks are propagated by cyclic freezing and thawing actions.

There is much research evidence to show that the presence of entrained air in the concrete has the effect of reducing the expansive stresses that are caused by water freezing in the pores. In regions subjected to the problems of cyclical freezing and thawing of concrete, the resulting damage to the concrete can be eliminated almost completely by including an air-entraining admixture in the concrete mixture.

9.4.3 Chemical factors and processes affecting durability

Many chemical environments are considered deleterious to concrete and a more detailed treatment of the mechanisms of deterioration associated with each of these environments is presented later in this chapter. A summary of some of the chemical mechanisms responsible for concrete deterioration is presented in Figure 9.6. This figure also shows common examples of each of the reaction forms as well as the resulting forms of concrete deterioration.

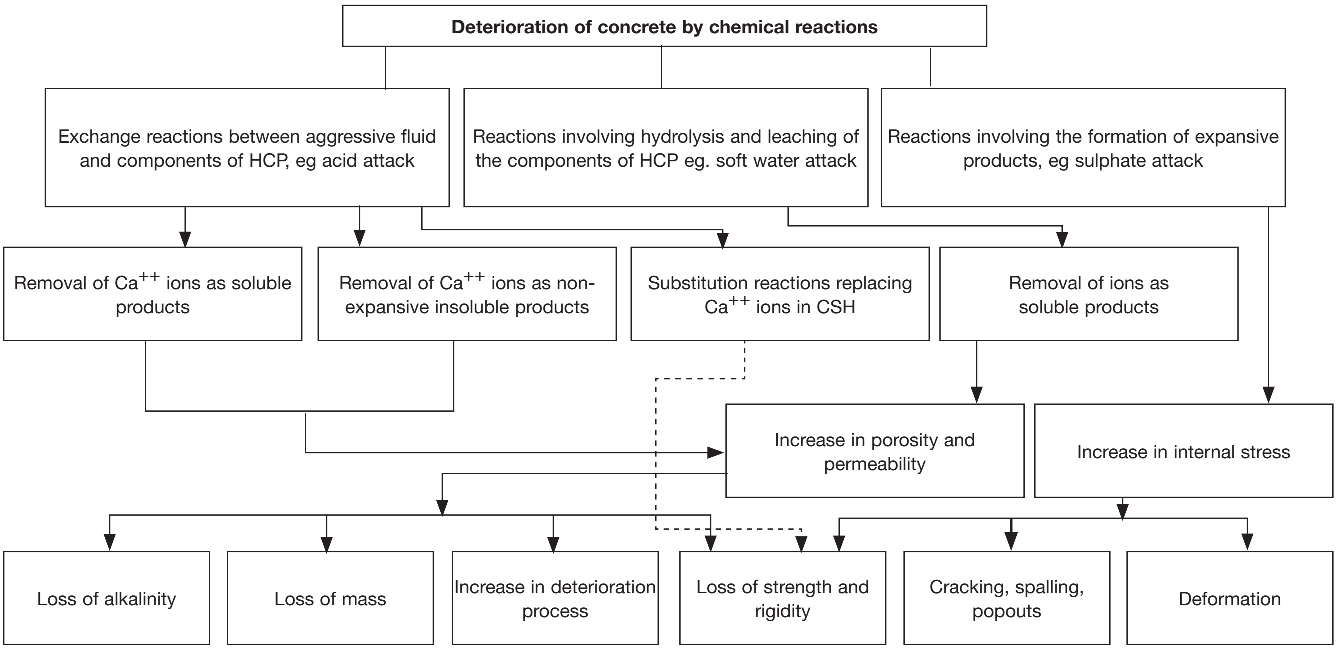
The following is a general description of the broad categories of chemical deterioration mechanisms shown in Figure 9.6.

Exchange reactions between acids and components of hardened cement paste (ion exchange or substitution)
Because of its generally alkaline nature, the products of hydration as well as unhydrated cement react with acids resulting in destruction of the cement paste matrix. This form of deterioration starts from the surface and continues into the concrete until all the acid or the cement paste has been consumed.

Dissolution of the products of cement hydration (ion removal)
The products of cement hydration are dissolved and leached out of the concrete causing the destruction of calcium silicate hydrate (CSH) gel with a concomitant loss of concrete strength. Dissolution of $Ca(OH)_2$ from concrete by pure water is typical of this form of deterioration.

Expansive stresses caused by conversion of the products of hydration by external agents (ion addition)
External aggressive agents enter the pore structure of the concrete and through reaction with the products of hydration, set up expansive forces which cause cracking and mechanical destruction of the concrete. The effect of sulphates on concrete is an example of this form of deterioration.

Alkali-silica reaction can be considered as a special case of this form of deterioration, in that the aggressive agents participating in the reaction are internal to the concrete.



HCP = hardened cement paste

Figure 9.6: Summary of some of the chemical reactions and their effects in concrete deterioration[9.20]

Nevertheless, destructive expansion is only possible if water is drawn in from outside the concrete. Hence, conditions of the external environment as well as the ability for water to move through the concrete, is also important in this form of deterioration. A more complete discussion of concrete deterioration by alkali-silica reaction is discussed in Chapter 10.

Steel corrosion in reinforced concrete caused by depassivation of the steel

This is a special form of deterioration caused by expansive stresses in the concrete generated by the presence of embedded corroding steel. An increase in volume occurs when steel corrodes. However, the pervasive nature of this form of deterioration demands that it be placed in a separate category as a particular area of study. In this category are agents and actions which, in themselves, are not aggressive to concrete but cause the destruction or de-stabilisation of an electro-chemically passive, gamma-ferric oxide layer on the surface of the reinforcing steel. This depassivates the steel, exposing it to corrosion if oxygen and moisture are present. Carbonation and chloride ion diffusion are the most common mechanisms resulting in this form of deterioration and these are discussed in more detail later in this chapter.

Other factors influencing the rate and extent of chemical deterioration

The following should be noted with regard to the broad categories discussed above:

- There are often “grey areas” in which a particular deterioration form cannot be placed in any particular category with certainty.
- There is often more than one category of deterioration operating at the same time. For example, concrete in contact with soft water containing dissolved carbon dioxide is deteriorated by leaching and acid attack.
- The chemical reactions associated with concrete deterioration are temperature dependent to varying degrees. In general, the rate of ingress of fluids into concrete, and the resulting rate of deterioration, increases with increasing temperature.
- Under normal environmental conditions, relative humidity or moisture content often plays a more important role than temperature. This is because the rate of movement of gases and ions through the pore structure of the cover concrete zone is significantly influenced by the amount of moisture in the pore structure. As examples:
  - The carbonation reaction will not take place if the pores are completely dry or completely saturated. The rate of carbonation is at a maximum when the relative humidity in the pore structure is between about 50 and 70%.
  - Penetration of chlorides into concrete takes the form of mainly ionic diffusion. The higher the moisture content of the pores, the higher is the rate of chloride penetration. The maximum rate of penetration occurs when the pores are saturated.
- The frequency and duration of wetting periods affect the moisture condition of the pore structure and therefore have a direct effect on the rate of deterioration. For example, carbonation ceases when the concrete surface is wet during a rainy period and will only restart when the concrete has dried to the depth of carbonation at the start of the rainy period. With the next rainy period, the depth of carbonation has increased and the time to dry to this depth will increase correspondingly. In this way, frequent rainy periods have a significant effect in reducing the rate of carbonation.
- The chemical composition of the air and the presence and amount of any potentially aggressive compounds should be identified. The more common of these, and their effects, are:
  - Carbon dioxide - carbonation
  - Salts - expansion on crystallisation
  - Chlorides - expansion on crystallisation and depassivation of steel
  - Sulphates - sulphate attack
  - Acidic vapours - concrete corrosion
- The microclimate of exposure can often have a marked effect on the rate of deterioration. The following are examples of categories that could be considered:
  - Exposure to rain and sun
  - Exposure to rain but no direct sun
  - Sheltered from rain and sun
- Synergy: consideration should also be given to synergistic interaction of two or more deterioration mechanisms. For example, the rate of free chloride penetration is greater if the concrete has already carbonated. This is because carbonation reduces the chloride binding capacity of the cement. On the other hand, while sea water has a relatively high concentration of sulphates, the effects of sulphate attack are significantly reduced by the presence of chlorides in the water. Chlorides render the reaction products of sulphate attack more soluble, which are then leached out rather than resulting in internal stress build-up.
- Concrete directly exposed to water requires special attention because of the greater potential for deterioration. This includes concretes that are submerged for long periods as well as those that are subjected to wetting and drying cycles, with the drying cycle being such that the concrete does not dry to an appreciable depth before the next wetting cycle (eg the splash zone in a marine environment). Deterioration of concrete exposed to water is often more severe than in the case of air exposure. The main reasons for this are:

- Most of the products of cement hydration are soluble in water to varying degrees.
- Water is able to dissociate dissolved salts and allow them to participate in ion-exchange and addition reactions.

However, concrete constantly submerged in sea water is less susceptible to reinforcement corrosion compared to concrete exposed to wetting/drying cycles, since, under submerged conditions, there is not enough oxygen available to sustain active corrosion.

9.4.4 Detailed discussion of some common mechanisms of chemical attack

Attack by atmospheric, sewer and silage gases

Nitrogen, in its elemental form, has little effect on concrete. However, when combined with hydrogen and other compounds to form salts such as ammonium hydroxide, this can be damaging to concrete. Such compounds are more commonly found in industrial environments and, when dissolved in atmospheric moisture or liquid water, will enter the concrete through the cover zone.

Other nitrogen compounds that may be encountered are its various oxides, usually designated NO_x, which are formed during high-temperature combustion processes. These oxides can dissolve in atmospheric moisture to form the corresponding acids which can react with the alkaline components of concrete in the same way as other acidic materials (see Attack by acids below) to lower the pH of the cover concrete.

Sulphur dioxide is an atmospheric pollutant resulting mainly from volcanic activity, discharges from tectonic fissures, and from the combustion of sulphur-bearing fossil fuels. It combines with atmospheric moisture to form sulphurous acid, which is oxidised to sulphuric acid and usually reaches exposed concrete surfaces in the form of acid rain. This process reduces the alkalinity of concrete and can have the more serious effect of spalling of the concrete as a result of sulphate attack. Horizontal concrete surfaces exposed to direct rainfall are particularly prone to this form of attack and, after some years of exposure, will show an "exposed aggregate" appearance, as shown in Figure 9.7.

Figure 9.7: A horizontal surface of concrete exposed to acidic rainfall in the Johannesburg urban environment. This concrete contains dolomitic aggregate and, in some areas, the aggregate particles are being attacked at a faster rate than the surrounding cement matrix. The width of the field of view is approximately 100 mm.

Hydrogen sulphide is sometimes encountered as an atmospheric pollutant in the vicinity of badly managed petrochemical and other plants where hydro-treated sulphur-bearing products are vented to the atmosphere. Hydrogen sulphide is also a major pollutant of the air spaces of sewers and is responsible for the typical upper-wall biogenically induced corrosion that takes place above the flowline in concrete sewer pipes and related structures.[9.20, 9.21] This form of deterioration is also common in deeper marine sediments containing organic deposits under anaerobic conditions. The formation of hydrogen sulphide under these conditions is complex, involving the action of various sulphide-producing bacteria. When released into adjacent moist areas where sufficient oxygen is also present, it is oxidised to sulphuric acid, this time by the action of sulphur-oxidising bacteria, and can then be responsible for acid and sulphate attack.

Carbon dioxide, although a minor component of the atmosphere, has an important effect in initiating the corrosion of reinforcing steel through a slow process referred to as carbonation. This is the process in which atmospheric carbon dioxide (CO_2) enters the pore structure of hardened cement paste and reacts with calcium hydroxide ($Ca(OH)_2$) to form calcium carbonate ($CaCO_3$):

$$CO_2 + Ca(OH)_2 \rightarrow CaCO_3 + H_2O \text{ ....} (5)$$

In this process, the pH of the concrete pore water is reduced from ca. 12,5 to ca. 8,5 upon complete carbonation. When this effect reaches the reinforcing steel, the reduced pH causes the passive gamma-ferric oxide layer on the steel to become unstable and the steel is depassivated. If sufficient oxygen and moisture are available, the steel will start corroding with subsequent loss in serviceability and load-bearing capacity of the structure.

Carbonation moves as a "front" into the concrete as shown in Figure 9.8. This front does not advance beyond a particular point until all the $Ca(OH)_2$ at that point has been converted to $CaCO_3$. Hence, the amount of $Ca(OH)_2$ in the pore structure of the concrete also has an influence on the rate of carbonation.

While some variation appears in the literature, the rate of advance of the carbonation front is generally taken as:

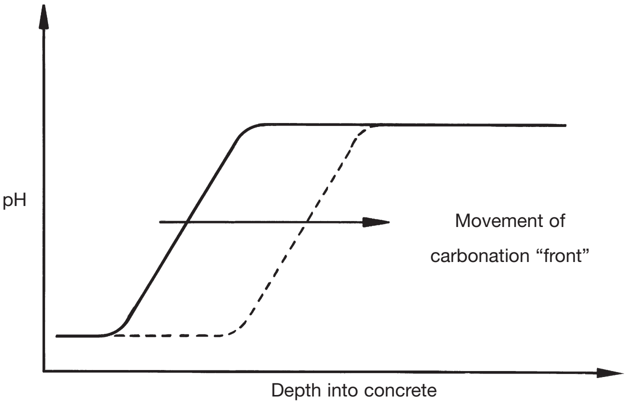


Figure 9.8: pH reduction due to carbonation moving as a "front" from the concrete surface

$x = Dt^n$ .... (6)

where: x = depth of carbonation
t = time of exposure
n = typically 0,5, but can vary from 0,4 to 0,6, depending on the conditions of exposure
D = carbonation coefficient

The rate of carbonation is strongly affected by the moisture content or relative humidity of the concrete pore structure. No carbonation takes place when the pores are completely dry or when they are fully saturated (in completely dry concrete, insufficient moisture is available for the chemical reaction and in saturated concrete, water-blocked pores prevent CO_2 diffusion). The rate of carbonation is at a maximum when the relative humidity is between 40 and 60%. The rate of carbonation also increases with increasing ambient temperature.

Dry periods promote carbonation whereas wet periods promote corrosion if de-passivation of the steel has already occurred. The risk of corrosion therefore increases with increasing time of dry periods. Hence, a better quality of concrete cover is required in climates with long dry periods and short wet periods.

The rate of carbonation is also influenced by the materials and mixture characteristics of the concrete. Important among these are:

- The binder composition and binder content of the concrete influence the nature of the pore structure and the amount of $Ca(OH)_2$ available for reaction. Smaller, disconnected pores as well as increasing availability of $Ca(OH)_2$ will reduce the rate of carbonation.
- Curing at early ages encourages hydration of concrete in the cover zone. This reduces pore sizes and the degree of inter-connection between pores, both having the effect of reducing the rate of carbonation.
- Incomplete compaction of the fresh concrete as well as surface cracking results in macro-voids, which encourage the entry of CO_2 and accelerate carbonation.

It is important to note that carbonation is generally not a durability problem in unreinforced concrete. In such structures, carbonation has the effect of refining and improving the near-surface pore structure of the concrete.

A common misconception is that submerged structures are immune from carbon dioxide attack. This is true only if the source of CO_2 is the air above the waterline. The normally low concentration of CO_2 in the atmosphere, its low solubility in water at atmospheric pressure and ambient temperature as well as the ineffectiveness of the gas transport mechanism across the air/water interface is responsible for the protection afforded by water to submerged structures. The position changes dramatically, however, if CO_2 is generated (by microbiological or other mechanisms) within the water contained in or surrounding the concrete structure, to form carbonic acid. Deeply submerged structures are especially vulnerable as the solubility of CO_2 increases with pressure with resultant higher concentrations of H_2CO_3. The initial reaction with the calcium hydroxide of the cement paste is the formation of almost insoluble calcium carbonate but this is rapidly followed by the conversion of the carbonate to the more soluble bicarbonate accompanied by accelerated leaching.

Attack by water

Water is an ubiquitous corrodent of concrete. It can be present as rainwater, wind-driven sea spray, immobilised soil water, stagnant impounded water, or it may be flowing turbulently through canals or pipelines or condensing on the surface of concrete towers. It can be pure or laden with many soluble contaminants. It can carry entrained abrasives or actively release gases that were dissolved in it under pressure at depths. The literature dealing with this topic is voluminous (see references 9.22 to 9.34) and only general principles can be dealt with in this chapter. Readers requiring a more detailed discussion should consult these and other references.

The most important factors affecting deterioration of concrete by water are the following:

Purity of the water

Pure or soft waters are aggressive to concrete primarily because of their "ion hungry" nature. These waters occur naturally in springs emanating from igneous rock formations in mountainous areas or in lowland streams or rivers which have not passed through calcareous strata. Soft waters are therefore characterised by their low dissolved ion content. When concrete is placed in contact with this water, the calcium hydroxide in the hardened cement paste is leached out as the water tries to establish an ion balance. Under these conditions the rate of $Ca(OH)_2$ removal is determined by the concentration gradient, ie the difference in concentrations of calcium compounds between the concrete and water phases.[9.27-9.29]

The hardness of water, as applied to concrete corrosion, is determined by the temporary hardness which is a measure of the concentration of the bicarbonate ion (HCO_3). This is different from the concept of total hardness which measures the concentration of calcium and magnesium cations.

The aggressiveness of soft water, measured as the rate at which $Ca(OH)_2$ is leached from the cement paste, is considerably increased if the water contains dissolved CO_2.[9.32-9.34] CO_2 dissolves in water to form carbonic acid (H_2CO_3) as follows:

$$H_2O + CO_2 \leftrightarrow H_2CO_3 \quad (7)$$

This CO_2 assists by first converting the $Ca(OH)_2$ to poorly soluble calcium carbonate (carbonation reaction) and then to soluble calcium bicarbonate which is easily removed by the water. These reactions are represented as follows:

$$Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O \quad (8)$$

$$CaCO_3 + CO_2 + H_2O \rightarrow Ca(HCO_3)_2 \quad (9)$$

Carbon dioxide exists in water mainly in the dissolved form with a small proportion combining to form carbonic acid. The effect of CO_2 in pure water is therefore to increase the acidity of the water. The solubility of CO_2 in water increases with decrease in temperature and at 18°C, pure water saturated with CO_2 has a pH of around 4,0. In natural soft waters the acidity is usually a function of the CO_2 content but this can be distorted by the presence of organic acids or polluting mineral acids.

It should be noted that a certain amount of dissolved CO_2 is required to stabilize the carbonate – bicarbonate equilibrium in the water (reaction (8) above). This CO_2 is therefore not available for $Ca(OH)_2$ leaching from concrete and it is only the CO_2 in excess of this amount which contributes to corrosion of concrete. This excess CO_2 is termed aggressive CO_2.

Figure 9.9 shows the surface of a 100-mm cube sample after exposure to soft water and abrasion for one year. In this case, the aggregate is not soluble in the soft acid water and the process of surface deterioration involves aggregate particles being removed when the surrounding cement paste has sufficiently deteriorated. When the aggressive soft water is largely stagnant, the use of calcareous aggregates, such as dolomite or limestone, can reduce the rate of deterioration because the aggregate particles also present calcium that can be leached to help reduce the concentration gradient that drives the deterioration process.

Acidity or alkalinity of the water

Acid waters are highly aggressive due to the enhanced reactivity and solubility of the cement binder components at low pH. Alkaline waters tend to be less corrosive and can be protective at high pH values.

Dissolved salts present in the water

These can profoundly influence the aggressiveness of the water. Calcium salts are generally protective, whereas magnesium and ammonium salts are corrosive. The sulphate radicle is especially injurious as the causative agent of spalling corrosion. Chloride ions become important where reinforcement is at risk due to inadequate cover, cracking or other factors.

Environmental conditions of exposure

These are of great importance in determining the durability of concrete exposed to aggressive water. The main rate-determining factors are the temperature of the water and the movement of the water relative to the concrete surface. Where concrete is exposed to moving or turbulent water, the sediment or solids load of the water can accelerate the deterioration by increasing the removal of deteriorated material from the concrete surface.

Wetting and drying

Wetting and drying of the cover concrete zone, such as encountered under marine tidal conditions, is responsible for additional stresses as soluble salts brought into the pores during the wet periods can crystallize during the dry periods and set up expansive pressures, which in turn can lead to spalling.

Assessing the aggressiveness of waters to concrete requires an holistic approach which integrates all the above factors into a quantitative measure of corrosive potential. The prediction model developed by Basson[9.35-9.37] provides a useful method to assess such aggressiveness. This prediction model has also been incorporated into South African national specifications[9.38], and is briefly covered in section 9.5.1.

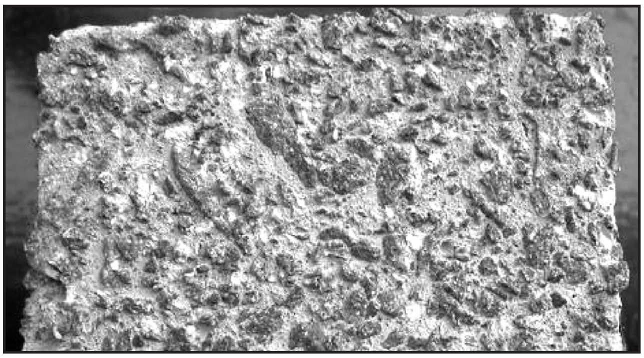


Figure 9.9: A concrete sample after approximately one year of exposure to soft water and abrasion. The width of the sample is approximately 90 mm.

Attack by soils

Completely dry soils are considered to be non-aggressive towards concrete. These soils are encountered only in desert and other very arid regions. In most areas periodic wetting of soils occurs as a result of the prevailing rainfall. In swampy or other water-logged areas soils may be permanently damp. Wet or damp soils may or may not be aggressive depending on the nature and concentration of water-soluble ions that are held in water present in the pore spaces. The aggressiveness of soils depends therefore on the amount and the aggressiveness of the water present in the soil, and these may be subject to periodic fluctuations. An analysis of the soil water, augmented by periodic measurement of soil conductivity, can be used to assess its potential effect on the durability of concrete structures such as foundations. Interpretation of the results of a groundwater analysis can be complex and should be left to a specialist in the field of groundwater aggressiveness. Because of the nearly static conditions that usually prevail at the soil-concrete interface, the chemical treatment of soils in this zone is feasible in some cases and can be used to reduce the corrosive potential of aggressive soils. As an example, addition of lime to backfilled soil around a concrete foundation can significantly reduce the aggressiveness of acidic water in the adjacent soil.

Attack by marine sediments

These can be considered to be a special case of water-logged soil conditions. The reason they are discussed separately in this chapter is that such sediments often contain large deposits of decaying organic matter derived from the buried detritus of marine organisms. The decay takes place mainly through bacterial action under anaerobic conditions, often under appreciable pressure from the overlying burden of marine sands and hydrostatic head. Corrosive gases such as hydrogen sulphide resulting from the decomposition reactions are effectively trapped and can reach high concentrations. When concrete piles are driven into, or concrete pipes laid through these sediments, they are disturbed, and gases previously sealed in are released. These gases are then in direct contact with oxygenated sea water, and concrete structures in the immediate vicinity are subject to the highly corrosive conditions generated by these gases and their oxidation products. The mechanism of attack is qualitatively similar to that encountered in sewers but, insofar as it takes place under submerged marine conditions at higher pressures and at higher corrodent concentrations, it is more severe.

Attack by acids

The integrity of hardened portland cement binders is dependent on maintaining the high levels of alkalinity that stabilize the gel compounds responsible for cementitious properties. Acids react with the alkaline components of the binder, lowering the degree of alkalinity or, in extreme cases, completely neutralising the alkalinity. These chemical changes are disastrous and disintegration of the matrix is the inevitable result. While cement types (see Chapter 1) differ in their degree of resistance to acids, these differences are small and no concrete made with such materials is truly resistant to common inorganic acids. Organic acids can be as corrosive if their reaction products are soluble, but in some cases the rate of attack can be retarded by the formation of an insoluble surface layer. Readers in need of more detail should consult references 9.32 to 9.34.

Attack by alkalis

These are generally benign materials except where aggregates susceptible to alkali attack are present as components of the concrete (see Chapter 10). Because of the high pH of alkaline environments, the integrity of the matrix is not otherwise threatened and alkaline attack is not usually a problem affecting the durability of concrete unless it is exposed to extremely high concentrations of the alkali metal hydroxides, such as concentrated caustic soda. Further details are provided in reference 9.39.

Attack by sulphates

The sulphate ion deserves special mention due to its importance in promoting spalling corrosion. The exact mechanism of sulphate attack is not clearly understood[9.30,9.31] but it is generally agreed that ion-exchange and ion-addition reactions result in changes in the molecular sizes and crystal habits of the compounds normally present in the HCP.

The most generally accepted reaction mechanism postulates that the deterioration effect of sulphates on concrete occurs through a complex mechanism that involves a number of secondary processes. The process is initiated by a reaction between the sulphate ion and $Ca(OH)_2$ in the hardened cement paste as follows:

$$Ca(OH)_2 + SO_4^{2-}(aq) \rightarrow CaSO_4.2H_2O + 20H^-(aq) \text{ .... } (10)$$

This reaction is termed gypsum corrosion and is associated with a small increase in volume. However, the effect of expansion due to gypsum corrosion is only significant at relatively high levels of sulphate concentration (3 000 ppm). It should be noted that, in a water environment where leaching action is possible, the gypsum will be removed with the calcium hydroxide because of its higher solubility. This will reduce the extent of subsequent sulphoaluminate corrosion. (Sulphoaluminate is the product of an intermediate reaction between sulphate and C_3A.

The major cause of deterioration is sulphoaluminate corrosion which occurs as follows:

$$\underset{\text{Monosulpho-aluminate}}{C_4AS^*H_{12}} + \underset{\text{Gypsum}}{2CS^*H_2} + \underset{\text{Water}}{16H} \rightarrow \underset{\text{Ettringite}}{C_6AS^*3H_{32}} \text{....} (11)$$

where: C = CaO; A = Al_2O_3; S* = SO_3; and H = H_2O

This reaction is accompanied by a large increase in solid volume, causing internal stresses that lead to cracking.

The cation accompanying the sulphate ion can affect the aggressiveness of the sulphate. Ammonium and magnesium sulphate are particularly aggressive because these cations also participate in reactions with $Ca(OH)_2$ in the HCP. In the case of magnesium sulphate, the reaction, also accompanied by an increase in volume, proceeds as follows:

$$MgSO_4 + Ca(OH)_2 \rightarrow CaSO_4 + Mg(OH)_2 \quad \text{................} \quad (12)$$

The magnesium hydroxide is precipitated because of its low solubility and the CSH decomposes to release $Ca(OH)_2$ so as to re-establish the balance. This is similar to the leaching action of pure water and also leads to the destruction of the binding agents of the HCP.

An interesting result of this action is the fact that concretes in sea water suffer less deterioration due to sulphate attack than would be expected purely from an assessment of the sulphate content of the water. Sea water contains a fair amount of magnesium and the protective effect of the $Mg(OH)_2$ deposited in the surface pores of the concretes contributes to the reduced deterioration. Another reason is that gypsum and ettringite are more soluble in solutions containing chloride ions, causing a reduction in the amount of expansion.

The net effect of reactions from attack by sulphates is usually expansive pressures that are generated within the cover or surface zone; these pressures result in tensile stresses in the concrete. Such stresses may exceed the tensile strength of the concrete and, unless the concrete is very lean, spalling is often the end result. Figure 9.10 shows a sample of concrete from an industrial floor onto which sodium sulphate had been spilled. The surface of the concrete is already seriously deteriorated and ready to spall. However, below this near-surface zone, the expansion of the cement phase has produced stress large enough to fracture the aggregate particles.

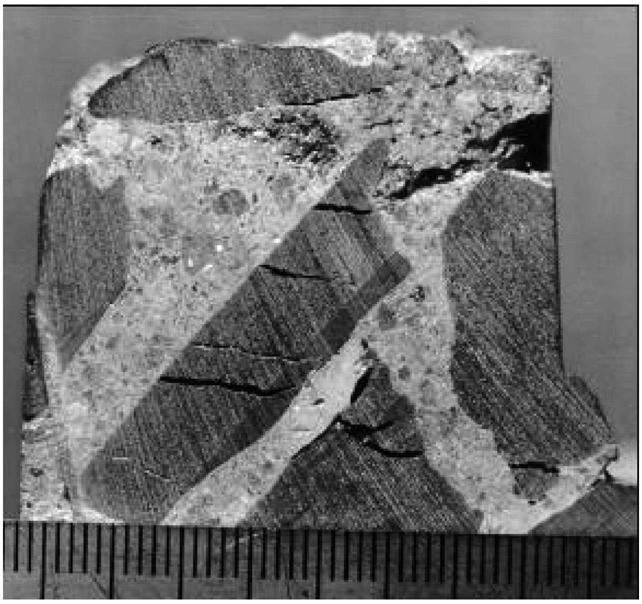


Figure 9.10: Sulphate attack of concrete from an industrial floor showing spalling action in the near-surface zone as well as cracking of aggregate particles due to expansion of the cement phase (the scale is in 1 mm graduations)

Other chemical influences

Many industrial and agricultural chemicals can react with concrete to reduce its durability. This is a specialised topic, however, and cannot be adequately addressed in this chapter. The reader may obtain further information from references 9.40 and 9.41.

9.4.5 Corrosion of reinforcement

Mechanisms of corrosion of steel

Detailed accounts of the complex mechanisms of metallic corrosion can be found in textbooks dealing with the subject.[9.42-9.44] For the purpose of this discussion, suffice to say that the corrosion of steel in the presence of electrolytes is an electrochemical process mediated by the formation of cathodic and anodic areas on the surface of the steel. These result from localised chemical or physical differences from point to point on the apparently homogeneous surface of the metal. The differences may arise during manufacture when small inclusions can be worked into the surface or when the surface becomes oxidised as a result of hot rolling operations. Localised stress points or even differences in the orientation of magnetic domains can also lead to differences in electrogalvanic potentials. The local electric currents that flow as a result of the formation of these electrolytic cells have been measured and found to be sufficient to account for the corrosion rates observed in practice. Localised differences can also arise within concrete as a result of differing oxygen or ionic concentration at the surface of the steel.

The equations governing the corrosion of a divalent metal, M, in the presence of oxygen can be simplified to the following:

At the anode: $M = M^{++} + 2e^-$ .... (13)
At the cathode: $\frac{1}{2} O_2 + H_2O + 2e^- = 2OH^-$ .... (14)

where e^- is an electron or unit of negative charge

From these equations it will be seen that oxygen is essential to sustain the reaction, which ceases in the absence of this element.

Steel reinforcement is usually delivered to the construction site in the as-manufactured condition with a tightly adhering layer of cracked millscale on its surface. Bends or other forming processes cause further cracks in this surface layer, which may delaminate. Welds are often not properly cleaned and acidic fluxes are often encountered. While on site, rusting of the steel can occur, and deposition of salts and other contaminants is seldom guarded against. Once embedded in concrete, the surface of the steel is therefore characterised by electrochemical corrosion cells having large cathodic and small anodic areas together with an electrolyte of contaminants sufficient to carry a corrosion current. If exposed to a source of oxygen, the corrosion cells will be activated and, because of the geometric relationship between anodic and cathodic areas, a particularly destructive form of pitting corrosion will occur.

Passivation of embedded steel

Fortunately, the corrosion cells present on the surface of the reinforcing steel are inactive when the steel is embedded in concrete. This is due to:

- The high pH conditions present at the concrete-steel interface, which promote the formation of a protective film of gamma ferric oxide on the surface of the steel
- The absence of aggressive electrolytes capable of destroying this protective film
- The insufficient availability of oxygen to sustain corrosion reactions, even in cases where corrosion cells may have become active due to surface contaminants when the steel was exposed before concreting

While these conditions are maintained, no corrosion of the steel reinforcement will occur.

Loss of passivation

Loss of passivation, and subsequent active corrosion of steel reinforcement, occur when the conditions described above are disturbed. The factors responsible may be one or both of the following:

- Carbonation or other acid attack of the concrete penetrating to the steel-concrete interface with a resultant lowering of the high pH conditions essential for maintaining the integrity of the protective ferric-oxide film on the steel surface. The carbonation process is discussed in section 9.4.4.
- Ingress of aggressive ionic solutions to the steel-concrete interface with resultant disruption of the passivating mechanism. The most notable example of this effect is the ingress of chloride ions in a marine environment.

Both these processes require some time for the carbonation front or the chloride ions to penetrate through the cover concrete and render the reinforcing steel liable to corrosion. The actual rate of reinforcement corrosion will then be determined by the types and concentrations of the ions that reach the steel as well as the concentration of oxygen that diffuses to the steel surface. In this regard the presence of the chloride ion is particularly important insofar as it catalyses the corrosion reaction, and even small quantities can be highly effective promoters of corrosion. Some important considerations of chloride attack are:

Chloride ion diffusion

A problem that presents itself at coastal environments is the ingress of chloride ions into the concrete. Chlorides may also be introduced into the concrete at the mixing stage, either as contamination of the mix constituents or as a component of an admixture. The presence of free chlorides at the level of the reinforcing steel also causes localised depassivation despite the pH being maintained at a high level. The reinforcing steel is then exposed to possible corrosion.

Some of the chloride which enters the concrete is chemically bound by the binder and depassivation only takes place when a critical concentration of free chlorides is reached at the level of the steel. The ability for the cement to bind chlorides ranges widely and depends on binder type. Alexander, et al[9.45] and Mackechnie[9.46] have shown the significant benefit of using FA and GGBS as cement extenders to minimise the effects of chlorides in reinforcement corrosion. This effect is clearly shown in Figure 9.11, which indicates the effects of these binder types in reducing the chloride conductivity of the cover zone of concretes with a range of strength grades, tested at 28 days after casting. Importantly, when the test is repeated at 90 days after casting, the FA and GGBS concretes show a further reduction in chloride conductivity and the two curves are closer to each other, while the CEM I curve remains largely unchanged. This indicates the benefits of continued curing and hydration of concretes containing FA and GGBS as well as the fact that, given sufficient hydration time, FA concretes can provide similar resistance to chloride penetration as GGBS concretes.

Magee[9.47] showed the benefits of using CSF to reduce chloride conductivity of concrete although the effects are

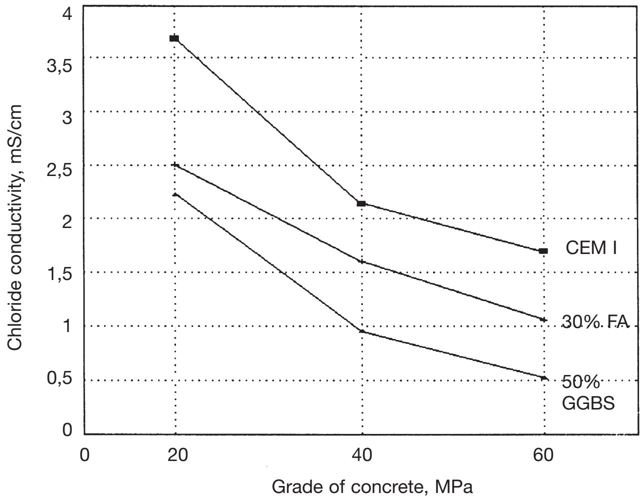


Figure 9.11: Effect of FA and GGBS in reducing the chloride conductivity of concretes over a range of strength grades[9.45]

less marked than for FA or GGBS concretes. Also, ternary blends of PC, GGBS and CSF gave further improvements in resistance to chloride ingress.

The diffusion of chloride ions into concrete takes place only when there is moisture in the pore structure. The higher the moisture content of the concrete, the higher is the rate of diffusion. Alternate wetting and drying cycles may create the added problem of salt crystallization with associated expansive cracking of the concrete.

Mackechnie[9.46] proposed a model for predicting the time rate at which the concentration of chlorides increases at any point below the surface of a concrete element exposed to external chlorides. The model has been assessed against measured results on old structures in South African marine environments and was found to give good predictions. Figure 9.12 shows the results of the prediction model for concretes made with the three types of cementitious materials shown, cured for the first three days at 90% RH, 60 mm depth of cover to reinforcement and subjected to extreme exposure in a marine environment. This figure again shows the significant advantage of using GGBS and FA as cement enhancers for concrete in marine environments. Regarding the use of Corex slag (GGCS) from the Western Cape, similar advantages in resisting chloride ingress are also realised.[9.48]

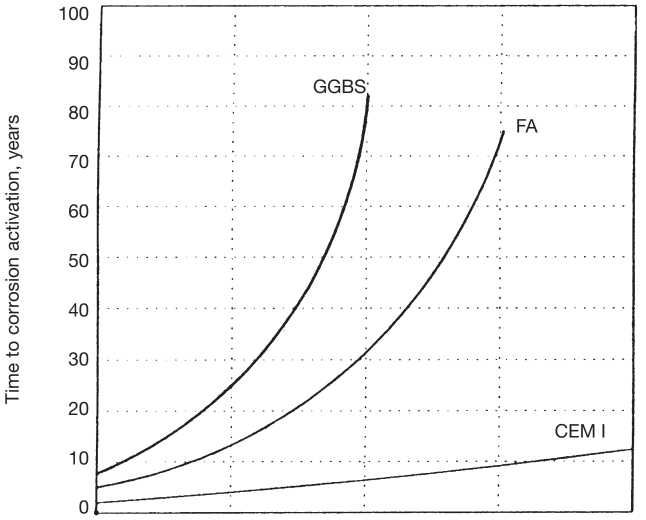


Figure 9.12: Model prediction results of the time to corrosion activation for different concrete types[9.46]

Synergy between carbonation and chloride penetration

When carbonation takes place in concrete into which chlorides have already penetrated (or when chlorides penetrate into carbonated concrete), the two processes interact to accelerate the depassivation of the reinforcement. The pH-lowering effect of carbonation causes some of the chloride which is chemically bound to the cement to be released. This increases the amount of free chloride ions available for depassivating the reinforcement. The ferrous chloride which forms initially is oxidised to ferric chloride, which in turn is precipitated as ferric hydroxide, so freeing the chloride ion for another cycle of corrosive attack. As mentioned above, oxygen is required for this reaction and its availability is often the rate-determining factor. In the absence of dissolved oxygen the reaction cannot go to completion and corrosion is stifled.

Influence of the environment on chloride penetration

The severity of chloride attack depends to a large degree on environmental exposure conditions. Classifications of environmental aggressiveness have been included in BS EN 206-1:2000[9.49] and modified for South African conditions, as discussed in section 9.7.3.

Steel corrosion in cracked concrete

As mentioned earlier, the penetrability of concrete, which is mainly governed by the quality and condition of the cover concrete, can substantially increase in the presence of cracks and may aggravate, for example, steel corrosion in reinforced concrete structures by creating preferential paths for the ingress of various types of potentially deleterious species (such as: H_2O, O_2, CO_2, Cl^-, SO_4^{2-}). The result is accelerated corrosion damage and consequently a substantial reduction in service life owing to rapid initiation and propagation of steel corrosion.

The extent to which concrete penetrability is increased depends on crack characteristics such as crack width, density and frequency, orientation (relative to the steel reinforcement), self-healing characteristics, degree of connectivity as well as whether the cracks are active or dormant. The accelerated corrosion damage referred to will depend on the combination of these crack characteristics and the deterioration mechanism. Otieno[9.50, 9.51] investigated the influence of crack widths on chloride-induced corrosion and observed that corrosion rates increased with increasing crack width, as shown in Figure 9.13.

The use of supplementary cementitious materials (SCMs) such as FA, CSF and GGBS reduces the effect of cracks to some extent but the impact of cracks on corrosion still remains detrimental.[9.52] For these reasons, the consideration of cracks is important in the service life design of reinforced concrete members.

Effect of reinforcement corrosion on structural appearance and durability

The rust stains that initially appear on the surface of a concrete structure are often the first signs of the corrosion of reinforcement that is taking place deeper within the concrete. However, the destructive effects of reinforcement corrosion are due to the fact that the corrosion products of steel occupy a volume more than twice that of the parent metal. Unless the concrete has an open pore structure (in which case rust may form in such pores with little or no pressure being generated), the expansive pressures set up can exceed the tensile strength of the concrete and spalling of the concrete cover over the reinforcement will occur. This exposes the steel directly to the environment and the accelerated corrosion that results soon causes further destruction of the remaining concrete adjacent to the steel.

Additional mechanisms for protection against corrosion

Coatings of various types may be applied to the steel reinforcement to enhance its performance under aggressive conditions where passivation by the mechanisms described above cannot be guaranteed for the anticipated service life of the structure. Organic coatings such as epoxies provide an impervious barrier on the steel surface, thus excluding the corrosive environment and preventing any reaction between it and the steel. Metallic coatings, such as hot-dip galvanising, are selected to be anodic to steel and thus afford cathodic protection. In both cases the coatings must be applied to properly prepared and thoroughly clean surfaces. Such preparation provides the bonus of the removal of millscale and other contaminants responsible for many of the electrochemical corrosion cells that are normally encountered. This has the advantage of obviating the worst effects of active corrosion cell reactions in areas where mechanical damage to the coating may have exposed the steel. Other forms of cathodic protection (see more specialist literature on this subject[9.53]) which depend on embedded or external sacrificial anodes or impressed currents have been employed in special cases. These are best implemented at the design stage since many factors, such as electrically conductive connection to reinforcement, spacing and placement of anodes and control of impressed currents, are crucial to the technical success and economic viability of this technique. The design and implementation of cathodic protection systems is best undertaken by specialists in this field.

Low-alloy steels have more tightly adhering protective oxide layers than mild steels and these may provide improved performance under some conditions. High-alloy and stainless steel surfaces have highly impervious, self-healing, tenaciously adhering oxide layers that greatly enhance their anticorrosive properties under many conditions, but they remain vulnerable to certain organic acids and oxygen-depleted environments that inhibit the formation or healing of their protective oxide layers. A thorough assessment of the conditions that will be encountered by the reinforcement is essential to ensure that the desired protective mechanisms will be able to operate and that an optimum choice is made.

9.4.6 Damage caused by electromagnetic and nuclear radiation

Portland cement concretes are composed almost entirely of inorganic compounds with molecules containing only ionic bonds. The electromagnetic radiation reaching the earth's surface from the sun does not contain photons that are sufficiently energetic to disrupt these bonds and concrete is characterised by excellent resistance to the incoming flux of electromagnetic radiation from this source.

When concrete is used as a shielding material for nuclear reactors the picture changes: close proximity to a source of

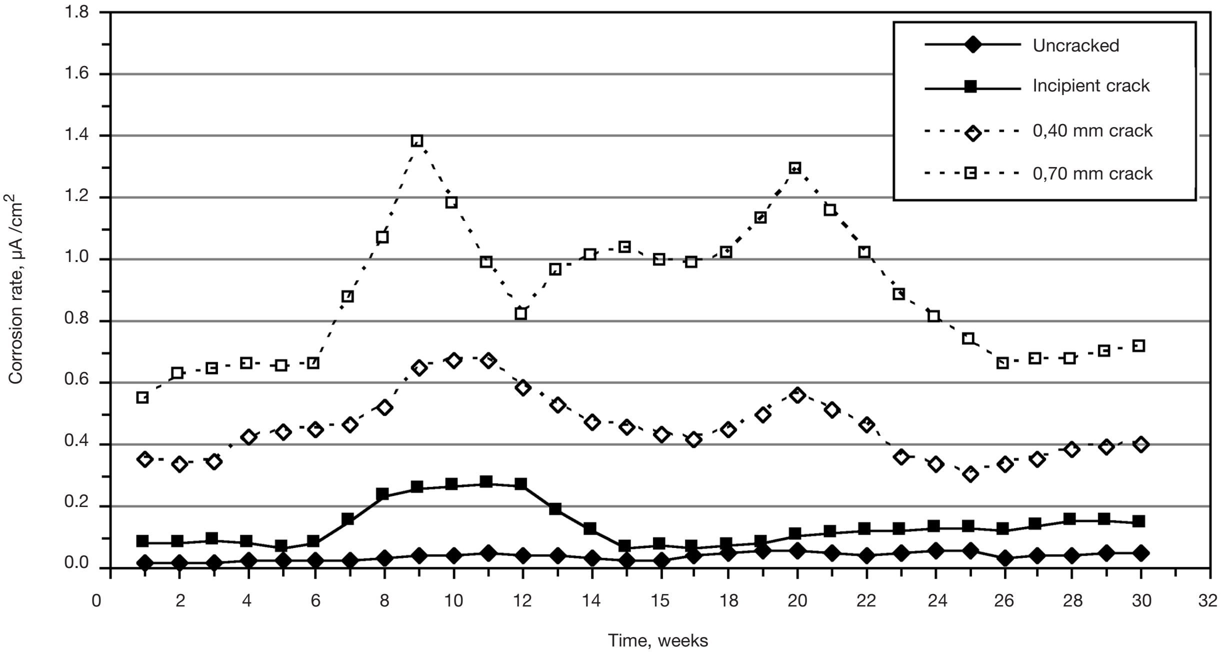


Figure 9.13: Influence of crack width on chloride-induced reinforcement corrosion (PC concrete, cover = 40 mm, w:c ratio = 0.4) ("Incipient" crack refers to a crack of effectively zero width created by loading to the point of incipient cracking)[9.50, 9.51]

energetic particles and photons unattenuated by distance and atmospheric absorption, coupled with the deliberate use of aggregates selected on the basis of being able to absorb such radiation, means that the concrete is required to dissipate in an acceptable manner a considerably larger and more penetrating energy flux. Unless precautions are taken to deal with the thermal effects and the generation of gases that may take place at depth within the concrete as a result of this bombardment, reduced durability may well result. Damage is caused mainly as a result of neutron and gamma radiation which interact with concrete to produce the following effects:

- Neutrons that penetrate into concrete are absorbed with the production of heat and secondary gamma rays. As a result of the low thermal conductivity of concrete this can result in localised significantly elevated temperatures.
- Radiolysis reactions between incoming gamma and neutron radiation and the water and other molecules present in, or at the surface of, the concrete generate gases consisting mainly of hydrogen and oxygen, but some nitrogen and carbon dioxide may also be present in the case of reactions taking place within porous cover zones. Internal pressures may develop as a result of this gas generation and the pressurised gases may promote corrosion of reinforcement by accelerating the reactions discussed earlier.

The design of durable concrete-based nuclear radiation shields is a specialised subject that requires detailed knowledge of relevant factors such as the types and energy fluxes of the particles to be absorbed, the absorptive capacities of the aggregates to be used, the degree of attenuation desired, the thickness of the concrete that will accomplish this and the anticipated service lifetime during which the performance must be maintained. Readers interested in more detail on this subject should consult more specialist literature such as reference 9.54.

9.5 Guidelines for ensuring durability

9.5.1 Concrete subjected to water-borne chemical attack

Two strategies are available for mitigating chemical attack of concrete. The first is to protect the concrete structure against attack by improving its intrinsic resistance through correct choice of cement type, mix design and sacrificial layer allowance, or to isolate it from the environment by means of suitable protective coatings. The second is to modify the environment to make it less aggressive to the concrete.

To decide which strategy, or combination of strategies, is the more appropriate it is necessary to quantify the degree of aggressiveness of the environment to which the concrete will be exposed. As chemical attack is usually mediated by water, either as the main corrodent or as the carrier of corrodents in aqueous solution, the most important factors that play a role in the aggressiveness of water are discussed below.

Temperature

The temperature of the water strongly influences the rate of reactions between the water and concrete. In general, the higher the temperature of the water, the more aggressive it will be. However, there are exceptions to this rule - some substances such as lithium sulphate (Li_2SO_4) and most gases have lower solubilities at higher temperatures.

pH

pH is significant because it indicates the potential of the water for dissolving the hardened cement paste (HCP). The lower the pH of the water, the greater is its aggressiveness.

Dissolved ions

In itself, a low pH is not a sufficient indicator of the aggressiveness of the water. It is also important to know the identity and quantity of the dissolved species responsible for the acidity of the water. The presence of sulphates, chlorides, magnesium and ammonium is also of importance.

Softness

The softness or calcium hardness of the water is also significant because of the potential for the water to leach $Ca(OH)_2$ from the HCP.

Turbulence and stagnance

Concrete exposed to moving water suffers the added effect of erosion and continuous removal of deteriorated material and soluble ions, exposing fresh surfaces for attack. Under stagnant conditions the speed of the reactions becomes diffusion-rate limited and the attack is slowed down significantly.

Wet/dry cycling conditions

Some concrete structures, eg those in tidal zones, are exposed to periods of immersion followed by periods of drying. Dissolved salts present in water that has penetrated pores and cracks during periods of immersion can crystallise from solution during periods of drying. The gradual accumulation of these crystals within the confined spaces will eventually fill them and exert expansive stresses. Where such conditions are present cognisance must be taken of their aggravating effects. In these conditions, the porosity of the concrete will be a critical factor.

Quantifying corrosivity

The so-called Basson index[9.35, 9.36] presents a useful quantification of the degree of aggressiveness of water to concrete. The method of calculation of this index has been incorporated into National Code of Practice SANS 10100-2.[9.38] The Basson index helps to quantify the aggressiveness of water to concrete by incorporating all of the influencing parameters discussed above. The calculation of the index presents a practical tool which can be applied to the complex problem of multi-factorial aggressiveness and can assist in choosing between various protection measures. Using key water parameters such as temperature, pH, dissolved ions, softness, turbulence or stagnance, and wet/dry cycling, water corrosivity is quantified in terms of two sub-indexes, a leaching corrosion sub-index, and a spalling corrosion sub-index. The sum of these two gives the so-called "aggressiveness" index. By relating these indexes to recommended aggressiveness limits, guidance is given on appropriate concrete mixes to resist the corrosive conditions and/or additional protective measures such as coatings to the concrete. (The interested reader should refer to the earlier (8th) edition of Fulton's Concrete Technology[9.37]).

Modifying the corrosivity of the environment

Within limits, there are certain applications that provide opportunities for reducing the degree of aggressiveness of the environment. Some of these methods are discussed below, with references provided to guide the reader to more authoritative texts. It must be emphasized that these methods should not be viewed as alternatives to ensuring good-quality, sound concrete nor should they be considered as being forgiving of poor quality concrete. They should rather be viewed as means for further enhancing the durability of concrete in particularly aggressive environments, where the concrete, on its own, would not provide an adequate service life.

Liming of acidic soils

It is generally not economically feasible to treat large areas of acid soil in order to sweeten the environment that concrete foundations or other buried structures will be placed in. But where the positions of these structures are accurately predetermined, the soil in the immediate vicinity can be excavated to a width and depth exceeding the requirements of the structure. This excess excavation can then be backfilled with an imported, uncontaminated soil which is mixed with sufficient builder's lime to provide an alkaline reserve for any acidity that may migrate from the adjacent soil. The imported soil should be selected and compacted with a view to minimising the permeability of the backfilled layer, thus minimising the flow of aggressive water at the concrete surface. This technique has been used in cases where the soil has become acidified due to having been over-treated with acidic fertilisers and on chemical waste disposal sites where acid sludges often form part of the dumped material. The advantages of such treatment are:

- There is a favourable protective environment for the concrete during the critical early-curing and hardening stages when it is especially vulnerable to attack.
- A cushioning layer of alkaline soil surrounds the concrete to protect against the later ingress of acids from the adjacent untreated soil.

The quantity of lime required for any particular soil condition may be determined by laboratory analysis of representative samples of soil taken from the contaminated areas concerned. Doubling of the amount required to neutralise the acids in the soil to provide an alkaline reserve is a reasonable rule-of-thumb. In practice the total quantity of lime has amounted to as much as 10% by volume of the soil being treated.

In the case of gross contamination of the soil with acidic materials, the alkaline reserve built into the backfill will eventually become depleted and acid attack of the concrete can then take place. Such attack can, however, be considerably delayed and replacement/pretreatment of the soil can very usefully extend the service life of a structure under these conditions.

Chemical treatment of aggressive waters

It is rarely economically justified to treat large water impoundments such as dams or lakes to reduce the aggressiveness of the waters contained therein. Where such waters are pumped through concrete pipes to concrete reservoirs, it may be feasible to pretreat that portion of the water being pumped to lessen its aggressivity, especially if similar treatment is in any case required to render the water fit for its intended end use.

Chemical treatment of water requires frequent analysis of representative samples of the water to determine the nature and dosage of the treatment, which can vary due to inflows or evaporation losses, eg during wet and dry seasons. The pioneering work of Langelier[9.55] is relevant in this context.

Generally only two types of aggressive waters can be beneficially treated: acidic and soft waters. Controlled liming of these waters serves to harden and sweeten them, thus making them far less aggressive. Reduction of other corrodents by chemical pretreatment is rarely economically feasible and indeed the injudicious use of certain flocculating agents to aid filtration of potable waters can actually increase dissolved sulphate levels.

The requirements of the end users of the pretreated water should always be borne in mind. Many laundries, for example, prefer to have soft water but would not object to slightly alkaline waters, whereas mineral-water and soft-drink manufacturers may have very different requirements. The post-treatment costs to such users of pretreated water must therefore be weighed against the costs of alternative methods of protecting concrete. These cautions are not intended to detract from the value of pretreatment as an anticorrosive measure; in selected cases it can be an effective technique.

9.5.2 Concrete subject to air-borne chemical attack

Atmospheric exposure conditions

The composition of the atmosphere is highly variable depending on location and prevailing weather. This variability must be taken into account when designing concrete structures exposed to atmospheric environments. Table 9.1 reflects the recommendations for concrete exposed to the atmosphere and has been divided into the following exposure zones:

- *Non-polluted:* This zone is characteristic of most non-industrialised rural and sparsely populated urban areas situated more than 20 kilometres from the coast.
- *Polluted:* Polluted zones are those occurring in most urban areas where heavy vehicular traffic conditions and moderate industrial activity are responsible for the release of corrosive gases into the atmosphere. Rural areas within 20 kilometres of the coastline are also included in this category as sea spray can be carried onto structures within this zone resulting in the deposition of potentially corrosive sea water salts on concrete surfaces.
- *Corrosive:* Corrosive areas are those in the immediate proximity of heavily air-polluting activities such as fossil fuel burning electric power generating stations, refineries, electroplating works and other chemical plants where the concrete structures are exposed to the released gasses.
- *Highly corrosive:* Such zones are those where acid rain occurs as result of the corrosive gases described above condensing onto structures or being washed onto them by the scrubbing action of naturally occurring rain. When these conditions are combined with the presence of marine aerosols generated by wind and wave action, the corrosivity is even further enhanced.

9.5.3 Concrete subject to physical attack

Section 9.4.2 deals in greater detail with the mechanisms underlying physical attack and should be consulted for the rationale of the following measures.

Improving the abrasion resistance of concrete

In addition to ensuring that the principles of producing good-quality concrete are adhered to, abrasion resistance of concrete is improved by ensuring that a particularly good quality surface is achieved. Marais and Perrie[9.56] advise:

- Attention to surface finishing techniques
- Particular attention to moist curing at early ages
- Consideration of the use of surface hardening agents

The abrasion resistance of concrete is determined by a combination of the strength and hardness of the aggregate together with the strength of the aggregate-cement paste bond (indirectly measured by the compressive strength of the concrete). As the strength of the concrete increases, the abrasion resistance of the concrete surface becomes increasingly sensitive to the strength and hardness of the aggregate used in the concrete. In concretes of low to moderate strength, abrasion damage of the surface occurs as a result of failure of the cement-aggregate bond and the aggregate particles are physically plucked out of the concrete surface. In this case, the strength of the aggregate itself becomes less important in determining the abrasion resistance of the concrete.

Measures for reducing erosion damage

These measures include the following:

- Improving the abrasion resistance of the concrete surface as discussed above
- Excluding as much abrasive material as possible from the system, eg building sand traps or designing stilling basins to be self cleansing
- Where possible, reducing the velocity of wind or water flow on the surface of the structure

Prevention of cavitation damage

The best way to prevent cavitation damage is to eliminate the occurrence of cavitation stresses wherever possible. This is done by appropriate design and detailing to keep flow velocities to a minimum and avoid sudden or abrupt changes in flow. Aeration of the water in areas of potential

Table 9.1: Recommendations for concrete exposed to the atmosphere

Specification for concrete to be used	Type of environment			
	Non-polluted	Polluted	Corrosive	Highly corrosive
Minimum cement content, kg/m^3	As dictated by strength requirements	340	380	420
Cement type	Any cement complying with SANS 50197-1 70% CEM I + 30% FA 50% CEM I + 50% GGBS			
Maximum w:c	As dictated by strength requirements	0,55	0,50	0,45
Minimum cover to steel, mm	25	30	40	50
Minimum strength	As per structural requirements			

cavitation damage has successfully been used in protecting dam spillways.[9.57]

Protection against freezing

It is now accepted that a controlled amount of air entrained in the concrete significantly reduces the damage caused by freezing. The entrained air bubbles act as capillary breaks and as pockets which accommodate the expanding ice and so reduce tensile stresses in the concrete. The amount of entrained air required is determined by the "spacing factor" [9.58] which is related to the average distance the water must travel to reach a free space.

In protecting concrete against freezing, the following points should he noted:

- Concrete must be protected against freezing during the first few days after placing even if it is air entrained.
- Air entrainment does not provide protection for the aggregates and where damage by freezing is a potential problem, attention must be given to the selection of aggregates.
- In structures like cold rooms, the low temperature has the effect of drying the concrete, thereby reducing the possibility of freezing damage.

9.5.4 Design and construction considerations for ensuring durability

Concrete mix design and materials selection

In cases where concrete durability is of concern, the selection of the concrete mix constituents and the mix proportions must be guided by the need to:

- Minimise the permeability of the concrete; and
- Maximise the chemical resistance of the concrete to the aggressive agents in the surrounding environment.

In practice, this means that aggregates, cementitious materials and admixtures must be selected so as to minimise the water content of the concrete and so reduce permeability. The chemical resistance of the concrete, which derives mainly from the cementitious material, will depend on the nature of the aggressive agents in the environment. For example, in a chloride environment, the chemical resistance of concrete is significantly improved if an iron slag (GGBS or GGCS) or FA are incorporated as cement extenders. The amount of total cementitious material is also important and should be judiciously selected. As examples, for a given water:binder ratio the cementitious material content of the concrete should, with due consideration for the other engineering properties of the concrete, be maximised in chloride or carbon dioxide environments, whereas the cementitious material content should be minimised in a sulphate environment, again with due consideration to the other performance requirements of the concrete. In addition to the importance of binder content, research is increasingly showing that the water:binder ratio is generally more important than the binder content per se, which may need to be considered for the specific case.

Cover to reinforcement

A dominant form of deterioration of concrete structures in South Africa is the corrosion of reinforcement resulting from inadequate cover, either by design and detailing or through negligence during construction. Section 8.2 of SANS 10100-2[9.59] provides guidance on the depths of cover required for different degrees of aggressiveness of the environment. These are minimum depths of cover and the suggested values are based on the assumption of acceptable curing conditions during construction. It needs to be noted that these limiting values for cover depths in the codes are prescriptive. Recently, the industry in South Africa has started to move to a performance-based design philosophy that includes the dual effects of concrete cover depth and quality of cover concrete, as discussed in section 9.7.

Consistently achieving the required cover depths on construction sites is often not a simple matter and requires particular attention and supervision. An important requirement is the provision of good quality, dense cover blocks, of the required thickness and placed on the reinforcement cage at spacings close enough to ensure that the cage does not move during concreting. An important aspect to be aware of is the allowable tolerances on concrete element dimensions and reinforcing steel bending dimensions. If the formwork is assembled to the minimum dimensional tolerance and the steel is bent to the maximum tolerance, a significant portion of the cover depth can be lost. As far as possible, the reinforcing steel should be detailed to allow relative movement between corresponding bars so as to allow for these dimensional variations while still maintaining the required cover depth.

In areas where reinforcing steel bars are spliced or overlapped, the overlap should be in a plane that is parallel to the surface of the concrete. If the plane of overlap is at right angles to the surface of the concrete, the cover depth could be reduced by the diameter of the steel bar.

Curing

Curing of concrete is primarily an activity that is aimed at encouraging the cementing reactions of the cement paste in the near-surface zone of the concrete member, thereby reducing the permeability or accessibility to external aggressive agents into the surface zone of the concrete. The various methods of curing are well documented and need not be repeated here (see Chapter 12). However, a few points need to be highlighted:

- The modern trend towards fast-track construction, while beneficial in terms of resource savings, often results in curing aspects being neglected. Construction methods such as slip-forming and bridge-jacking require special attention to curing because of the short time in which the concrete remains in the formwork.

- It is important that the curing system be designed around the type of structure as well as the construction method. Water curing is not always possible, convenient or economically feasible. For example, water curing of vertical surfaces requires special equipment and control, and may render the area inaccessible for further work to be carried out.
- Membrane-forming curing compounds can be convenient and effective means of curing concrete. However, not all curing compounds are equally effective and this aspect must be discussed with the supplier of the particular compound being used.

As a general rule, especially when durability is of concern, all concrete should be cured for the equivalent of at least five days of water curing at 22ºC. Refer to Section 10.8 of SANS 10100-2[9.60] for further details on methods and duration of curing.

It must be noted that the recently introduced performance-based design concept for concrete durability, as discussed in sections 9.6 and 9.7, deviates from the prescriptive approach for curing methods given in the code and allows a more rational construction procedure.

Design detailing

In addition to specifying the required cover to reinforcement with due regard to durability requirements, a number of other aspects of detailing can influence the durability performance of a concrete structure. Some aspects that require attention are listed below:

- Consider the use of small-diameter bars, well distributed to assist in reducing crack widths.
- Avoid abrupt changes in ratios of cross-sectional area of concrete and steel and ensure adequate reinforcement splice lengths.
- Ensure that, where steel is spliced, sufficient space remains for the concrete to be well compacted around the steel.
- Avoid contact between dissimilar metals.
- Ensure the provision of adequate movement joints and water drainage facilities in the structure.

9.6 Recent approaches to durability of reinforced concrete structures

9.6.1 General

For concrete structures, durability is generally defined as the ability of the structure to remain serviceable over a specified period of time without significant deterioration. However, as mentioned before, reinforced concrete structures all too often experience early deterioration and lack of durability. Coupled with capital investment and high costs for repair and rehabilitation, the concept of "design for durability" has gained increasing importance in recent years and is beginning to be considered as of equal importance as design for safety and serviceability.[9.60, 9.61] Modern approaches to the design of concrete structures include the aspect of durability and maintenance, the costs of which need to be minimised at the design stage.

For reinforced concrete structures, the main durability problem is corrosion of the reinforcement, resulting from the ingress of chloride ions or carbon dioxide, and the subsequent depassivation of the steel. Service life and durability of reinforced concrete structures therefore depend primarily on the penetrability and thickness of the concrete cover. Traditionally, concrete compressive strength has been used as an indicator of durability. However, increasingly engineers, researchers and specifying authorities accept that strength is not an adequate indicator for durability as it does not account adequately for the influence of constituent materials or construction process variables such as placing, compaction and curing. These factors affect the quality of the surface zone of the concrete and therefore have a direct influence on durability by controlling the movement of aggressive agents from the environment into the concrete.

9.6.2 From prescriptive to performance-based durability design

In general, design approaches for durability can be divided into prescriptive concepts, also termed deemed-to-satisfy concepts, and performance-based concepts. Prescriptive concepts result in material specification from factors such as exposure class and life span of the structure. Following this approach, durability specifications in most existing codes and standards are based primarily on establishing constraints to the mix proportions of the concrete as a function of the severity of the exposure.

Durability specifications in the South African standards[9.62] and the new European standards[9.63], for example, follow the prescriptive concept and are of the "recipe" type, setting limits on w:c ratio, cement content, and compressive strength for different exposure classes. Design for durability includes the correct choice of exposure class and compliance with material requirements, concrete cover specifications, and curing procedures.

However, many researchers and engineers argue that durability is a material performance concept for a structure in a given environment and that as such it cannot easily be assessed through simple mix parameters.[9.60, 9.64-9.69] The prescriptive approach ignores, to a large extent, the different performance of the various cement types and of the mineral components added to the cements or to the concrete itself, as well as the influences of on-site practice during the construction process.

Performance concepts, on the other hand, are based on quantitative predictions for durability from exposure conditions and measured material parameters. The resistance of the structure, measured through durability parameters of the actual concrete used, is compared against the environmental load, taking the influences of time into account. On this basis, deterioration of a structure during its lifetime is quantified using appropriate deterioration models. For the application of performance concepts, the following elements need to be developed:[9.60]

- Limit state criteria
- A defined service life
- Deterioration models
- Compliance tests
- A strategy for maintenance and repair
- Quality control systems

Different levels of sophistication may be applied to performance-based design for durability, including the use of durability indexes, the application of analytical deterioration models, and full probabilistic approaches, as discussed in the following section.

9.6.3 Different strategies for service life design

The fib Model Code for Service Life Design[9.70] proposes a design approach to avoid or minimise deterioration caused by environmental action, comparable to present approaches to design for loading. Based on quantifiable models for the "loading" (ie the environmental actions) and the resistance (ie the resistance of the concrete against the considered environmental action), the following design options are presented:

- Option 1: Full probabilistic approach
- Option 2: Semi-probabilistic approach (partial safety factor design)
- Option 3: Deemed to satisfy rules
- Option 4: Avoidance of deterioration

The full probabilistic approach, which is intended to be used for exceptional structures only, should be based on probabilistic models that are sufficiently validated to give realistic and representative results of deterioration mechanisms and material resistance. The basis of this approach is formed by appropriate test methods and statistical evaluation models.

In the partial safety factor approach, the probabilistic nature of the problem (scatter of material resistance and load) is considered through partial safety factors. It is based on the same models as for the full probabilistic approach and is intended to present a practical, yet statistically reliable design tool.

The deemed to satisfy approach is comparable to the durability specifications given in most current codes and standards, ie specifications based on a selection of certain design values (dimensioning, material and product selection, execution procedures) depending on environmental classes. The difference between the deemed to satisfy approach envisaged by the fib Model Code and traditional service life design rules is that the latter are commonly not based on material physical and chemical models but largely on practical experience, whereas the fib method is intended to be calibrated against the full probabilistic approach.

The fourth level of service life design (avoidance of deterioration) is based on the use of deterioration-resistant materials such as stainless steel, or protection systems such as coatings.

The necessary design tools for service life design are appropriate service life models and test methods for the verification of material characteristics, as discussed in the following.

9.6.4 Service life models for reinforced concrete structures

The service life of a concrete structure may be determined using a quantitative service life model to predict the time to cracking (or unacceptable damage) for a given environment. Service life models are generally based on laboratory and site data that are necessary to calibrate the model. Two different approaches may be taken. Firstly, concrete of a particular mix design is placed in a specific environment and data collected are used to predict the ingress of harmful substances, ie chlorides or carbon dioxide, in the same kind of concrete placed in a similar environment. Secondly, if the penetration depth of chlorides or carbon dioxide is known for a particular concrete at a particular time, it is possible to determine the time to corrosion, where the extent of corrosion is such that the structure will need repairs as a precautionary measure. For a full probabilistic approach, the variability of these aspects needs to be taken into account.

The two-stage service life model

A number of service life models for reinforced concrete structures exist. Many of these adopt the two-stage service life model first proposed by Tuutti[9.71], in which the deterioration is split into two distinct phases, namely the initiation period and the propagation period, as shown in Figure 9.14.

The initiation period defines the time it takes for chlorides or carbon dioxide (or other contaminants) to penetrate from the external environment through the concrete cover and accumulate at the embedded steel in sufficient quantity to break down the protective passive layer on the steel and thereby initiate an active state of corrosion. The length of this period is a function of the concrete quality, depth of cover, exposure conditions and the threshold or critical concentration required to initiate corrosion. No or minimal

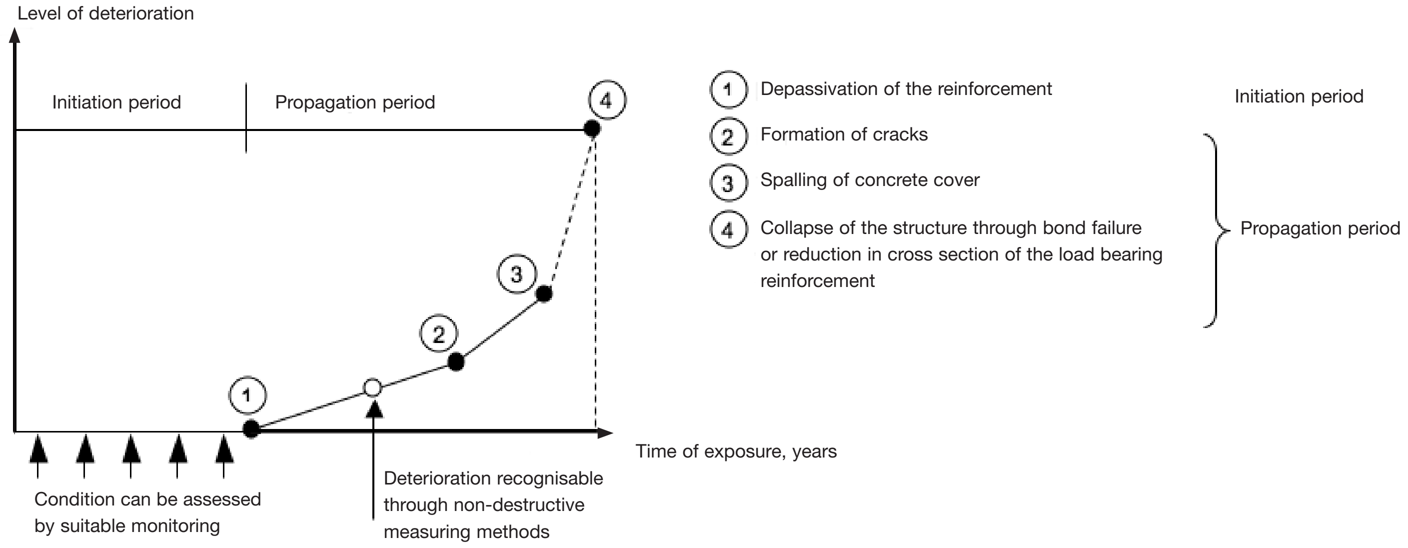


Figure 9.14: Deterioration process of reinforcement corrosion: two-phase model for service life[9.70]

damage to the concrete is assumed to occur during this period.

During the propagation period, reinforcement corrosion takes place, which may result in deterioration of the concrete itself. The propagation period can be divided into different limiting states, namely the onset of corrosion, the formation of cracks as a result of expanding corrosion products, delamination and spalling of concrete cover, and ultimately collapse of the structure.

Examples of service life models

Various service life models have been developed in different parts of the world.[9.72] The European model "DuraCrete"[9.70, 9.73] and the North American "LIFE-365"[9.74] were designed to model both chloride and carbon dioxide ingress into concrete. Other models dealing with chloride ingress include the South African chloride prediction model[9.75, 9.76] and the Scandinavian model "Clinconc".[9.77, 9.78, 9.80]

The European, Scandinavian and South African models are performance-based approaches, ie they are based on measurement of material properties of the concrete mix or structure under consideration. In these models, the onset of the propagation period is predicted using Fick's second law of diffusion, with which typically chloride penetration profiles can be modelled based on the relevant diffusion coefficient of the concrete, environmental influences and the chloride surface concentration.

Diffusion coefficients of concretes based on various materials and mix proportions are determined through experiments, usually using laboratory-cured concrete at an age of 28 days. For the estimation of chloride diffusion coefficients, various test methods are applied in different parts of the world. The European and Scandinavian models apply the Rapid Chloride Migration (RCM) test[9.79, 9.80] while the chloride conductivity test [9.63, 9.75, 9.76, 9.80, 9.81] is used in South Africa (see also section 9.7.2). Regarding carbonation models, the effective carbonation resistance of concrete is commonly determined through accelerated carbonation tests, in which laboratory-cured specimens are tested under defined conditions at a selected reference time.[9.70]

In contrast to these performance-based service life models, the North American "LIFE-365" is based solely on computer simulations and does not involve testing. The service life and life-cycle costs of reinforced concrete structures are estimated based on certain input parameters such as mix proportions, constituent materials, preventative measures (corrosion inhibitors, coatings, stainless or epoxy-coated steel) and environmental conditions.

9.6.5 Test methods for concrete cover properties

Since the service life of reinforced concrete structures largely depends on the quality of the cover zone, it is necessary to have reliable tests to quantify this. These are briefly covered below. They include accelerated tests which have been designed to provide useful design information in time periods appropriate for project specifications.

Gas permeability tests

Gas permeability tests are commonly applied to characterise the concrete's ability to resist carbonation and comprise the measurement of gas flow through a concrete sample under an externally applied pressure gradient. The test results can be linked either to analytical models for the estimation of diffusion coefficients, or empirically related to the rate of carbonation. Empirical relationships are established either through concrete samples that are exposed to accelerated carbon dioxide ingress (carbonation chamber), or through long-term studies on in-situ structures. One of the first accepted test methods was the Cembureau test[9.82], which is sometimes applied as a reference laboratory test to calibrate in-situ permeability tests.[9.83] Newer methods comprise non-destructive techniques for permeability testing, such as the Torrent Permeability Tester, which is described in the following. The South African Oxygen Permeability Test is discussed in section 9.7.2.

The Torrent Permeability Tester

In some European countries, air (gas) permeability characteristics are determined in-situ using the Torrent Permeability Tester, TPT.[9.84] The distinctive characteristic features of the Torrent Permeability Tester test equipment are a two-chamber vacuum cell and a regulator that balances the pressure in the inner (measuring) chamber and in the outer (guard-ring) chamber (Figure 9.15). The outer guard ring prevents air from the surrounding areas from flowing into the pressure measurement chamber, which would influence the test results. During the test, the cell is placed on the concrete surface and a vacuum is produced with the pump. Due to the external atmospheric pressure and the rubber rings the cell is pressed against the surface and thus both chambers are sealed. The rate at which the pressure increases in the inner chamber is recorded and related to the permeability of the underlying concrete

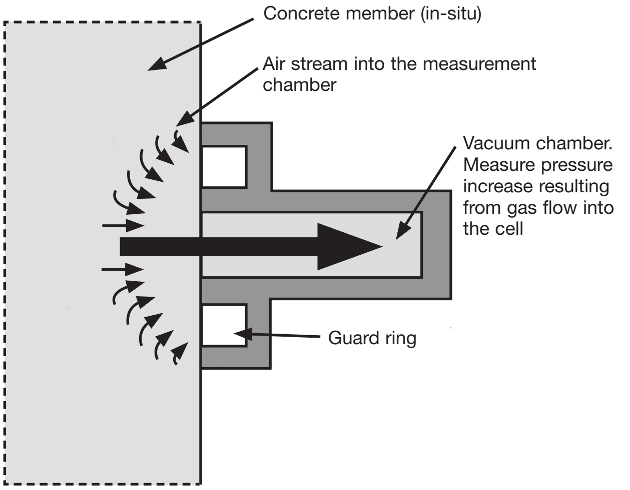


Figure 9.15: Detail of air-flow into the vacuum cell of the Torrent Permeability Tester

Accelerated chloride tests

Diffusion is the primary means of chloride ingress, often allowing chloride ions to reach the level of the reinforcing steel within the life span of the structures. As diffusion is a slow process, chloride tests have been developed to accelerate the diffusion process. This is necessary for efficient quality control during construction as well as for characterisation of concrete mixes in the design stage. The test results are commonly related to the chloride diffusion coefficient of the concrete, which can be used as an input parameter in service life models. For design purposes, tests are usually carried out at a sample age of 28 days. Various test methods, guidelines for their application, as well as common limitations are discussed in the literature.[9.80, 9.83, 9.85] The Bulk Diffusion Test, which is one of the most common and internationally accepted chloride test methods is briefly discussed in the following. The South African chloride conductivity test is discussed in section 9.7.2.

Bulk Diffusion Test (ASTM C1556 – 04)[9.86]

In the bulk diffusion test[9.79], samples are saturated with calcium hydroxide solution, sealed on all sides except the top face, and submerged in a 2.8 M NaCl solution (Figure 9.16), for a minimum of 35 days. After this time chloride profiling is done, by grinding layers of the sample and measuring the chloride at different depths. The depths at which the chlorides are measured depend on the water: binder ratio of the concrete. The chloride concentration profile is then used to determine the diffusion coefficient and surface concentration values.

Depending on the quality of the concrete the test may need to be extended up to 90 days (excluding the initial 28-day curing time), and longer for research purposes. The bulk diffusion test is the most fundamental test and is not affected by the unknown implications of using an electrical field to accelerate the diffusion process. As the test is time-consuming it is generally not suitable for quality assurance purposes; however it has been used to calibrate other test methods. This test is best suited for long term research projects.

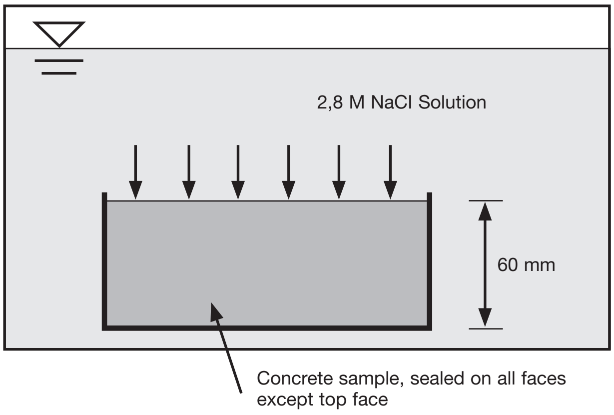


Figure 9.16: Schematic of the bulk diffusion test [9.86]

Cover depth measurements and specifications

In South African and European design codes, the nominal cover to reinforcement comprises a minimum cover plus a variable margin for construction tolerances. The minimum cover commonly relates to durability specifications as outlined earlier. In Europe, the additional margin for surfaces cast against formwork is typically in the range of 5 to 15 mm. The designer selects the margin taking into account consequences of low cover, the level of workmanship expected on site and the quality control procedures.[9.49] Tolerance margins for South African construction practice are proposed at 10 mm for precision work, 15 mm for normal in-situ work, and 20 mm for heavy civil work.[9.88] The most common method of measurement of cover thickness is the use of non-destructive covermeters. Comparative tests carried out by RILEM TC189-NEC demonstrated that modern covermeter devices are suitably accurate and produce reliable results, particularly when performed by well-trained operators.[9.83, 9.87] Cover measurements are relatively simple to perform, are commonly non-destructive, and can provide valuable insight into the quality of construction at comparatively low cost. Durability problems resulting from low cover depths can therefore largely be prevented through appropriate quality control measures during construction.

9.7 Performance-based durability testing, design and specification in South Africa

9.7.1 General

In recent years, an approach to improving the durability of reinforced concrete construction has been developed in South Africa. The philosophy involves the understanding that durability will be improved only when measurements of appropriate cover concrete properties can be made. Such measurements must reflect the in-situ properties of concrete, influenced by the dual aspects of material potential and construction quality. Key stages in formulating this approach were developing suitable test methods, characterising a range of concretes using these tests, studying in-situ performance, and applying the results to practical construction. The approach links so-called durability index (I) parameters, service life prediction models, and performance specifications. Concrete quality is characterised in-situ and/or on laboratory specimens by use of durability index tests for oxygen permeation, water absorption, and chloride conductivity.[9.45, 9.64, 9.89 - 9.94] The service life models in turn are based on the relevant DI parameter, depending on whether the design accounts for carbonation-induced or chloride-induced corrosion. Designers and constructors can use the approach to optimise the balance between required concrete quality and cover thickness for a given environment and binder system.

A framework for the development and application of performance-based specification methods for concrete durability is illustrated schematically in Figure 9.17.

In view of the various options for service life design presented by fib[9.70] (see section 9.6.4), the South African Durability Index approach to date has adopted the strategy of providing deemed to satisfy rules, which limit durability index values and cover depths for given environmental classes and selected binder types. Limiting durability index values are based on service life models, both of which can also be related to partial factor design and to full probabilistic modelling of relevant parameters.[9.95] The South African approach therefore aims at combining Options 1, 2, and 3, as presented by fib, to provide a practical tool for durability design and quality control.

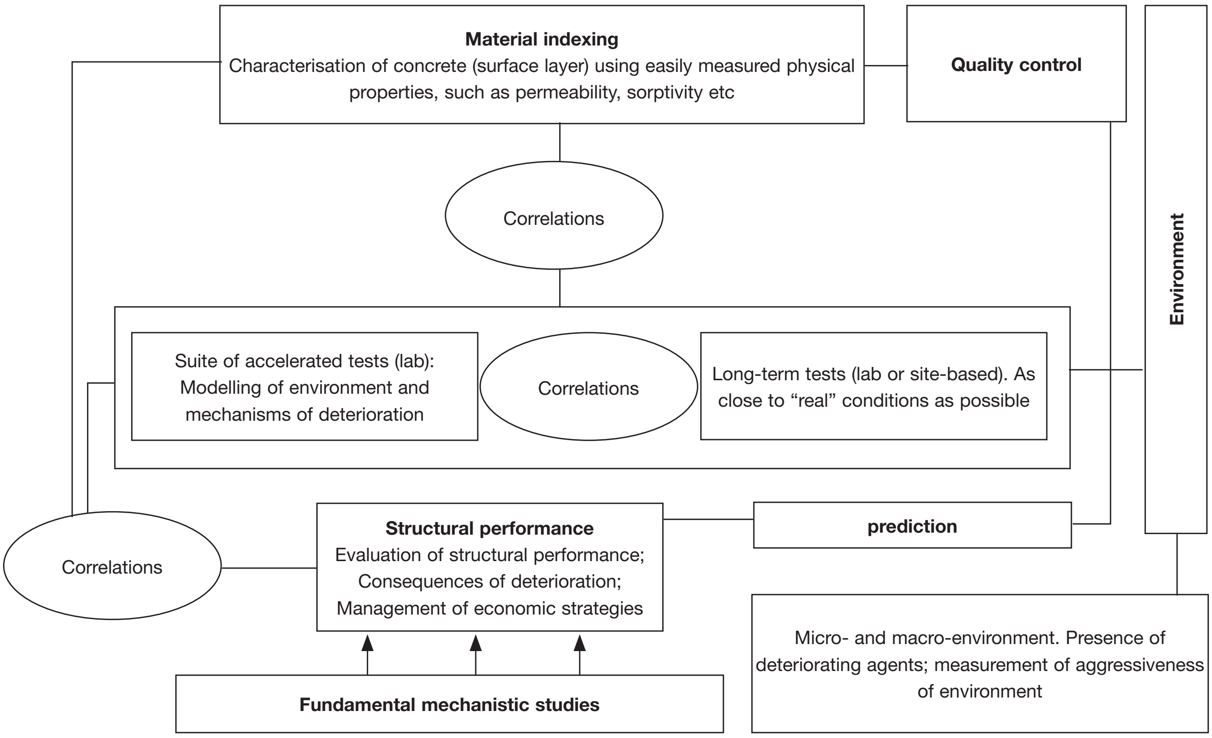


Figure 9.17: Framework for carrying out durability studies

9.7.2 Durability index test methods

General

The Durability Index test methods comprise oxygen permeability, chloride conductivity and water sorptivity. The concrete surface layer is most affected by curing initially, and subsequently by external deterioration processes. These processes are linked with transport mechanisms such as gaseous and ionic diffusion and water absorption. Each index test therefore is linked to a transport mechanism relevant to a particular deterioration process. The tests have been shown to be sensitive to those material, constructional, and environmental factors that influence durability, and they provide reproducible engineering measures of the microstructure of concrete.[9.46, 9.64, 9.65, 9.90, 9.91, 9.93] The tests characterise the quality of concrete as affected by choice of material and mix proportions, placing, compaction and curing techniques, and environment. Test equipments and test procedures are described in detail in the literature[9.96, 9.97] and basic principles are discussed in the following.

Oxygen permeability test

The Oxygen Permeability Index (OPI) test method consists of measuring the pressure decay of oxygen passed through a 30-mm thick slice of a (typically) 68 to 70-mm diameter core of concrete placed in a falling head permeameter (Figure 9.18). The oxygen permeability index is defined as the negative log of the coefficient of permeability. Common OPI values for South African concretes range from 8,5 to 10,5, a higher value indicating a higher impermeability and thus a concrete of potentially higher quality. Oxygen permeability index is measured on a log scale, therefore the difference between 8,5 and 10,5 is substantial. An empirical prediction model for carbonation was formulated using the oxygen permeability test. Using this approach, 50-year carbonation depths may be predicted for different environments.

The oxygen permeability test assesses the overall micro- and macrostructure of the outer surface of cast concrete, and is particularly sensitive to macro-voids and cracks which act as short-circuits for the permeating gas. Thus the test is very useful to assess the state of compaction, presence of bleed voids and channels, and the degree of interconnectedness of the pore structure. Correlations between oxygen permeability index values recorded at 28 days and carbonation depths after natural exposure have been found to be good (Figure 9.19).[9.91] A particular OPI value can be obtained either by extending the duration of curing of low strength concrete or by decreasing the w:b ratio in the event that curing is likely to be minimal or ineffective. Figure 9.20 shows typical test results for Oxygen Permeability Index values, measured on water-cured samples at 28 days of age, in relation to w:b ratio and binder type.[9.99]

In a comparative international study of various test methods for durability indicators, the OPI test was found to be able

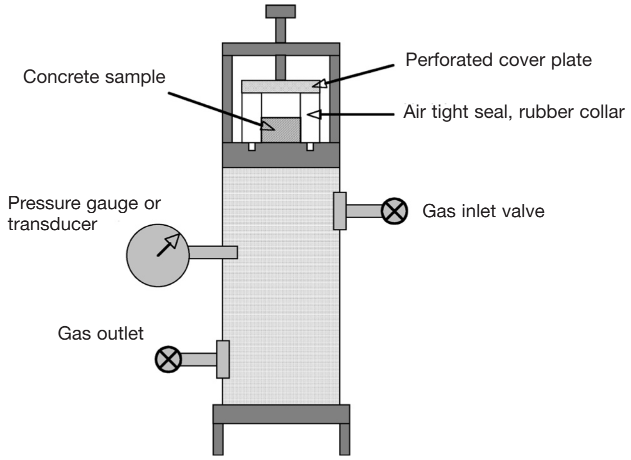


Figure 9.18: Schematic of an Oxygen Permeameter

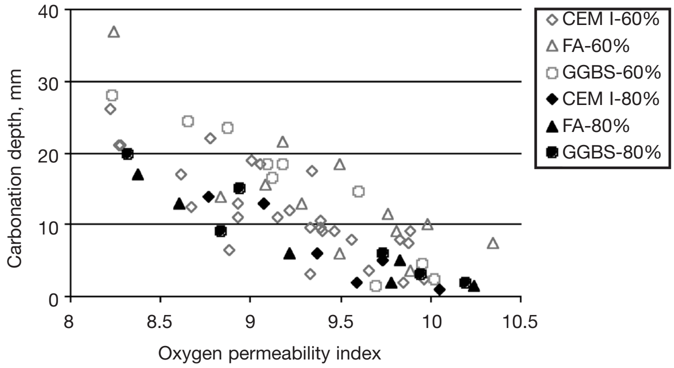
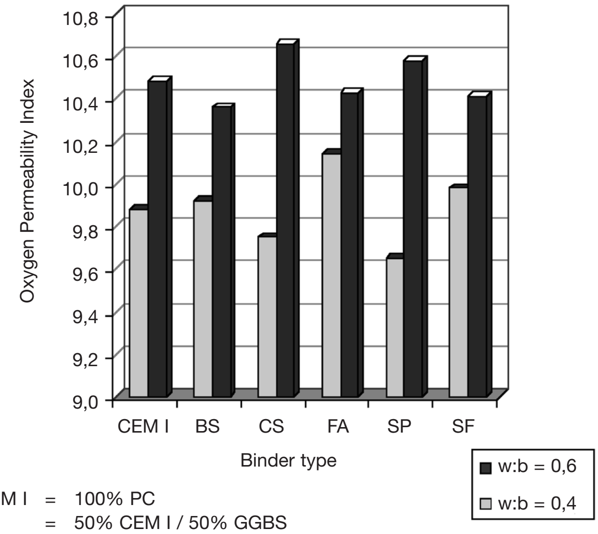


Figure 9.19: Carbonation depth in various concretes versus oxygen permeability index (measured at 28 days) for four years exposure at an average relative humidity of 60% or 80%.[9.91]



CEM I = 100% PC
BS = 50% CEM I / 50% GGBS
CS = 50% CEM I / 50% GGCS
FA = 70% CEM I / 30% FA
SP = 85% CEM I / 15% Superpozz (FA)
SF = 93% CEM I / 7% CSF

Figure 9.20: Typical test results for Oxygen Permeability Index values, measured on water-cured samples at 28 days of age, in relation to w:b ratio and binder type[9.98]

to detect differences in w:b ratio, binder type, and curing condition on a statistically highly significant level.[9.83, 9.85, 9.99] The same study revealed that results obtained with the OPI test equipment correlate well with other existing test methods for oxygen permeability, such as the Cembureau method and the Torrent Permeability Tester.

Chloride conductivity test

Intrinsic diffusion tests, where concrete is exposed to high and low concentrations of the diffusing species on opposite faces, have been successfully used to measure the coefficient of diffusion. However, diffusion is a slow process, even when using high concentration gradients, and these tests may take months or even years to yield useful results. Accelerated diffusion tests using an applied potential difference have therefore been developed to obtain more rapid results in the laboratory.

The South African chloride conductivity test apparatus (Figure 9.21) consists of a two-cell conduction rig in which concrete core samples are exposed on either side to a 5M NaCl chloride solution. The core samples are preconditioned before testing to standardise the pore water solution (oven-dried at 50°C followed by 24 hours vacuum saturation in a 5M NaCl chloride solution). The movement of chloride ions occurs due to the application of a 10 V potential difference. The chloride conductivity is determined by measuring the current flowing through the concrete specimen. The apparatus allows for rapid testing under controlled laboratory conditions and gives instantaneous readings.

Chloride conductivity decreases with the addition of FA, GGBS, and CSF in concrete, extended moist curing and increasing grade of concrete. Portland cement concrete for instance generally has high conductivity values with only high-grade material achieving values below 1,0 mS/cm. GGBS or FA concretes in contrast have significantly lower chloride conductivity values. While the test is sensitive to construction and material effects that are known to influence durability, results are specifically related to chloride ingress into concrete. Correlations between 28-day chloride conductivity results and diffusion coefficients after several years marine exposure have shown to be good over a wide range of concretes.[9.75] Typical test results for chloride conductivity values, measured on laboratory-cured samples at 28 days of age, are presented in Figure 9.22.[9.98]

An international study revealed that the chloride conductivity test is able to detect differences in w:b ratio, binder type, and curing condition on a statistically highly significant level and that test results obtained with the chloride conductivity method generally correlate well with results obtained from other accepted test methods for chloride resistance.[9.83, 9.85, 9.99]

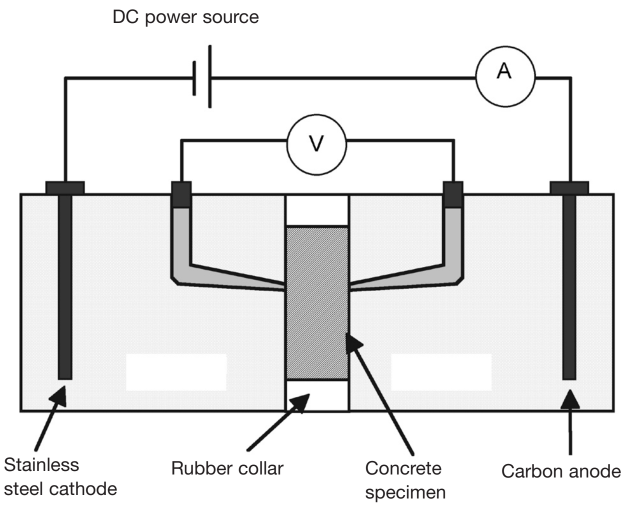
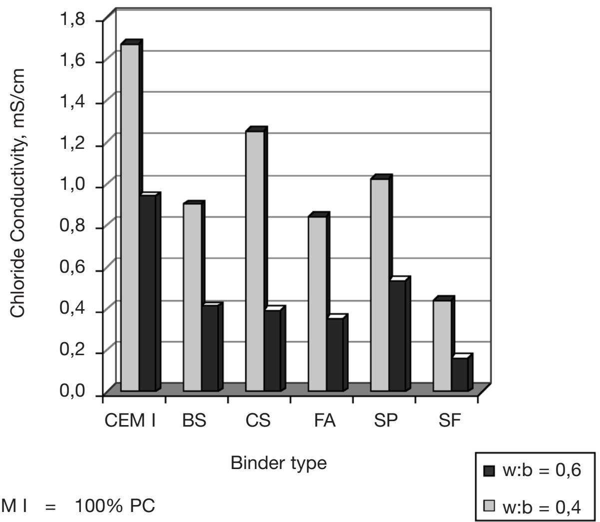


Figure 9.21: Schematic of chloride conductivity cell

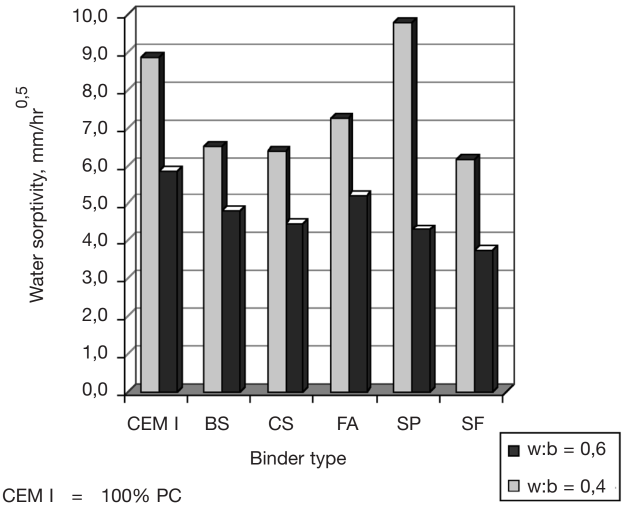


CEM I = 100% PC
BS = 50% CEM I / 50% GGBS
CS = 50% CEM I / 50% GGCS
FA = 70% CEM I / 30% FA
SP = 85% CEM I / 15% Superpozz (FA)
SF = 93% CEM I / 7% CSF

Figure 9.22: Typical test results for chloride conductivity values, measured on water-cured samples at 28 days of age, in relation to w:b ratio and binder type[9.98]

Water sorptivity test

Several bulk absorption tests have been developed for concrete in which the specimen is immersed in water and the total mass of water absorbed is used as a measure of the absorption. These tests merely measure the porosity of the concrete but cannot quantify the rate of absorption and do not distinguish between surface and bulk effects. In South Africa, a modified version of Kelham's sorptivity test was chosen as a compromise between accuracy and ease of use.[9.100, 9.101] Concrete samples (usually cores 68 to 70 mm diameter, 30 mm thick) are initially preconditioned at 50°C to ensure uniformly low moisture contents at the start of the test. The same specimens that were used in the oxygen permeability



CEM I = 100% PC
BS = 50% CEM I / 50% GGBS
CS = 50% CEM I / 50% GGCS
FA = 70% CEM I / 30% FA
SP = 85% CEM I / 15% Superpozz (FA)
SF = 93% CEM I / 7% CSF

Figure 9.23: Typical test results for water sorptivity, measured on water-cured samples at 28 days of age, in relation to w:b ratio and binder type.[9.98]

test may be used in the water sorptivity test. The circular sides of the core are sealed to ensure uni-directional absorption from the bottom (usually exposed) face. The concrete samples are exposed to a few millimetres of water with the test surface facing downward. At regular time intervals, the specimens are removed from the water and the mass of water absorbed is determined using an electronic balance. Measurements are stopped before saturation is reached and the concrete is then vacuum-saturated in water to determine the porosity. A linear relationship is observed when the mass of water is plotted against the square root of time, and the sorptivity of the concrete is determined from the slope of the straight line produced. The water sorptivity test has been found to be very sensitive to the nature and extent of early curing of the cover concrete, and can thus be used to assess construction quality.

Typical test results for water sorptivity values, measured on laboratory-cured samples at 28 days of age, are presented in Figure 9.23.[9.98]

9.7.3 Application of the durability index approach

General considerations

The sensitivity of the South African index tests to material and constructional effects makes them suitable tools for site quality control. Since the different tests measure distinct transport mechanisms, their suitability depends on the property being considered. Durability index testing may be used to optimise materials and construction processes where specific performance criteria are required. At the design stage the influence of a range of parameters such as materials and construction systems may be evaluated in terms of their impact on concrete durability.

The durability indices, obtained with the above test methods, have been related empirically to service life prediction models. Two corrosion initiation models have been developed, related to carbonation- and chloride-induced corrosion. The models derive from measurements and correlations of short-term durability index values, aggressiveness of the environment and actual deterioration rates monitored over periods of up to ten years. The models allow for the expected life of a structure to be determined based on considerations of the environmental conditions, cover thickness and concrete quality.

Index values can be used as the input parameters for these service life models, together with other variables such as steel cover and environmental class, in order to determine rational design life. The environmental classes are related to the EN 206 classes as modified for South African conditions (Table 9.2) while concrete quality is represented by the appropriate durability index parameter. The oxygen permeability index is used in the carbonation prediction model, while the chloride model utilises chloride conductivity. The service life models can also be used to determine the required value of the durability parameter based on predetermined values for cover thickness, environment, and expected design life. Alternatively, if concrete quality is known from the appropriate DI, a corrosion-free life can be estimated for a given environment. Limiting index values can be used in construction specifications to provide the necessary concrete quality for a required life and environment. Thus, a framework has been put in place for a performance-based approach to both design and specification.[9.68, 9.94]

Table 9.2: Environmental classes (natural environments only) (after EN 206)[9.49]

Carbonation-induced corrosion	
Designation	**Description**
XC1	Permanently dry or permanently wet
XC2	Wet, rarely dry
XC3	Moderate humidity (60-80%) (Ext. concrete sheltered from rain)
XC4	Cyclic wet and dry
Corrosion induced by chlorides from sea water	
Designation	**Description**
XS1	Exposed to airborne salt but not in direct contact with sea water
XS2a	Permanently submerged
XS2b*	XS2a + exposed to abrasion
XS3a	Tidal, splash and spray zones Buried elements in desert areas exposed to salt spray
XS3b*	XS3a + exposed to abrasion

* These sub-clauses have been added for South African coastal conditions

Specifying durability index values

Two possible approaches to specifying durability index values are a deemed-to-satisfy approach and a rigorous approach. The former is considered adequate for the majority of reinforced concrete construction and represents the simpler method in which limiting DI values are obtained from a design table, based on binder type and exposure class, for a given cover depth (50 mm for marine exposure and 30 mm for carbonating conditions).[9.68, 9.94] The rigorous approach will be necessary for durability–critical structures, or when the design parameters assumed in the first approach are not applicable to the structure in question. Using this approach, the specifying authority would use the relevant service life models developed in South Africa. The designer can use the models directly and input the appropriate conditions (cover depth, environmental classification, desired life, and material). The advantage of this approach is its flexibility as it allows the designer to use values appropriate for the given situation rather than a limited number of pre-selected conditions. The rigorous approach allows a trade-off between material quality (ie durability index value) and concrete cover, with lower quality allowable when cover is greater.

As-delivered concrete quality vs as-built concrete quality

A clear distinction must be drawn between material potential and in-situ construction quality. Although specifications are usually only concerned with as-built quality, the processes by which such quality is achieved cannot be ignored. There are two distinct stages and (possibly) responsibilities in achieving concrete of a desired quality. The first is material production and supply, which could be from an independent party such as a readymix supplier. A scheme for acceptance of the as-supplied material must be established so that the concrete supplier can have confidence in the potential quality of the material. The second stage is the responsibility of the constructor in ensuring that the concrete is placed and subsequently finished and cured in an appropriate manner. It is ultimately the as-built quality that determines durability and the constructor has to take the necessary steps and precautions in the construction process to ensure that the required quality is produced. If the as-built quality is found to be deficient, the specification framework must have an internal acceptance scheme that is able to distinguish whether the deficiency arises from the as-supplied material or the manner in which it was processed by the constructor. To enable this, a two-level quality control system has been proposed in South Africa, with testing of both material potential and as-built quality.[9.68, 9.94] Material potential is represented by as-supplied concrete specimens with a laboratory-controlled wet curing period, while as-built quality is determined using in-situ sampling of concrete members. As a general rule, concrete in the as-built structure may be of lower quality compared with the same concrete cured under controlled laboratory conditions. To account for the improved performance of laboratory concrete over site concrete, the characteristic values for the durability indexes of the laboratory concrete are adjusted to the potentially higher quality achievable.

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Chapter 10
Alkali-silica reaction

Bertie Oberholster

10.1 Introduction

The reaction between alkalis and certain mineral constituents in some aggregates may result in expansion within concrete and consequent cracking and distress of elements made with it. Concrete affected in this way becomes unsightly and its engineering properties tend to deteriorate. Reinforcement in such concrete is less well protected against corrosion. In addition, indirect effects such as deterioration of serviceability may become apparent.

Alkali-aggregate reaction (AAR) was first recognised in the USA in 1938 and reported on by Stanton[10.1, 10.2]. The first International Meeting on the subject which was held in Køge, Denmark in 1974 was attended by less than 30 delegates and the proceedings were not published. The 13th International Conference on Alkali-Aggregate Reaction in Concrete (ICAAR), which was held in Trondheim, Norway in June 2008, was attended by 172 delegates from 30 countries. The serious impact that this mechanism of deterioration has on the durability of concrete and the consequent costs involved in the rehabilitation or replacement of affected structures or the premium that has to be paid on precautionary measures against the reaction occurring in new structures have been recognised for many decades in all parts of the world.

In South Africa the problem was first identified in structures in the Cape Peninsula in the 1970s[10.3] and was extensively investigated. The results of the extensive laboratory and field investigations conducted in South Africa over a period of 15 years were included for the first time in the Sixth (revised) edition of Fulton's Concrete Technology (1986). The nature of science is that it is not static and new ideas on several aspects of ASR have evolved since 1986. Unfortunately, in South Africa basic research on alkali-silica reaction (ASR) has more or less stagnated beyond the end of the 1980s, with the result that this fourth version of the chapter contains changes necessitated mainly by alterations in national specifications, new types of cement introduced to the South African market, information gleaned from on-going investigation of structures affected by ASR and the results of international research.

This chapter:
- Gives definitions of the terms used in describing ASR.
- Indicates the incidence of the phenomenon in South Africa.
- Discusses the mechanism of expansion and cracking and factors influencing ASR.
- Gives guidelines for minimising the risk of ASR.
- Gives guidelines for the investigation of structures and the diagnosis of ASR.
- Proposes remedial measures for structural elements affected by ASR.

10.2 Reactions between alkalis and aggregate: terms used

Various terms are used to describe deleterious reactions between alkalis and aggregate in concrete. These terms are explained below.

Alkali-silica reaction

This takes place between the alkaline pore solution of concrete and metastable forms of silica such as opal, trydimite, cristobalite and volcanic glasses that are found in some aggregates, to produce expansive alkali-silica gel.

Alkali-silicate reaction

This term was originally introduced to draw a distinction between the previous type of reaction and the reaction of the alkaline pore solution with aggregates such as greywacke and argillite found in Nova Scotia, Canada. It was claimed that exfoliation and water uptake by certain phyllosilicates (clay minerals) play a large part in the alkali-expansion of such rock types.[10.4] However, there does not appear to be any evidence that clay minerals are involved in the expansion, and the basic reaction seems to be the same in both cases. Terminology that has been used for the expansive reaction between the alkaline pore solution and chert, chalcedony, cryptocrystalline quartz, microcrystalline quartz or strained quartz in rocks such as quartzite, greywacke, argillite, hornfels, phyllite, granite, granite-gneiss, granodiorite, etc. is alkali-silicate reaction, alkali-siliceous rock reaction, alkali-quartz-bearing rock reaction and alkali-quartz-rock reaction. However, there appears to be little merit in using a term other than alkali-silica reaction for all the types of alkali-aggregate reaction mentioned above.

Alkali-carbonate-rock reaction

This expansive reaction occurs between the alkaline pore solution and certain argillaceous dolomitic limestones and argillaceous calcitic dolostones.[10.5, 10.6] No alkali-silica gel is formed. The harmful expansion is believed to be the result of a dedolomitisation reaction that occurs when the alkali hydroxides react with small dolomite crystals in the clay matrix. In addition, moisture uptake by dry clay material may also cause expansion of certain dolomitic limestones which have been opened up by the dedolomitisation reaction. This type of reaction has not yet been encountered in South Africa, although, dedolomitisation of travertine dimension stone has been observed by the author.

While in South Africa the term alkali-aggregate reaction is used by some to mean ASR, the latter term will be used throughout in the text that follows.

10.3 Alkali-silica reaction in South Africa and Namibia

South Western Cape

The highest incidence of ASR in South Africa occurs in the Cape Peninsula and environs, and the aggregates involved are from the Malmesbury Group. The structures affected are, amongst others, a large number of bridges, a concrete highway, several storage dams, a hydro-electric power station, sports stadiums, a large concrete dome structure, large-diameter prestressed water reticulation pipes, reservoirs, warehouse columns and pile caps, the hardstanding slab of a container terminal and electric mast foundations.

Further afield, prestressed concrete railway sleepers on the Sishen-Saldanha line containing granite aggregate and an irrigation dam containing Bokkeveld Group aggregates have been affected.

Eastern Cape

On a smaller scale, some exposed concrete structures in the Eastern Cape have been affected by the reaction, and the aggregates involved are Table Mountain Group quartzite, Enon Formation quartzite pebbles and Quaternary gravels. Structures affected are several irrigation and water storage dams, an irrigation water concrete pipe line, bridges, an airport apron and airport building, pile caps, retaining walls and electric mast foundations.

Gauteng and Free State

ASR has affected structures such as reservoirs, bridges, reinforced concrete portal frames of a highway and an airport runway in this area, some seriously. In all cases the aggregates used contained quartzite and shale of the Witwatersrand Supergroup.

Mpumalanga

In the Hoedspruit area concrete taxiways and mast footings containing granite coarse aggregate cracked badly as a result of ASR (Confidential report by R E Oberholster, Cembeton Consult, August 2008).

Namibia

Concrete mast foundations at Swakopmund, Namibia have been affected by ASR (Confidential report by R E Oberholster, Cembeton Consult, July 2008). The coarse aggregate was reported to be from a granite quarry at Walvis Bay.

10.4 Mechanism of expansion and cracking

When the alkaline pore solution of concrete reacts with reactive forms of silica minerals in the aggregate, an alkali-silica gel forms; this gel can imbibe water and swell. When the tensile stress due to the swell pressure of the gel exceeds the tensile strength of the concrete, typically by 4 MPa, cracking occurs.

10.5 Factors that influence alkali-silica reaction

Reaction leading to deleterious expansion may occur only if all of the following three factors are present in the concrete:

- A sufficiently high alkalinity of the pore solution
- A sufficient amount of deleteriously reactive mineral(s) in the aggregate
- Environmental conditions that will promote the reaction

Not only must all three factors be present simultaneously but, for some aggregates, they must be present in a specific combination. Each of these factors is discussed in the following sections.

10.5.1 Alkalis

It is customary to refer to the alkalis in cement in terms of sodium oxide equivalent [$\%Na_2O\text{-eq} = \%Na_2O + (0.658 \times \%K_2O)$]. The alkali content of SANS 50197-1[10.7] common cements is usually specified as the percentage acid-soluble Na_2O-eq by mass of cement. When considering concrete as a whole, the alkali content is expressed in terms of kilograms per cubic metre, ie kg Na_2O-eq/m^3 of concrete.

The main source of the alkalis responsible for a pore solution sufficiently alkaline to initiate and sustain the reaction, is usually, but not always, the cement. Alkalis are normally present in the cement as the neutral sulphates, Na_2SO_4, K_2SO_4 or the mixed salt $(Na,K)_2SO_4$, which are highly soluble. After mixing, the pore solution of concrete made with a high-alkali cement contains almost entirely Na^+, K^+ and OH- ions with very low concentrations of Ca^{2+}, $SO_4{}^{=}$ and Cl^-, and the pH of the pore solution is in the range 13 to 14, depending on the alkali level. If no or very little alkali is present, then the pH of the pore solution is between 12,5 and 12,9. In addition, alkalis may be derived from the reaction of calcium hydroxide produced by cement hydration with alkali-containing minerals in the aggregate, or from outside sources such as mixing water, sea water, sea spray, aggregates containing salts and chemical admixtures such as sodium lignosulphonate.

Based on the work done in the USA between 1940 and 1960, portland cement having an Na_2O-eq of more than 0,6 %, was for many years generally regarded as a high-alkali cement and one which might possibly be unsuitable when used in combination with a potentially alkali-reactive aggregate. In general, however, it is the product of the alkali content of the cement and the cement content of the concrete that determines the alkalinity of the pore solution and, therefore, to what extent reaction with a reactive aggregate will occur. It is, therefore, more appropriate to limit the alkali content per m^3 of the concrete than to limit the alkali content of the cement. Clearly, very rich mixes of high cement content will include greater quantities of alkali and be more likely to cause expansion when used with reactive aggregate than normal mixes having 300 kg or less cement per cubic metre of concrete. Therefore, in South Africa the lower limit of alkali content where deleterious expansion of concrete no longer happens, is expressed as kilogram of Na_2O-eq per cubic metre. This limit has been found to be different for different aggregates.[10.8]

During hydration of the cement, only some of the total alkalis dissolve in the pore solution and are available for reaction with the aggregate; these are referred to as the active alkalis. The proportion of active alkali varies from cement to cement but is, for South African CEM I cements, on the average about 80 % of the total Na_2O-eq of the cement. Since about 1985, most CEM I cements in South Africa have had an Na_2O-eq of less than 0,6 %, with some even lower than 0,3 %. However, since about 1998 the alkali content has tended to increase, in some cases to between 0,6 and 0,8 % Na_2O-eq.

In the case of blended cements, the difference between the total and active alkali content can be greater compared to CEM I cement. This is discussed in more detail in section 10.7.2.

10.5.2 Aggregate type

The stone or sand or both of these ingredients in the mix can be reactive. For the reaction to occur and deleterious expansion to take place, the aggregate must contain sufficient alkali-reactive constituent and, in addition, must be dense. Porous aggregates tend to provide enough void space within them to accommodate the expansive gel that develops.

Both the amount of reactive constituent as well as the reactivity of the constituent influence, the amount or rate of expansion. For example in the ASTM C 227-03 test method[10.9] it was found that as little as 2 % of opal in sand resulted in deleterious expansion, while a minimum of about 20% of Malmesbury metasediment (hornfels, greywacke, phyllite) in the sand was required for deleterious expansion to occur.[10.10]

In South African case studies it appears that reactions mostly take place inside the aggregate and the reactive silica must be accessible to the pore solution. The location of the reaction therefore relates to the accessibility of the reactive material to the alkalis and this may occur through the presence of planes of weakness generated during crushing or through a natural fracture. The total surface area exposed to alkalis also plays a predominant role, eg compare the surface areas of the following materials: opal, 40 000 m^2/kg; microcrystalline flint, 220 m^2/kg.[10.11]

A characteristic of ASR involving an aggregate such as opal is that for each cement with a different alkali content, maximum expansion is observed at different opal content. Thus for each particular cement, expansion increases differently with an increase in opal content until a maximum expansion is reached and then decreases again with further increase in opal content. The opal content at which maximum expansion occurs will be different for each particular cement. Likewise, for a specific opal content, expansion increases with an increase in alkali content until a maximum is reached and then decreases again. The opal or alkali content at which maximum expansion occurs is called the pessimum opal or alkali content. The concept of the pessimum content is illustrated in Figure 10.1. A pessimum effect has not yet been encountered with South African aggregates.

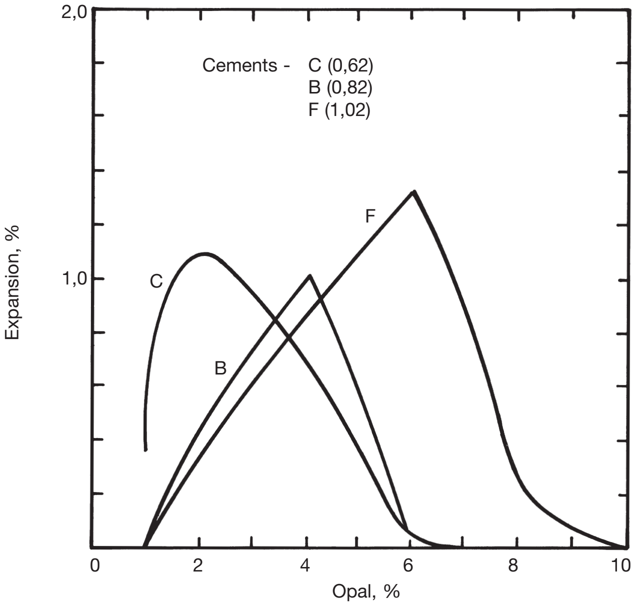

Figure 10.1: Illustration of the concept of pessimum content for mortar prisms where a percentage of the minus-300-μm-plus-150-μm fraction of the sand was replaced by opal

In South Africa and Namibia, certain aggregate sources have been identified as potentially alkali reactive, both through service record and by laboratory testing. They are the following:

South Western Cape

- Malmesbury Group – hornfels, spotted hornfels, quartz-albite-epidote hornfels, meta-greywacke, quartzite, mylonite, phyllite, sandstone, lava
- Cape Granite Suite – very rarely
- Cape Super Group – occasionally orthoquartzite of the Table Mountain Group; arkose of the Bokkeveld Group
- Quaternary Period – occasionally river gravels

Eastern Cape

- Cape Super Group – orthoquartzite of the Table Mountain Group
- Enon Formation – quartzite pebbles
- Quaternary Period – quartzite pebbles

Natal

- Natal Group – quartzite, sandstone (no deleterious service record)
- Karoo Sequence – tillite of the Dwyka Formation (no deleterious service record)

Gauteng and Free State

- Witwatersrand Super Group – quartzite, shale
- Dolomite Series – more than 20% chert (no deleterious service record)

Mpumalanga

Archaean granite and gneiss (based on aggregate identified in cores from structures affected by ASR)

Namibia

Archaean granite and gneiss (based on aggregate identified in cores from structures affected by ASR).

Not only is there a great difference in the reactivity of aggregates from different geological formations, but the reactivity of aggregates from the same geological formation or even from the same quarry can differ widely. It is important to realise that an aggregate belonging to one of the above rock types should not be accepted to be alkali reactive, unless the particular source has been proven to be so through its service record or through laboratory testing. Conversely, if a siliceous aggregate does not belong to one of the above types, it should not be accepted to be innocuous in the presence of a highly alkaline pore solution, unless proven by service record or laboratory testing to be non-reactive.

The reactive constituents in South African aggregates are probably strained quartz as identified by features such as deformation bands, deformation lamellae and extinction bands and microcrystalline quartz.

10.5.3 Environmental conditions

A properly compacted concrete, made with an alkali-reactive aggregate and cement providing the required concentration of alkali in the pore solution, will expand and crack only if the environmental conditions to which it is exposed are conducive. The effects of temperature, moisture and climate are discussed below.

Temperature

In general, for laboratory conditions, there is an increase in expansion rate as the storage temperature is increased to about 60ºC. Under field conditions, the rate of expansion can be expected to approximately double for each 10ºC increase in the mean annual ambient temperature.

Moisture

The amount of mixing water in concrete is generally sufficient for reaction to occur and, provided that the concrete does not lose water and dry out, to be maintained. It has been suggested that the cut-off internal relative humidity of concrete below which deleterious ASR will not occur, is in the range 75 to 85%.[10.12] Any environmental condition that will prevent concrete from drying out and the internal relative humidity dropping below the critical value will be conducive to the reaction occurring, for example, exposure to wetting by rain or sea spray. Condensation that occurs on concrete surfaces due to thermal gradients plays an important role in promoting the reaction in relatively arid environments. It has been observed that concrete pile caps and columns of a warehouse were badly cracked below the floor in the fill area as a result of ASR, while the portion of the columns above the floor was uncracked and ASR was absent after more than 20 years. Similarly, it can be observed that concrete pavements and railway sleepers affected by ASR are less cracked, or even unaffected, where protected from the elements by bridge superstructures.

Evidence of the effect of internal relative humidity is the observation that reaction product is usually only noticed from about 60 mm inwards in cores drilled from exposed concrete structures (see Figure 10.2).

The overall effect of daily and seasonal fluctuations in temperature and moisture conditions on structures affected by ASR is to increase the width and number of cracks and exacerbate the effect of the cracking.

Climate

The effect of climate on the rate of expansion is complex. In an investigation, large specimens of the same concrete mix were exposed out of doors in four different centres in South Africa. Rate of expansion, in descending order, was Durban, Cape Town, Pretoria, Port Elizabeth.[10.13] (Durban has the highest annual rainfall, temperature and relative humidity, Cape Town is a winter-rainfall area and Port Elizabeth has the lowest annual rainfall.)

Figure 10.2: Two halves of a core showing that reaction product does not develop in the outer ca. 60 mm of concrete, apparently because the relative humidity is too low

10.6 Minimising the risk of ASR

10.6.1 General considerations

As discussed in section 10.5, deleterious ASR in concrete can occur only if the alkalinity of the pore solution is sufficiently high, if the aggregate is alkali reactive, and if the concrete is sufficiently moist, ie if the internal relative humidity is more than 75%. It follows, therefore, that if any of these factors can be eliminated, deleterious ASR will not occur.

A number of issues complicate the prescription of precautionary measures against ASR. The measures proposed are usually at least partially based on laboratory data, because quantitative field data are not always available. Additional complications are, for example, the difference in the reactivity of aggregates of the same type, and also the difference in the reactivity of different types of aggregate. There are no clear-cut limits between deleteriously and non-deleteriously alkali-reactive aggregates, or between safe and unsafe alkali concentrations in concrete. Climatic variations and local conditions to which structures and elements are exposed further complicate the issue, for example, some elements of a structure may be exposed to continuously moist conditions while other parts may be dry. The result is that, if everything is taken into account and precautions are prescribed which eliminate all risk, the proposal may be too conservative. It is, therefore, important to assess the likelihood of damage occurring, and the economic consequences of such damage. This will determine whether all risk of ASR has to be eliminated or whether some degree of expansion and cracking is acceptable.

10.6.2 Cement

In assessing the need to control the alkali content of the concrete or to use ground granulated slag (GGBS), silica fume (CSF), fly ash (FA), burnt shale (P) or an admixture such as lithium salt, it is assumed that the specifier has knowledge about the potential alkali reactivity of the aggregate to be used. GGBS will be referred to under the generic term slag (S) from hereon and it will include granulated modified basic oxygen furnace and steel slags, which have been found to be sound and suitable for use as supplementary cementitious material (Confidential report by R E Oberholster, Cembeton Consult, November 2005).

Controlling alkalis in the concrete

A cement having a % Na_2O-eq of not more than 0,6 is generally regarded as a low-alkali cement and one which may be suitable for use with a potentially alkali-reactive aggregate. However, this is a purely arbitrary criterion, which assumes that the contribution of alkali from other sources is small, that the maximum cement content will not exceed about 350 kg/m^3 and that all reactive aggregates are equally susceptible to attack by the alkalis in the pore solution. It is, of course, the total alkali content in the pore solution of the concrete that counts. This is derived predominantly from the hydration of the clinker portion of cement, and is therefore the product of the percentage alkali in the cement and the mass of cement in the concrete. Chemical admixtures (such as accelerators, retarders and workability aids), mix water and sea spray may contribute alkali and this must be taken into consideration. Alkali may also be contributed by sodium- and potassium-bearing minerals in the aggregate by reaction with $Ca(OH)_2$ released during the hydration of cement. However, it is thought that this factor is accounted for in the case of South African research by the fact that the limit of alkali content above which deleterious expansion is found to occur in concrete is different for different aggregates. These limits are given in Table 10.1. It needs to be explained that each value represents the absolute lower limit at which no deleterious expansion was encountered after 52 weeks in the concrete prism test made with a range of aggregates from the same lithostratigraphic unit. For example, in the case of aggregate from the Tygerberg Formation of the Malmesbury Group, nine different aggregates were tested in combination with seven different portland cements (now known as CEM I) with alkali content ranging from 0,16% to 1,30% total Na_2O-eq (0,11% to 1,10% active Na_2O-eq respectively), ie 0,56 kg/m^3 to 4,56 kg/m^3 total Na_2O-eq (0,39 kg/m^3 to 3,85 kg/m^3 active Na_2O-eq, respectively). The results showed that at an alkali content of the concrete of 3,8 kg/m^3 active Na_2O-eq, all the aggregates expanded deleteriously, and at 1,8 kg/m^3 active Na_2O-eq, none expanded deleteriously (see Figure 10.3). The corresponding total Na_2O-eq of the two cements were rounded off to, 4,6 kg/m^3 and 2,2 kg/m^3 respectively.

The other aggregate types in Table 10.1 were not tested as extensively as those of the Tygerberg Formation, and some of the limits, especially those of the Cape Granites, may be found too high after more extensive testing.

For each of the aggregates, therefore, either the maximum permissible cement content for cement with a particular Na_2O-eq, or the maximum Na_2O-eq for a particular cement content, can be calculated. In practice only the total Na_2O-eq of cement is usually known. It is for this reason that the limits given in Table 10.1 are for total Na_2O-eq. The difference between the total alkali content of the cement from which the alkali content of the concrete is calculated, and the active alkali actually available for reaction, provides a small but useful margin of safety for establishing acceptable limits. However, it is important to note that these limits apply only to CEM I cement. For all the other SANS 50197-1[10.7] common cements, the active Na_2O-eq has to be calculated as explained in section 10.8. The addition of Archaean granite from Mpumalanga and Salem granite from Walvis Bay, Namibia is based on the identification of ASR in cores taken from cracked concrete structures containing these aggregates.

If the alkali content of concrete cannot be kept at or below the limits in Table 10.1, one of the supplementary major constituents of SANS 50197-1[10.7] (referred to as extenders in SANS 1491 [10.14] and mineral admixtures in ASTM C 441-05.[10.15]) or a lithium compound as discussed below can be used.

Table 10.1: Limits of total alkali content, derived from the clinker component of cement, per cubic metre, of concrete for South African aggregates from different lithostratigraphic units

Rock type	Total Na_2O-eq, kg/m^3
Witwatersrand Supergroup quartzite, shale	2,0
Dolomite Group chert	2,8
Malmesbury Group metasediments	2,1
Table Mountain Group orthoquartzite	2,8
Bokkeveld Group arkose	2,8
Natal Group quartzite, sandstone	2,8
Dwyka Formation tillite	2,8
Enon Formation quartzite pebbles	2,8
Quarternary quartzite gravels	2,8
Archaean granite	Not determined
Cape granite	4,0
Salem granite	Not determined

Examples of calculating the active alkali content of concreting materials are given in section 10.8.

The use of cement extenders

For the purpose of minimising the risk of ASR in concrete, only three supplementary major constituents (extenders) currently need to be considered in South Africa, namely, S, CSF, and FA.

It has been shown that the reduction in expansion with a reactive aggregate obtained by blending a cement extender with a high-alkali cement is slightly greater than can be ascribed to a mere dilution effect, ie reduction in active alkali content.[10.16] To prevent deleterious expansion when using an alkali-reactive aggregate in combination with high-alkali cement, the cementitious material should contain an extender complying with SANS 1491[10.14] using a minimum of the following material by mass:

- at least 40% S by mass

or

- at least 15% CSF by mass

or

- at least 20% FA by mass

Percentages refer to the total mass of the cement. It is recommended that, where the Na_2O-eq of CEM I cement exceeds 1%, the S or FA be increased by 10 percentage points. There is some doubt about the long-term effectiveness of CSF in preventing ASR with a very-high-alkali cement.[10.17] Factors other than ASR also have to be considered when CSF is used as an ameliorating measure. The use of 15% CSF by mass of concrete will probably lead to stickiness of the fresh mix, increase the water requirement and require the use of a superplasticiser.

The use of chemical admixtures

McCoy and Caldwell,[10.18] using Pyrex glass as the reactive aggregate in mortar bar tests at 38°C, established in 1951 that the most promising chemicals to control expansion due to ASR were various lithium-containing compounds ($LiCl$, LiF, $LiNO_3$, Li_2CO_3, Li_2SiO_3, and Li_2SO_4), provided they were used in sufficient quantity. In the last 15 years, several researchers have demonstrated that the addition of lithium compounds can effectively control ASR in concrete.[10.19-10.23] Lithium nitrate ($LiNO_3$) and lithium hydroxide monohydrate ($LiOH.H_2O$) appear to be the most effective lithium compounds. The efficacy of the lithium compound depends on the nature of the reactive aggregate. Indications are that a minimum molar ratio of [Li]/[Na+K] of 0,60 is required to prevent deleterious ASR.[10.20]

Gajda[10.24] has shown that by intergrinding between 0,8 and 1,2% by mass lithium hydroxide monohydrate with a high-alkali clinker (>1,0% Na_2O-eq) a cement was produced that did not cause deleterious expansion due to ASR. It is possible to use a combination of S, CSF or FA and a lithium compound in order to reduce the cost of the lithium addition.

Portland cement clinker containing lithium

Lithium-bearing ores in the raw feed to the kiln can be used to incorporate lithium into the clinker. Apart from acting as a flux during clinkering and therefore reducing the energy costs per ton of cement produced, "incorporation of lithium into cement manufacture should allow some kiln dust to be recycled back into the portland cement without adding to ASR susceptibility due to the cement's increased alkali content".[10.25]

10.6.3 Potentially reactive aggregates

It has been established, both from their service record and from laboratory tests, that aggregates derived from the crushing of the following rocks are deleteriously alkali reactive and should, therefore, only be used if the alkali content of the concrete can be controlled or a supplementary major constituent used as described in section 10.6.2.

Witwatersrand Supergroup quartzite/shale
Malmesbury Group metasediments
Table Mountain Group orthoquartzite
Bokkeveld Group arkose
Enon Formation quartzite pebbles
Quaternary quartzite pebbles
Cape granite

Furthermore, laboratory tests have shown that aggregates derived from the following rocks are potentially alkali reactive:

Natal Group quartzite and sandstone
Dwyka Formation tillite
Karoo basalt (some) and dolerite (some)

In addition, recent diagnostic examination of cores from structures has shown that concrete containing Archaean granite from Mpumalanga and Salem granite from Walvisbay, Namibia, had cracked and deteriorated as a result of ASR (R E Oberholster, Cembeton Consult confidential reports, 2008).

Microcrystalline quartz is probably the reactive constituent in the aggregates mentioned above. However, the presence of quartz showing strain effects such as undulatory extinction, deformation lamellae and deformation bands and suturing has been found to he a good indicator of potentially alkali-reactive rocks. The list of minerals and rock types given in Table 10.2 was compiled from a source of which the author is unknown and modified as more information became available. It serves as a guideline for the identification of potentially alkali-reactive aggregates. When an aggregate does not belong to a source that has been found to be reactive by its service record or previous testing, its reactivity must be determined in the laboratory.

10.6.4 Environment

External moisture

Concrete structures exposed to the elements, ie changing conditions of moisture content and temperature, or to continual dampness, are vulnerable to ASR. However, where the concrete is well protected and dry, any aggregate complying with SANS requirements may be used regardless of the reactivity of the aggregate or the alkali content of the mix. In the latter case it must be remembered that some elements or parts of elements of a structure that is well protected may still be exposed to conditions of high humidity, for example, beams or parts of columns buried in soil or subject to condensation due to thermal gradients.

Concrete containing an alkali-reactive aggregate and a high alkali content can be protected against reaction and expansion by limiting access of water by improved drainage; or treatment with an approved hydrophobic material, such as silane or siloxane [10.27. 10.28]; or cladding.[10.29]

Exposure to sea spray

Exposure of concrete to sea spray increases the sodium content of the concrete, especially in the first 90 mm, and leads to increased expansion where a reactive aggregate is used and the alkali content of the mix is high.[10.30] Therefore, if a reactive aggregate is used in concrete in a marine environment, the contribution of the alkali from the sea spray must be taken into account in the calculation of the alkali content of the concrete over a long period. The average alkali content of the concrete could increase by as much as 1,5 kg Na_2O-eq /m^3 in 10 years.[10.30]

In contrast with concrete exposed to sea spray, concrete submerged in sea water does not appear to be affected by ASR.[10.31] This is based on the observation that in Cape Town harbour dolosse, made with the same combination of cement and alkali-reactive aggregate, below the tidal zone are not cracked, while those in and above the tidal zone are cracked.[10.32] A possible explanation for this is that the alkalinity of the pore solution adjusts to that of the sea water.

10.7 Methods and criteria for evaluating aggregate, cement and cementitious materials

10.7.1 Evaluating aggregates

Until about 1983 there were some half dozen methods to determine whether an aggregate was potentially alkali reactive. Since then there has been a proliferation of methods, especially those that give results within three to 14 days.

Preliminary screening methods

Petrographic examination and the gel-pat test are preliminary screening methods and do not give quantitative results, and are used to make a provisional assessment of the aggregate.

Indicator tests

A second group of methods such as the mortar bar tests and concrete prism tests that are performed using standard mix proportions and a fixed high-alkali content, can be regarded as indicator tests that give certain limits which separate potentially reactive aggregates from innocuous aggregates but do not give information on the actual alkali content of the concrete at which deleterious expansion will occur.

Table 10.2: Minerals, rocks and other substances that are potentially deleteriously reactive with alkalis in concrete

Minerals		
Opal Tridymite Cristobalite Chalcedony, cryptocrystalline, microcrystalline or glassy quartz Coarse-grained quartz that is intensely fractured, granulated and strained internally or rich in secondary inclusions Siliceous, intermediate and basic volcanic glasses Vein quartz		
Rocks		
	Rock	**Reactive component**
Igneous	Granodiorite Charnockite Granite	Strained quartz; microcrystalline quartz
	Pumice Rhyolite Andesite Dacite Latite Perlite Obsidian Volcanic tuff	Silicic to intermediate silica-rich volcanic glass; devitrified glass; tridymite
	Basalt	Chalcedony; cristobalite; palagonite; basic volcanic glass
Metamorphic	Gneiss Schist	Strained quartz; microcrystalline quartz
	Quartzite	Strained and microcrystalline quartz; chert
	Hornfels Phyllite Cataclasite[10.26] Mylonite Argillite	Strained quartz; microcrystalline to cryptocrystalline quartz
Sedimentary	Sandstone	Strained and microcrystalline quartz; chert; opal
	Greywacke	Strained and microcrystalline to cryptocrystalline quartz
	Siltstone	Strained and microcrystalline to cryptocrystalline quartz; opal
	Shale	
	Tillite	Strained and microcrystalline to cryptocrystalline quartz
	Chert	Cryptocrystalline quartz; chalcedony; opal
	Flint	
	Diatomite	Opal; cryptocrystalline quartz
	Argillaceous dolomitic limestone and calcitic dolostone Quartz-bearing argillaceous calcitic dolostone	Dolomite; clay minerals exposed by dedolomitisation
Other substances		
Synthetic glass; silica gel		

Performance tests

Thirdly, performance tests include the standard concrete prism tests using mix proportions that are intended for a specific project and long-term outdoor exposure of 300-mm cubes or large beams, using the same mix proportions as for the standard concrete prism tests or different mix proportions. The specimens may be stored in different geographical areas to establish the effect of different climatic conditions and to compare inland exposure with marine exposure. These tests give information on the limits of alkali content that can be tolerated in the concrete without harmful expansion occurring.

The results obtained with aggregate of the Tygerberg Formation of the Malmesbury Group obtained from different quarries in combination with seven different cements are shown in Figure 10.3. The Na_2O-eq of the cements ranged from 0,16% total and 0,11% active to 1,30% total and 1,10% active. An expansion of >0,05% after 52 weeks indicates deleterious expansion. It is clear from Figure 10.3 that all the aggregates expanded deleteriously with cement F, however only some expanded deleteriously with cement B. With cement C (not shown in the figure) with a total Na_2O-eq of

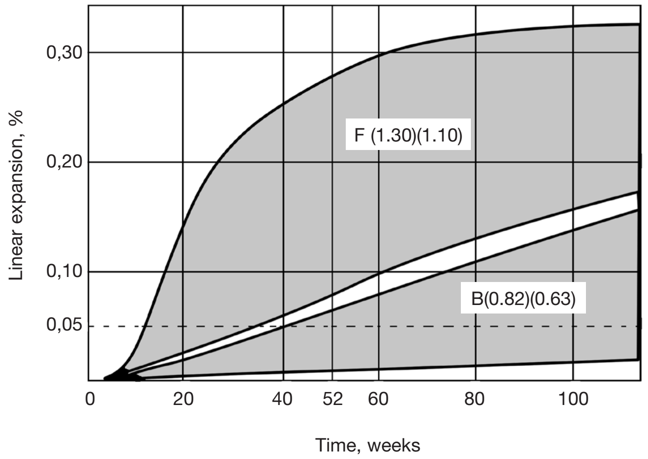

Figure 10.3: Distribution of expansion of concrete prisms made with various Tygerberg Formation aggregates and cements F(1.30)(1.10) and B(0.82)(0.63). Stored over water in sealed containers at 38°C.[10.10] Combinations expanding less than 0.05% at 52 weeks are regarded as non-deleterious reactive

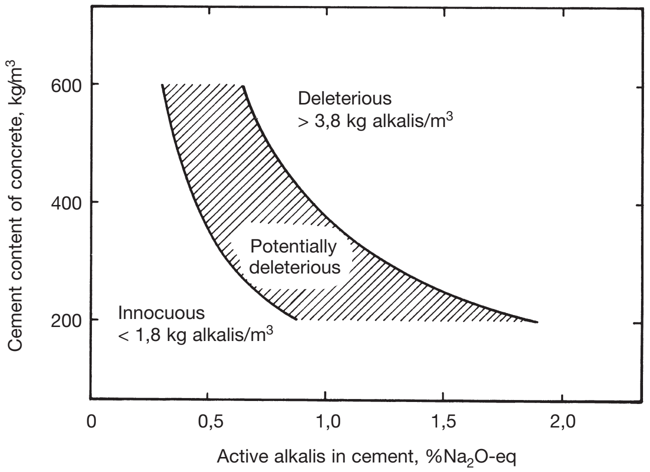

Figure 10.4: Marginal values for the cement content of concrete and active alkali content of cement when used in combination with Tygerberg Formation aggregates to predict whether in actual practice cracks will appear in the concrete as a result of ASR[10.10]

0,62% and active Na_2O-eq of 0,48%, none of the aggregates expanded deleteriously. The cement content of the prisms was equivalent to 350 kg/m^3, therefore, the Na_2O-eq where all the aggregates expanded deleteriously was 4.6 kg/m^3 total and 3,8 kg/m^3 active. The Na_2O-eq where deleterious expansion was not recorded with any of the aggregates was 2,2 kg/m^3 total and 1,8 kg/m^3 active.

The latter statement is important. Since about 1980, provision has been made in South Africa for the fact that aggregates differ in their alkali reactivity and, therefore, that for different aggregates, deleterious expansion will occur at different alkali concentrations in the concrete. It has also been stressed in the past that, although criteria are given for the different tests to separate deleteriously reactive, potentially deleteriously reactive and innocuous aggregates, the demarcation is not sharp. Furthermore, aggregate produced from a particular lithostratigraphic unit, is not necessarily uniform in terms of its properties, eg alkali reactivity. In the case of the metasediments (quartzite, greywacke, hornfels, phyllite, mylonite) of the Malmesbury Group it was found that at an alkali content of 3,8 kg active Na_2O-eq per m^3, deleterious expansion will definitely occur even with the least reactive aggregate in the Group and at a lower limit of alkali content of 1,8 kg active Na_2O-eq per m^3, deleterious expansion will definitely not occur, even with the most reactive aggregate of the Group. Between these two limits, the rate and amount of expansion will depend on the reactivity of the aggregate. Figure 10.4 illustrates the above concept for Malmesbury aggregate.[10.10] For the portland cements used in the tests at that stage, the corresponding total Na_2O-eq/m^3 was 2,2 kg and 4,6 kg respectively.

In deciding which test method to request, the engineer should ensure that:

- Sufficient time has been allowed, from the time of submitting the samples, for the test to be completed and the results to be available before a final decision on the materials to be used has to be made.
- Sufficient material which is representative of that intended for use is submitted for the test.
- The test method prescribed is appropriate for the material to be assessed.
- The laboratory performing the test has the knowledge and expertise to decide whether the test is appropriate, has experience of the performance of the aggregate or similar aggregate in practice and is capable of interpreting the results.

A brief outline of the conventional and some of the rapid methods is given below.

Petrographic examination

When coarse and fine aggregates are examined it is recommended that the descriptive nomenclature of ASTM C 294-05[10.33] be used and that ASTM C 295-08[10.34] be taken as a general guide. The presence, and if possible, quantities of materials such as opal, tridymite, cristobalite and chalcedony that are known to be reactive with the alkalis in concrete must be determined petrographically. For this purpose it is useful to refer to Table 10.2. In the case of rocks such as greywacke, hornfels, argillite, phyllite, quartzite, granite, mylonite, granite-gneiss, etc, attention must be given to the presence of crypto- and micro-crystalline quartz and features associated with the deformation of quartz, such as undulatory extinction, sutured boundaries and signs of intergrowth or reaction with the matrix as a result of low-grade metamorphism.

RILEM recommended test method TC 106-1[10.35] provides procedures for the petrographical examination and classification of aggregate for potential alkali reactivity.

The gel-pat test

The gel-pat test[10.36] does not require sophisticated equipment and can be performed in site laboratories. For quartz-bearing aggregates it is proposed that it be performed at 80ºC. The specimens, gravel-sized particles, are examined every day with a magnifying glass, while illuminating them from the side, but without removing them from the oven. After 10 days the containers are removed from the oven and the pats carefully taken out of the NaOH solution and examined for reaction.

ASTM C 289-07. Potential reactivity of aggregate (chemical method)[10.37]

This test can be done quickly and requires a relatively small sample. ASTM states that it can be used as a quality control tool to periodically check samples from an existing source with an acceptable service history. Although it works well for some aggregates, it is not recommended for aggregates such as greywacke, hornfels, argillite, phyllite, quartzite, granite, mylonite, granite-gneiss, etc, which may be found to be innocuous according to this test but have a poor service record when used with high-alkali cement. It has been proposed[10.10] that when the duration of the test is increased from one to three days a quantity of more than 1% of silica dissolved indicates a potentially alkali-reactive aggregate. A variation of the method, which apparently gives good results with a wide range of siliceous aggregates, was proposed by Sorrentino et al.[10.38]

The test is not appropriate for carbonate aggregates.

ASTM C 227-03. Potential alkali reactivity of cement-aggregate combinations (mortar-bar method)[10.9]

This is one of the most useful tests to determine the susceptibility of a particular combination of cement and aggregate to harmful expansion. However, the criteria given for this method in ASTM C 33-93,[10.39] namely that cement-aggregate combinations showing expansions greater than 0,05 % at three months or greater than 0,10 % at six months should be considered capable of harmful reactivity, should not be taken as rigid limits. South African research has shown[10.10] that many aggregates passing the above criteria when used in combination with high-alkali cement, have a service history of being alkali reactive. It is well established that slowly expanding quartz-bearing rocks such as greywacke, hornfels, argillite, phyllite, quartzite, granite, mylonite, and granite-gneiss, give lower rates of expansion than, for example, opal-containing aggregates. Therefore, based on the extensive research in South Africa, it has been proposed that for slowly-expanding aggregates the test procedure and criteria be adapted to provide for (a) the evaluation of the potential of deleterious expansion of specific cement-aggregate combinations and (b) the reactivity of different aggregates. Thus, any cement-aggregate (slowly-expanding aggregate) combination tested in the prescribed proportion (equivalent to 600 kg cement/m^3, giving an expansion of more than 0,05% in 52 weeks, should be considered capable of harmful expansion. Likewise, any slowly expanding aggregate tested in combination with a cement with an active alkali content of 1,10 % Na$_2$O-eq, at a cement content of 600 kg/m^3 of mortar, which shows an expansion of more than 0,05 % in 52 weeks, should be regarded as potentially deleteriously reactive.[10.10] ASTM subsequently added the following to ASTM C 33-93,[10.39], "Test Method C 227 is not suitable for slowly reactive aggregates, and its use for this purpose is not advised."

Dolomitic aggregates that are harmfully reactive with alkalis in concrete may not produce notable expansion in this test.

Despite the simplicity of the test, only laboratories with extensive experience should tackle it. Many researchers of international repute have experienced great difficulty in producing expansion with materials that are known to be reactive. The major problem appears to be controlling the relative humidity in the containers at a sufficiently high level. Because of the problems experienced with the test, a control aggregate of known reactivity should always be included in each series of tests.

Accelerated mortar-prism (bar) tests [10.35, 10.40, 10.41]

These methods are based on the National Building Research Institute (NBRI) accelerated mortar prism test, [10.42] which has been investigated extensively in a number of overseas countries and found to be reliable and reproducible.[10.35, 10.43 Different criteria have been established for the method in different countries, but the most widely accepted criteria for expansion are given below:

South Africa, at 12 days	ASTM C 1260-07 after 16 days
• Less than 0,10% aggregate innocuous • More than 0,10% but less than 0,20%: aggregate slowly reactive or inconclusive • Equal to or more than 0,20%: aggregate deleteriously reactive, rapidly expansive	• Less than 0,10% indicative of innocuous behaviour in most cases • Between 0,10% and 0,20%: include both aggregates that are known to be innocuous and deleterious in field performance • More than 0,20%: indicative of potentially deleterious expansion

RILEM TC 106-AAR-2 regards the test method unsuitable for the evaluation of porous flint aggregate.[10.35]

Concrete prism method

Concrete prisms are used in many parts of the world for evaluating the potential alkali-reactivity of cement-aggregate combinations. In South Africa prisms measuring 75 mm x 75 mm x 300 mm are normally used. The prisms are stored over water in sealed containers at 38°C as in ASTM C 227-03[10.9]. The advantage of this method is that the ingredients can be used in the proportions specified for the actual construction.

For quartz-bearing aggregates such as greywacke, hornfels and quartzite, it has been proposed that an aggregate which produces an expansion of more than 0,05% within 52 weeks, in combination with a cement with an active alkali content of 1,1% Na_2O-eq and a cement content of 350 kg/m^3 of concrete, should be considered potentially deleteriously alkali reactive.[10.10] Likewise, any combination of cement and aggregate showing an expansion of more than 0,05% after 52 weeks by this method should be considered a deleteriously expansive combination.

Variations are the RILEM TC 106-AAR-3[10.35], CSA, A23.2-14A [10.44] and ASTM C 1293-06[10.45] methods.

Ultra-accelerated concrete performance tests

RILEM TC-ARP/10/20[10.37] developed a draft ultra-accelera-ted concrete prism test AAR-4, which is used for assessing the reactivity performance of particular concrete mixes. Results can be obtained after three months. It is recommended that the test be run for six months.

ASTM C 586-05. Potential alkali reactivity of carbonate rocks for concrete aggregates (rock cylinder method)[10.48]

The expansive characteristics of carbonate rocks, immersed in a solution of 1 M NaOH at room temperature, are determined. Rock cylinders with a length of 35 mm and a diameter of 9 mm are used. Modifications of this method are used to test quartz-bearing aggregates. It has, however, been found not to be a practical method for determination of the alkali reactivity of rocks such as greywacke and hornfels. [10.10]

10.7.2 Evaluating common cements and supplementary major constituents (extenders)

Total alkali content

Routine analysis by cement producers in South Africa for the major and minor oxide content of cement is done by X-ray fluorescence spectrometry. However, it is not considered appropriate to apply this method to the determination of Na_2O content in view of the inadequate precision at present obtainable by X-ray fluorescence spectrometry using fused beads at the levels normally present in cement or its major constituents. For the accurate determination of the total alkali content, SANS Method 50196-21[10.47] must be followed.

Although the individual alkali oxides are reported to the nearest 0,01%, the Na_2O-eq is expressed to the nearest 0,1%. Results between different laboratories for the same cement can differ by more than 0,05% for the individual alkali oxides.[10.10]

South African standard specification SABS 471:1982 [10.48] made provision for a limit on alkali content that may be specified when portland cement (CEM I) is to be used in concrete with aggregates that may be deleteriously reactive, with the inclusion of a clause that stated that, should a purchaser require a low-alkali content cement, such a cement shall have a total Na_2O-eq not exceeding 0,6%. However, this specification was replaced by SANS 50197-1, which does not provide for a limit on the alkali content.

CEM II and CEM III cements were not widely in use when the above clause was included in SABS 471. Since the total Na_2O-eq of S, CSF and FA mostly exceeds that of clinker, the likelihood that the total Na_2O-eq of CEM II, CEM III, CEM IV or CEM V cement will be more than 0,6% is therefore quite high. Whenever alkali-reactive aggregate is to be used in concrete and the alkali content of the cement exceeds the limit that is regarded as safe, the type of cement should be established and in the case of cement blends, the necessary adjustments, as discussed in section 10.8.3, should be made.

In Germany portland blastfurnace cement is regarded as a low-alkali cement if:

- For a slag content of at least 50%, the total Na_2O-eq is not more than 1,1%.
- For a slag content of at least 65 %, the total Na_2O-eq is not more than 2,0%.[10.49]

Currently only three of the supplementary major constituents that are being used in South Africa (and complying with SANS 50197-1), can potentially inhibit deleterious ASR, ie S, CSF and FA. FA and CSF possess pozzolanic properties while S is a latent hydraulic cement. All three these materials could have a higher total alkali content than some of the CEM I cements produced in South Africa.

Water-soluble alkali content

It is maintained by some researchers that the water-soluble alkali-content of CEM I cement gives a better indication than the total alkali content of the quantity of alkalis that is available to react with an alkali-reactive aggregate. A method for determining the water-soluble alkali content is given in ASTM C 114-07.[10.50] Analyses showed that the water-soluble Na_2O-eq of South African CEM I cements ranged between 11 and 81% of the total Na_2O-eq.[10.10] The percentage will be lower for CEM II and CEM III cements.

The water-soluble alkali content of supplementary major constituents is usually low (less than 0,02%) and is not normally determined.

Active alkali content

Since a very good correlation was found between the available alkali content of a cement as determined by ASTM C 311-07.[10.51] but omitting the addition of $Ca(OH)_2$, and the expansion of mortar prisms made with a reference alkali-reactive aggregate of the Malmesbury Group, using the ASTM C 227 mortar-bar test, this value is taken as the active alkali content of cement.[10.10] It was proposed that as a general guideline to inhibit ASR, the active alkali content of a low-alkali CEM I cement should not exceed 0,5% Na_2O-eq.

The active alkali content of South African CEM I cement ranges between 67% and 98% of the total Na_2O-eq with an average of 81%. Results of the test are obtained after 28 days. For large projects where an alkali-reactive aggregate is to be used with CEM II or CEM III cement of which the total Na_2O-eq exceeds the calculated "safe" limit, it is recommended that the active alkali content be determined and used in the calculations.

The active (available) alkali content of South African S, CSF and FA is determined by ASTM C 311-07 [10.51] and has been found to be on average 51%, 42% and 32% respectively of the total alkali content.[10.10, 10.52] A limit of 1,5% Na_2O equivalent is placed on the available alkali content of these materials as specified in Table 1 of ASTM C 618-08 if it is to be blended with CEM I cement, to minimise the risk of ASR.[10.53] However, this limit might be too high, especially at low replacement levels of the clinker component.

Effectiveness in preventing expansion due to ASR

ASTM C 441[10.15] is a test method that has been used to evaluate the effectiveness of S or a pozzolan in reducing expansion due to ASR. ASTM C 311,[10.51] which is a modification of ASTM C 441, provides a procedure for evaluating the effectiveness of FA or natural pozzolan in reducing ASR expansion. In both these tests, mortar bars are prepared with Pyrex glass as a standard reactive aggregate and the S or pozzolan replacing a portion of a high-alkali cement. Guidance is provided in Appendix XI of ASTM C 311 for the interpretation of results.

Because the results obtained with the above test method have poor reproducibility, it has been recommended that the test be modified by using non-reactive quartz sand as aggregate and replacing the minus-2,36-plus-1,18-mm and minus-1,18-plus-0,60-mm fractions with Pyrex glass. The prisms are measured at 14 days, one month, two months and three months. A reduction of 75% in the expansion of the mortar prisms containing the Pyrex glass is regarded as proof that the extender is suitable for preventing ASR.[10.10] The suitability of Pyrex glass as a standard reactive aggregate has recently also been questioned by other researchers.[10.54]

Based on the research of Davies and Oberholster[10.55], the accelerated mortar prism method[10.56] is used in South Africa to determine the minimum effective amount of S or pozzolanic material to control expansion due to ASR. Multiple runs of the test with various amounts of S or pozzolan replacing CEM I cement and a reference reactive Malmesbury metasediment aggregate are conducted. The effective amount of S or pozzolan is the amount that reduces the expansion to below 0,10%. The method is also used to evaluate the effectiveness of CEM II and CEM III cements in reducing deleterious expansion in combination with the reference Malmesbury or any other alkali-reactive aggregate.

Potential deleterious expansion of supplementary major constituents (extenders)

Section 10.1.13 of ASTM C 595-08.[10.57] describes the procedure to establish whether the pozzolan or cement extender (supplementary major constituent) itself might cause deleterious expansion when it reacts with the CEM I cement. The material, to be used in the production of a blended cement, is tested by blending it with the clinker or cement, in quantities of 2,5, 5,0, 7,5, 10,0, 12,5 and 15% and making prisms with a non-reactive sand as described in ASTM C 227.[10.9] The expansion of the mortar prisms is measured at 91 days and all blends should meet the requirements of maximum expansion of 0,05%.

10.8 Calculating the alkali content of concreting materials

10.8.1 General

Methods for determining the alkali content of SANS 50197-1 common cements and supplementary major constituents, and definitions of acid-soluble alkali, total alkali, water-soluble alkali, available alkali and active alkali content, are given in section 10.7.2.

10.8.2 CEM I cement

The alkali content of cement clinker produced in South Africa is given in Table 10.3.[10.58] However, the alkali content of CEM I produced from a particular clinker can be more or less than that of the clinker depending on the alkali content of the gypsum that is added to control the set of cement and also of the minor additional constituent. Gypsum from a natural source usually contains impurities in the form of quartz and phyllosilicates (clay minerals). The alkali content (usually potassium) of the clay per mass can be more than that of the clinker, with the result that addition of gypsum increases the alkali content. However, the alkali of the clay is practically insoluble in water and will not contribute significantly to the alkali content in the pore solution of concrete as a result of hydration. The minor additional constituent that is added to cement may also contain alkali. However, this is generally less than that of the clinker and also practically insoluble in water. Furthermore, analyses have shown that the active Na_2O-eq of clinker can be between 7 and 32% less than the total Na_2O-eq. (See Table 10.3.)[10.58] All these factors make it very difficult to lay down limits for acceptable alkali content of concrete made with reactive aggregate, based on the Na_2O-eq of cement as determined by SANS 50196-21.

An example of the contribution of the different constituents to the alkali content of a CEM I cement is given below.

The total Na_2O-eq of a clinker used for the manufacture of a CEM I cement is 0,66% and of the impure gypsum 1,37%. The $CaSO_4.2H_2O$ content of the impure gypsum is 38,45%, therefore, 61,55% ends up in the products as minor additional constituent (NDM). The composition of the resulting CEM I cement and its total and active Na_2O-eq, if it is assumed that the alkali of the impurities in the gypsum is insoluble, are given in Table 10.4.

The alkali content of synthetic gypsum that is used in South Africa is generally less than 0,2% Na_2O-eq and therefore, contributes an insignificant amount to the total alkali content of cement; consequently, practically all of the alkali is contributed by the clinker.

Although the active alkali content of CEM I cement is a better indication than the total alkali content of the alkalis available for reaction with the reactive constituents in an aggregate, the limits for alkali content per m^3 of concrete given in Table 10.1 for the different aggregates are, for practical reasons, the total alkali content in the case of CEM I cement. The criteria established at that stage incorporated a factor that took into account that the active alkali content of South African CEM I cement was on average 85% of the total alkali content. The latter value may now be less if the minor additional constituent that is used contains alkali, or it may be more if synthetic gypsum is used to control the set of cement. Therefore, because cement manufacturers do not determine the active alkali content of their cements and do not state whether they use gypsum from a natural source or synthetic gypsum, practical considerations determine that the total Na_2O-eq of CEM I cement has to be used to calculate the alkali content per m^3 of concrete that can participate in ASR.

If a CEM I cement with a total Na_2O-eq of 0,7% is to be used with an aggregate from the Malmesbury Group in the South Western Cape, then the maximum safe limit of cement content can be calculated as below, using the criteria given in Table 10.1.

Table 10.3. Na_2O-equivalent of South African clinkers, 2008[10.58]

Factory	Total Na_2O-eq, %	Total Na_2O-eq, %
PPC Hercules	0,6	0,5
PPC Hercules	0,7	0,6
PPC Dwaalboom	0,4	0,3
PPC Jupiter	0,5	0,4
PPC Slurry	0,6	0,5
Lafarge Lichtenburg	0,3	-
AfriSam Dudfield	0,3	-
AfriSam Ulco	0,6	-
CIMPOR Simuma	0,8	-
PPC Riebeeck	0,9	
PPC Riebeeck	0,8	0,6
PPC De Hoek	0,7	0,6
PPC Port Elizabeth	0,6	0,5

Table 10.4. Contribution of impurities in gypsum to alkali content of CEM I

Constituent	%	Total Na_2O-eq, %	Active Na_2O-eq, %
Clinker	90,73	0,60	0,45
$CaSO_4.2H_2O$	3,56	0	0
NDM	5,71	0,131	0
Total	100	0,73	0,45

Calculated from the total of 9,27% impure gypsum

10.8.3 Other SANS 50197-1 cements

CEM II, CEM III, CEM IV and CEM V cements are extensively used in South Africa. The calculation of the theoretical "safe" alkali content of concrete when one of these cements is used will depend on the type, the source and quantity of major additional constituent and the alkali content of the clinker. The contribution of gypsum to the total alkali content will in most cases be probably less than 0,01%. Table 10.5 shows the type and amount of extender commonly used and Table 10.6 the alkali content.

Although replacement of a portion of a CEM I cement with S and FA will in most cases increase the total alkali content of blended cement, the active alkali content of the pore solution could be decreased. In the United Kingdom, the active alkali content of PC, S and FA is taken as 100, 50 and 17% respectively of the total acid-soluble alkali content.[10.59] It is seen from Table 10.6 that the corresponding values for South Africa are 36 to 63% for S and 24 to 38% for FA. The average active alkali content for 17 South African cements was found to be 81%.[10.10] The Na_2O-eq of South African CSF was previously found to be 1% total and 0,3% available.

Example: A CEM I is to be used with aggregate from the Malmesbury Group

Maximum acceptable total Na_2O-eq derived from clinker kg/m^3	2,1 kg Na_2O-eq/m^3
Total Na_2O-eq of cement, % (analysed)*	0,7% Na_2O-eq
Maximum acceptable cement content, kg/m^3	2,1 x 100/0,7 = 300 kg/m^3
* It is assumed that all of the alkali of the CEM I cement is available for ASR	

For the purpose of calculating the alkali content it may be assumed that the maximum S or FA content is 18% and 33% when dealing with CEM II A and B respectively. Where these are combined it may be assumed that the ratio is 60/40 for FA and S respectively.[10.58]

Portland slag cement and Portland blastfurnace cement

The total and active Na_2O-eq of S produced at the different plants is given in Table 10.6. The ratio of active to total alkali content of the particular slag in the blend is used for the calculation of the active alkali content.

If 40% or more S is used, even with CEM I cement with a total Na_2O-eq as high as 1,3%, deleterious expansion due to ASR is effectively controlled. However, if a portland cement blend with less than 40% S is used, it is recommended that the following guidelines and worked examples be followed to calculate the active alkali content and therefore maximum acceptable cement content.

Portland fly ash cement

The total alkali content of FA is determined by X-ray fluorescence spectrometry or more accurately by SANS 50196-21.[10.47] The active available (available) alkali content is determined using sections 15 and 16 of ASTM C 311-05.[10.51] A limit of 1,5 % Na_2O-eq was previously placed on an FA as specified in Table 2 of ASTM C 618-05 if it is to be blended with CEM I to minimise the risk of ASR. This requirement does not appear in ASTM C 618-05.[10.53] South African FAs effectively control expansion as a result of ASR even if the active alkali content is as high as 1,10% Na_2O-eq, provided a minimum of 20% by mass of the high-alkali clinker is replaced with FA. If a high-alkali cement contains less than 20% FA, then the active alkali content and therefore maximum acceptable cement content must be calculated.

Table 10.5. Extender contents of South African cements, 2008

Cement Type	Slag	Fly ash	Limestone
CEM II/A - S	18	-	-
CEM II/A - V	-	15	-
CEM II/A - V	-	18	-
CEM II/B - S	33	-	-
CEM II/B - V	-	33	-
CEM II/A - L	-	-	18[1]
CEM II/B - L	-	-	30[2]
CEM II/A – M(V-L)	-	10	8
CEM II/A – M(V-L)	-	15	4,5
CEM II/B – M(S-L)	30[3]	-	5[3]
CEM III/A	45	-	-
CEM IV/A-V	-	30	-
CEM IV/B-V	-	40	-
CEM IV	-	45	-
CEM V/A	18	25	-

1. CEM II/A 42,5 cements have 10-12% limestone
2. Slurry has 23% Limestone, Dwaalboom 27% Limestone, Ulco 25% limestone
3. This applies to De Hoek and the composition changes to 20% limestone and 8-10% slag depending on slag availability

Example: A CEM II/B-S is to be used with a Witwatersrand quartzite aggregate

Total Na_2O-eq analysed, %	0,80
Total Na_2O-eq contributed by 33% slag, from Table 10.6, %	(33/100) x 0,95 = 0,31
Active Na_2O-eq contributed by Vanderbijlpark S, from Table 10.6, %	(36/100) x 0,31 = 0,11
Total Na_2O-eq contributed by clinker, %	0,80 – 0,31 = 0,49
Na_2O-eq available for ASR*, %	0,49 + 0,11 = 0,60
Maximum acceptable cement content, kg/m^3	(2,0 x 100)/0,60 = 333

* It is assumed that all of the alkali in the clinker is available for ASR

Table 10.6. Alkali content of extenders commonly used used in South Africa, 2008[10.58]

Extender	Total alkalis			Active alkalis[1]			
	Na_2O, %	K_2O, %	Na_2O-eq, % (a)	Na_2O, %	K_2O, %	Na_2O-eq, % (b)	(b/a) x 100, %
Hercules	0,04	0,72	0,51	0,02	0,08	0,07	15
Jupiter	0,09	0,58	0,47	0,01	0,05	0,04	8
Slurry	0,21	0,18	0,32	0,003	0,012	0,011	3
Dwaalboom limestone							
Port Elizabeth limestone	0,21	0,19	0,33	0,05	0,16	0,16	47
De Hoek NDM	0,16	0,26	0,33	0,01	0,01	0,02	5
Riebeeck NDM	0,15	0,28	0,34	0,01	0,02	0,02	7
Vanderbijlpark GGBS	0,40	0,85	0,95	0,12	0,33	0,34	36
Newcastle GGBS	0,39	1,08	1,10	0,19	0,61	0,60	55
Saldanha GGCS	0,20	0,57	0,57	0,11	0,38	0,36	63
Lethabo fly ash	0,34	0,94	0,96	0,09	0,21	0,23	24
Matla fly ash	0,55	0,96	1,18	0,22	0,35	0,45	38
Kendal fly ash	0,25	0,95	0,87	0,10	0,30	0,30	35

1. Referred to as available alkalis in ASTM C 311:05[10.51]

The total and active Na_2O-eq of FA produced at the different plants are given in Table 10.6. The ratio of active to total alkali content of the particular FA in the blend is used for the calculation of the active alkali content.

Example: A CEM II/A-V or W is to be used with a Witwatersrand quartzite aggregate

Total Na_2O-eq, analysed,%	0,80
Total Na_2O-eq contributed by 18% Matla FA, %	(18/100) x 1,18 = 0,21
Active Na_2O-eq contributed by Matla FA, %	(38/100) x 0,21 = 0,08
Total Na_2O-eq contributed by clinker, %	0,80 – 0,21 = 0,59
Na_2O-eq available for ASR, %*	0,59 + 0,08 = 0,67
Maximum acceptable cement content, kg/m^3	(2,0 x 100)/0,67 = 299
* It is assumed that all of the alkali in the clinker is available for ASR	

Portland silica fume cement

The total alkali content of CSF is determined by X-ray fluorescence spectrometry or more accurately by SANS EN 50196-21.[10.47] The active (available) alkali content is determined using sections 15 and 16 of ASTM C 311-05.[10.51] There is doubt about the long-term effectiveness of CSF in preventing deleterious expansion of concrete due to ASR, especially if the active Na_2O-eq of the cement blend exceeds 1,0% and the CSF content is less than 15% (m/m). Currently CEM II/A-D cement in bags is not sold in South Africa. Blends specified by the customer are supplied in bulk. It is recommended that if less than the recommended 15% by mass of CEM I is replaced by CSF to mitigate deleterious ASR, the following procedure be followed to calculate the alkali content and therefore maximum acceptable cement content:

Example: A blend of 85% CEM I and 15% CSF is to be used with a Witwatersrand quartzite aggregate

Total Na_2O-eq of CEM I, analysed, %	0,80
Available Na_2O-eq of CSF, analysed, %	0,30
Na_2O-eq of blend available for ASR (taking into account 5% NDM), %*	(80/100 x 0,80) + (15/100 x 0,30) = 0,69
Maximum acceptable cement content, kg/m^3	(2,0 x 100)/0,69 = 290
* It is assumed that all of the alkali in the clinker is available for ASR	

In all the above calculations it has been assumed that none of the three extenders at the quantities used play any additional role in reducing the harmful effects of ASR. This may not be correct, in which case the values for acceptable cement content calculated above may be on the low side.

Portland limestone cement

The alkali content of limestone used by different manufacturers for the production of limestone cements differs from plant to plant. The figures in Table 10.6 show that the total Na_2O-eq of limestone used at PPC plants as extender or minor additional constituent (NDM) ranges from 0,32 to 0,51% and the active Na_2O-eq from 3 to 47% of the total. The value of 47% can be regarded as an exception because the material probably contains chloride salts. Therefore, when used with a low-alkali clinker, addition of limestone can increase the total Na_2O-eq of the product compared to CEM I cement, but when used with a high-alkali clinker, the alkali content of the product will be less compared to CEM I cement. An example of the contribution of limestone to the alkali content of CEM II B-L cement is given below. The active Na_2O-eq of the limestone was 8% of the total Na_2O-eq.

Example: Effect of limestone on the Na_2O-eq of CEM II B-L

Total Na_2O-eq of clinker, %	0,70
Total Na_2O-eq of limestone, %	0,50
Total Na_2O-eq of CEM II B-L cement (30% limestone), %	0,49 + 0,15 = 0,64
Active Na_2O-eq of CEM IIB-Lcement, %	0,49 + (8/100 x 0,15) = 0,50
* It is assumed that all of the alkali of the CEM I is available for ASR	

Portland composite cement

The clinker content of Portland-composite cements can ranged from 94 to 65% and the extender content from 6 to 35%. The main constituents other than clinker of CEM II A-M, and CEM II B-M cements have to be declared by designation of the cement. Table 10.5 shows that at the time of writing this chapter, two types of composite cement were being manufactured, namely CEM II A-M (V-L) and CEM II B-M (S-L). To calculate the maximum cement content acceptable when used with an alkali-reactive aggregate the same procedure as for portland-slag and portland-fly ash cement should be followed.

Blastfurnace cement

The clinker content of blastfurnace cement can be between 64 and 5% and the S content between 36 and 95%. In 2008 the S content of South African blastfurnace cement CEM III A was 45% and in line with the recommendations in section 10.6.2 this amount when used in combination with a high-alkali clinker, should be sufficient to inhibit ASR. Therefore, calculations of the active alkali content should not be necessary if the slag content is at least 40%. However, where the Na_2O-eq of CEM I made with a high-alkali clinker exceeds 1%, the S content should be increased by 10 percentage points. The German guidelines are expressed slightly differently in terms of the alkali content of the blastfurnace slag itself, namely that (portland) blastfurnace cement is regarded as a low-alkali cement if:

- The slag content is at least 50%, for a total Na_2O-eq that is not more than 1,1%.
- At least 65%, for a total Na_2O-eq that is not more than 2,0%.[10.49]

Pozzolanic cement

The main constituents other than clinker of pozzolanic cements CEM IV A and CEM IV B have to be declared by designation of the cement; the clinker content can be between 89 and 45% and the other main constituents, CSF, natural or calcined pozzolana and FA, between 11 and 55%. In 2008 two pozzolanic cements were produced using South African clinker, a CEM IV/A-V 32,5 with 30% FA and a CEM IV/B-V 32,5 R with 40% FA. Some CEM IV/A-V 32,5 was produced using imported clinker with Na_2O-eq of 0,77%. To calculate the maximum cement content acceptable when this cement type is used with an alkali-reactive aggregate the same procedure as for portland-slag and portland-fly ash cement should be followed. The ameliorating effect of the pozzolan that is used on ASR should be taken into consideration.

Composite cement

For composite cements CEM V A and CEM V B the main constituents other than clinker have to be declared by designation of the cement. The clinker content can range from 64 to 20% and the content of the sum of CSF, natural or calcined natural pozzolan and FA, from 18 to 50%. In 2008 only one producer supplied a composite cement, namely CEM V A 32.5N with a slag content of 18% and FA content of 25%. The maximum cement content acceptable when this cement type is used with an alkali-reactive aggregate is calculated following the same procedure as for portland slag and portland fly ash cement.

10.8.4 Mixing water

In the case of water analyses, alkalis are usually reported in mg/ℓ of Na+ and K+ ions which is different from cement analyses where the alkalis are reported as percentage Na_2O and K_2O (Na x 1,35 = Na_2O; K x 1,20 = K_2O).

Analyses of mains water and water in the major storage dams are available from most local authorities and from the Department of Water Affairs respectively. The alkali content of water supplied by the local authorities in the three major centres, Gauteng, Durban and Cape Town, is low and will not contribute significantly to the active alkali content of concrete. The alkali content of some of the dams supplying the major centres with water is higher, ie about 60 mg/ℓ Na^+ and 10 mg/ℓ K, but even this is too low to make a significant difference to the active alkali content of concrete.

If sea water is used as mixing water, it will contribute a significant amount of alkalis to the concrete mix. Assuming an alkali content of 11 000 mg/ℓ Na^+ and 400 mg/ℓ K^+ in sea water used in a concrete containing 375 kg/m^3 of cement at w:b = 0,5, the contribution by the water to the active alkali content of the concrete will be:

$$\{375 \times 0{,}5/100\} \times [(1{,}10 \times 1{,}35) + (0{,}658 \times 0{,}04 \times 1{,}20)] = 2{,}84\ \text{kg/m}^3$$

10.8.5 Chemical admixtures

The alkali content of chemical admixtures for concrete that are being used in South Africa is between about 0,2 and 7%. In many instances the contribution to the alkali content of the pore solution is significant and cannot be ignored. For example, one of the most widely-used superplasticisers has a Na_2O-eq of about 5%. If added at a dosage rate of 1% by mass of cement to concrete with a cement content of 350 kg/m^3, it will add 0,17 kg/m^3 Na_2O-eq to the concrete.

Example: A CEM II/B-S is to be used with a Witwatersrand quartzite aggregate with the abovementioned superplasticiser dosed at 1% by mass of cement. From Table 10.1, the maximum acceptable Na_2O-eq is 2,0 kg/m^3

Total Na_2O-eq, analysed, %	0,80
Total Na_2O-eq contributed by 33% slag, from Table 10.6, %	(33/100) x 0,95 = 0,31
Active Na_2O-eq contributed by Vanderbijlpark S, from Table 10.6, %	(36/100) x 0,31 = 0,11
Total Na_2O-eq contributed by clinker, %	0,80 – 0,31 = 0,49
Na_2O-eq available for ASR, %*	0,49 + 0,11 = 0,60
Total Na_2O-eq of superplasticiser, %	5
Na_2O-eq of cement plus superplasticiser available for ASR, %	(0.99 x 0.60) + (0.01 x 5) = 0.64
Maximum acceptable cement content, kg/m^3	(2,0 x 100)/0,64 = 312
* It is assumed that all of the alkali in the clinker is available for ASR	

10.9 Investigation of damaged structures

Owners or authorities may monitor the condition of structures regularly and systematically or on an ad hoc basis. Often, cracking or other forms of deterioration may be noticed by chance. Regular repair and maintenance due to normal in-service wear and tear are usually planned for in advance. However, sometimes repair and maintenance that had not been planned for may be required, for example due to mechanical damage, poor workmanship or unsuspected processes of degradation, such as ASR.

In certain instances, the main reason for repairing a structure may be deterioration as a result of ASR. More often than not, repairs may be part of a general programme of maintenance. Therefore, the contribution to the deterioration of mechanisms other than ASR, such as reinforcement corrosion due to carbonation, sulphation or chloride ingress must also be established.

When it is foreseen that a structure requires maintenance, the following steps are recommended:

- Carry out a preliminary inspection to establish the extent and nature of the deterioration and take photographs.
- Revisit the site accompanied by specialists to decide which tests should be done, the types of samples that should be taken and their position.

Where ASR is suspected, take core samples as follows:

- Preferred minimum diameter, 100 mm
- Preferred minimum length, 150 mm

- Record the:
  - Date
  - Type of element
  - Exact position of core
  - Direction of drilling, ie horizontally, vertically upward or downward, inclined
  - Core diameter, total length and different sections
  - Location and condition of reinforcing steel
  - Position and direction of cracks in immediate vicinity of the core position
- Number core clearly
- Indicate on the outer face, the top of the core where drilled horizontally, and the direction of the longest dimension when drilled vertically

The engineer doing the structural assessment must be able to transfer the information such as the position, width, length, distribution and direction of cracks, position and condition of reinforcing steel, depth of carbonation, location and intensity of ASR, etc, from the core description to his structural analysis model.

10.10 Diagnosis of ASR in damaged structures

10.10.1 Appearance of the structure

ASR in concrete is characterised by some or all of the following phenomena. Some of these, when present in combination, are diagnostic of ASR. Others are often associated with ASR, but are not diagnostic because they are visually similar to features observed in concrete unaffected by ASR.

Some features that are indicative of ASR are:

Cracking

The most obvious manifestation of the reaction is cracking. Unrestrained concrete shows typical map-pattern cracking (see Figure 10.5). Conditions of restraint or loading induce forces that tend to influence the crack pattern in the concrete. The result is that cracks in columns, for instance, tend to run vertically (see Figure 10.6), while cracks in retaining walls, which are not as free to expand laterally as vertically, tend to run horizontally (see Figure 10.7).

Expansion

Closure of expansion joints, spalling, off-setting and warping of structural members and pavements, and misalignment of

Figure 10.5: Typical crack pattern attributed to ASR in unreinforced concrete

machinery based on concrete are evidence that expansion has taken place (see Figure 10.8).

Presence of gel

Drops of resinous gel may be observed on concrete surfaces (see Figure 10.9). In some cases the gel is stained. Streaks of gel, similar to the occurrence of calcium carbonate efflorescence, may be seen on vertical surfaces.

Discolouration

Dark discolouration giving the impression of permanent dampness is commonly observed along the edges of cracks. Cracks may also be bordered by light-coloured zones (see Figure 10.10).

Dampness

Damp patches on the surface are sometimes seen (see Figure 10.11).

Figure 10.6: Vertical cracks attributed to ASR in a circular bridge column (The cracks have been repaired with an epoxy resin.)

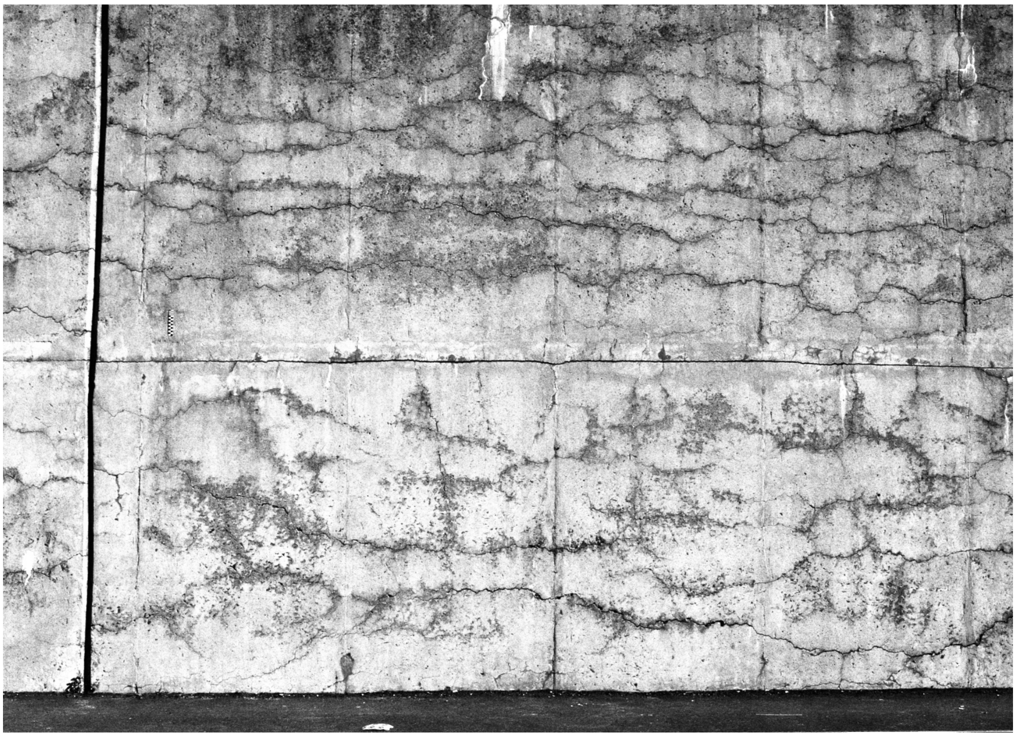

Figure 10.7: Predominantly horizontal cracks caused by ASR in a bridge retaining wall

Figure 10.8: Closing of joint and horizontal offsetting of parapet of a rail-over-road bridge due to expansion

Figure 10.9: Drops of resinous gel oozing out a concrete wall of a hydro-electric power station affected by ASR

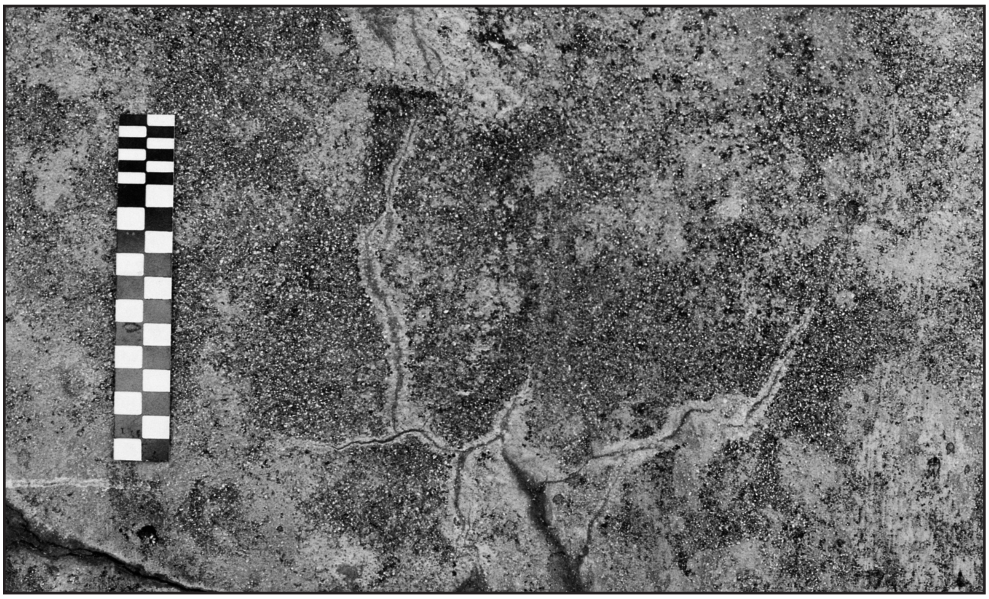

Figure 10.10: Discolouration of concrete along cracks

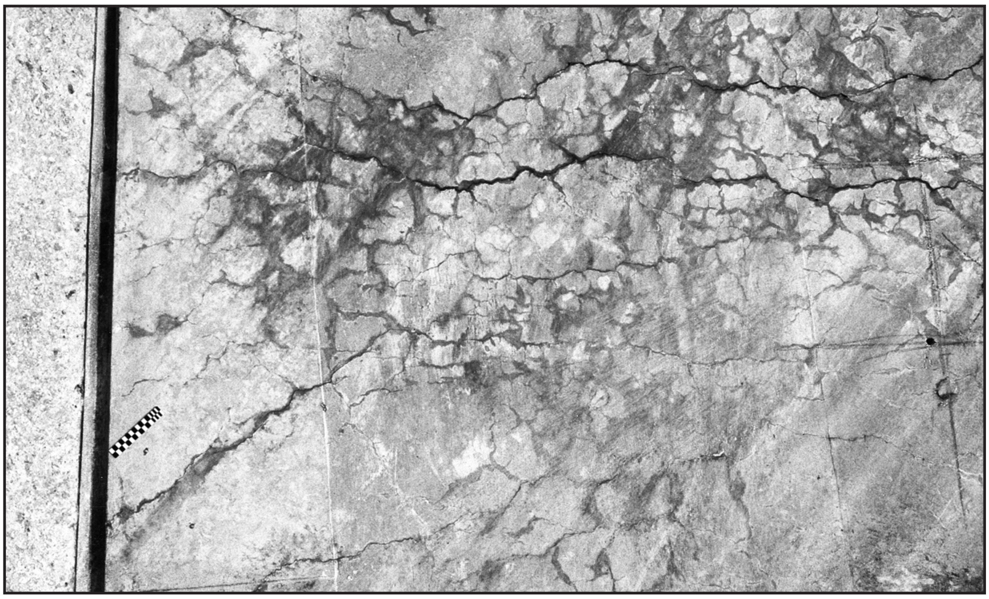

Figure 10.11: Damp patches on the surface of a reinforced concrete arch affected by ASR

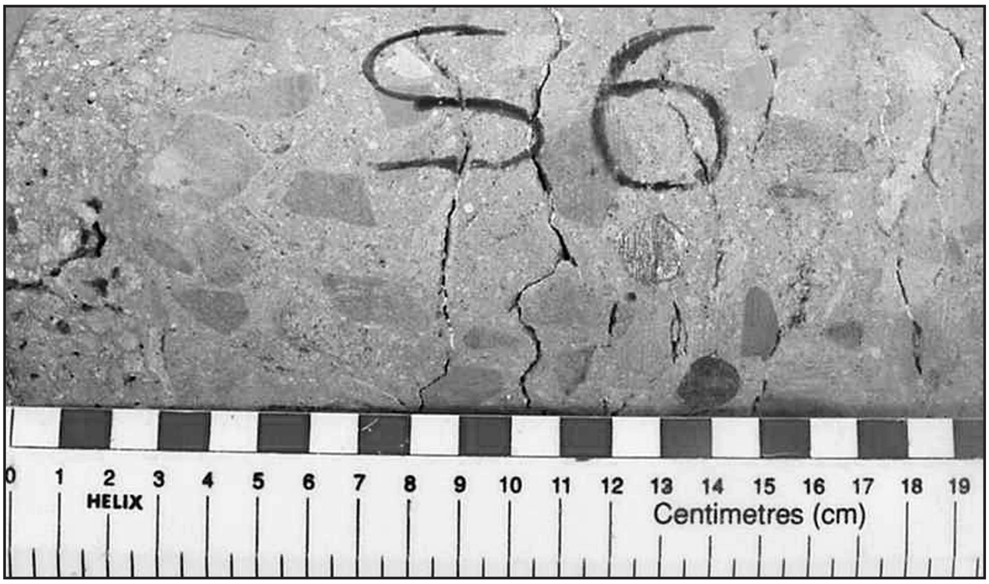

Figure 10.12: Horizontal cracks in core drilled vertically from a bridge deck

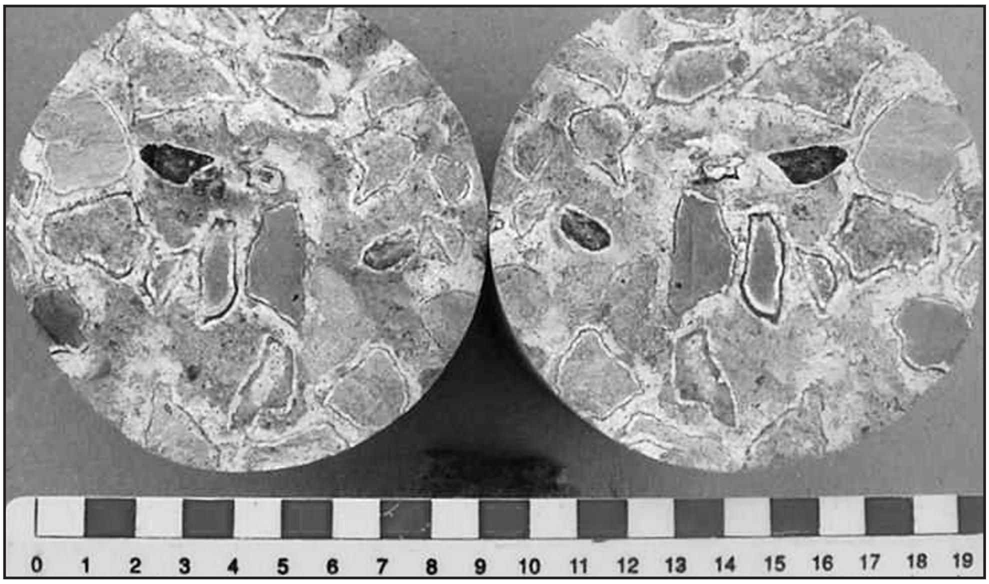

Figure 10.13: Two halves of core in Figure 10.12 showing white reaction product on aggregate surfaces. Note how the reaction product develops in cracks through the aggregate.

Pop-outs

Reacting particles near the surface of the concrete may produce pop-outs. While this feature is common in some parts of the world, it has not yet been observed in South Africa.

10.10.2 Examination of concrete specimens

Examination of cores or lumps of concrete with the naked eye and under a microscope at low magnification may reveal one or more of the following features:

- Cores drilled from structures display dark staining along cracks through the aggregate and in the case of cores drilled vertically from a bridge deck or pavement, cracks that are parallel or sub-parallel to the surface of the core (see Figure 10.12).
- A white deposit is visible on fracture surfaces of the aggregate. The deposit appears to be concentrated around the periphery of the aggregate, 0,1 to 0,5 mm from the edge and usually 0,3 to 1 mm thick, giving the impression of a reaction rim around the aggregate as shown in Figure 10.13. This feature is especially prominent in concrete containing dark-coloured aggregates, such as Malmesbury metasediments. The concentration of the white material on the fracture surface of the aggregate diminishes from the periphery towards the centre. The white deposit may also be observed on the surface of the mortar. Voids are lined or completely filled with reaction product, which can be translucent or porcelaineous, or porcelaineous in the interior and translucent on the exterior. The material can be soft or hard.
- Translucent gel is present at the interface between the aggregate and the mortar.
- Cracked aggregate is stained by viscous gel.
- Cracks are to be seen in the aggregate, and are in many cases partially filled with reaction product.
- Cracks in the mortar are filled with reaction product.

However, it needs to be emphasised that other mechanisms such as delayed ettringite formation (DEF) on its own or in association with ASR may also be the cause of the expansion and cracking of concrete. Therefore, positive identification of ASR as the cause of expansion and cracking of concrete should be supported by the examination of thin sections with a petrographic microscope and reaction products by X-ray diffraction analysis and scanning electron microscopy combined with energy dispersive analysis.

Davies and Oberholster published imformation on the morphology, chemical composition and d-values of ASR products.[10.60] A staining method to identify ASR products (in the field and laboratory) has been developed.[10.61]

10.11 Detailed investigation

If ASR has been confirmed, it is necessary to carry out a detailed investigation of the structure. Factors to be considered are: expansion and cracking; concrete strength; the severity of the effects of ASR in structural terms; and durability and serviceability.

10.11.1 Expansion and cracking

Expansion and cracking due to ASR are usually manifested to different degrees in different parts of a structure, and may in fact be completely absent in some parts. This may be due to a number of different reasons, for example:

- Precast elements and concrete cast in situ have different mix proportions and very often contain cement and aggregate from different sources.
- Concrete in different placements may differ because of non-uniformity in materials, proportioning and compaction.
- Different elements of a structure and portions of the same element are exposed to different environmental conditions.
- Different elements of a structure and portions of the same element are exposed to different environmental conditions.
- Alkalis may concentrate due to migration or be leached by moisture.

Expansion

It is important to know whether expansion has stopped or is still continuing before remedial work can be carried out on a structure. It is also important to know how much the concrete has expanded, because this has an influence on structural properties.

In measuring expansion on a structure, the influence of different levels of stress on expansion measured on different elements and on different faces of an element, at different positions, eg parallel to the edge of or in the centre of a face, and in different directions, must be taken into account.

Estimated current expansion

Determination of the current expansion can only be made indirectly. A method has been proposed[10.29] whereby at least five lines are drawn, not less than 1 m long, with at least 250 mm between them, in a direction perpendicular to the principal crack orientation on the most severely affected face of each placement of concrete being assessed. It is suggested that more lines should be used where the dimensions of the concrete dictate line lengths of less than 1 m. The expansion strain, together with the strains from other causes, can be assumed to be equal to the sum of the widths of the cracks divided by the length of the line drawn on the concrete. The current total strain equals the sum of the widths divided by length, expressed in mm/m. Other strain effects, structural as well as non-structural, must be taken into consideration in the interpretation of the results. However, these other strain effects may make the method too uncertain.

Measurements on large concrete beams, all but two of them being unreinforced, exposed to the weather gave the following relationship between the mean crack width and expansion due to ASR after five years: [10.27]

% Expansion = (0,312 x Crack width, mm) + 0,0236

However, both of these methods probably only give an approximate indication of expansion.

Potential additional expansion

The potential maximum free expansion that may still occur in a structure may be estimated from the free expansion of cores extracted from the structure and stored in sealed containers at approximately 100% relative humidity and 38ºC.[10.8] Cores of about 100-mm diameter are preferred. Initial expansion that takes place is caused by the uptake of water by the cement paste and the aggregate, and is a normal phenomenon ascribed to the recovery of drying shrinkage. The magnitude of this expansion will depend on the quality of the concrete as a whole and the degree of desiccation of the concrete core being tested. Alkali-silica reaction products present in the concrete will also contribute to the initial expansion. The reversible drying shrinkage recovery should be deducted from the total expansion. It is difficult to differentiate between initial expansion resulting from reversible drying shrinkage recovery and from ASR. However, if readings are taken on a daily basis for the first two weeks, the slope of the expansion curve should give an indication of the recoverable drying shrinkage. The reversible drying shrinkage recovery of a normal sound concrete should be of the order of 200 to 300 microstrain. However, tests done on cores from a structure affected by ASR after one year's exposure to a relative humidity of approximately 100% and 38ºC, have given values of between 430 and 650 microstrain for drying shrinkage and 260 and 580 microstrain for wetting expansion[10.8] determined in accordance with BS 1881- 5.[10.62] These values give an indication of the contribution of alkali-silica reaction products to the dimensional changes resulting from changes in moisture conditions.

Stark[10.63]proposed the following procedure. Three 100-mm diameter companion cores drilled from a structure are stored under the following conditions respectively: (i) one continuously over water in a sealed container at 38ºC, (ii) one continuously in N NaOH solution at 38ºC and (iii) one continuously in water at 38ºC. Storing condition (i) gives an indication of the potential for continued expansion, (ii) of the presence of unreacted reactive particles and (iii) allows corrections to be made for wetting expansion.

It must be remembered that the expansion of concrete in a structure is less than that of concrete cores in the laboratory. Concrete in the structure is under constraint whereas laboratory cores are unrestrained; environmental conditions are also different. However, the prime objective of testing cores is to establish whether the structure, as represented by one or more core samples, has the potential to expand. It is important, though, to take cores from the different elements of a structure and different exposure positions to obtain a representative picture of the potential expansions.[10.29]

Rate and duration of expansion

It is not possible to predict, from the expansion of cores under laboratory conditions, the rate and duration of expansion under field conditions. In South Africa, the earliest manifestations of ASR have usually been observed between three and four years after completion of a structure although measurements on concrete specimens stored outside showed that expansion starts after about 10 months, reaches significant proportions at about 15 months and stops after about 10 years. Monitoring of a mass concrete structure has shown that expansion was still taking place after 42 years, but at a slow rate. Therefore, once remedial work has been carried out, it is important to install targets and monitor dimensional changes.

Cracking

The expansive forces, which develop locally in individual aggregates as a result of ASR, lead to cracks in the aggregates or open up pre-existing cracks which gradually extend into the surrounding concrete. The gel concentrates in the cracks in the aggregates but also permeates into the surrounding concrete, resulting in random micro-cracking. The expansion is not uniform throughout the volume of the concrete but is greater in the vicinity of expanding aggregates. Because of differences in, amongst others, moisture conditions, porosity, alkali concentration and degree of carbonation between the heart and surface of the concrete, differential strains develop leading to macro-cracking in the surface.

Both the width and depth of cracks are greatly influenced by the amount and position of reinforcement in the concrete as well as other external forces acting on the concrete.[10.64] Although there is a relationship between the width of the cracks observed on the surface and the depth of the cracks, it is found that the crack width decreases greatly at the plane of the reinforcement steel and is either terminated or branches out into finer cracks merging into micro-cracking. It has been stated that the depth of macro-cracks does not usually exceed the lesser of the cover and about one tenth of the member thickness.[10.64] As pointed out in section 10.10 in unrestrained concrete, map-pattern macro-cracking develops, while in the presence of restraint to expansion, the macro-cracks will tend to be parallel to the direction of the restraint.

10.11.2 Concrete strength

The expansions that develop in structures affected by ASR are highly variable and differential, as has been pointed out. It is important to take this into account in the evaluation of the structural effects of ASR.

The effect of ASR on the mechanical properties of concrete is summarised in Table 10.7. The data are from reference 10.29 as summarised from a literature review by Clark,[10.65] data published in references 10.66 and 10.67 and unpublished data supplied by the Task Group of reference 10.68, and were derived from lower bounds of laboratory data obtained from tests on cast cubes, cylinders and prisms, and from tests on cores extracted from structures. The Task Group pointed out that it should be noted that the uniaxial compressive strength, as obtained from testing a long (length:diameter ratio = 2,5 or greater) cylinder or core, is reduced by ASR to a greater extent than is the cube strength. It is this uniaxial compressive strength which is required for structural assessment. The residual tensile strength is affected by the test method. The values quoted in the table are appropriate to the splitting tensile or torsional tensile strength. It is emphasised that the residual strengths and stiffnesses on actual structures will be modified from the figures in the table. This is because the concrete in actual structures is generally restrained by the adjacent material and is in a biaxial or triaxial stress state. These effects will tend to reduce the damage to the concrete and increase its residual mechanical properties. The actual 28-day strength of concrete is, in most cases, in excess of its design value by an amount which is greater than any subsequent reduction due to ASR. Hence compressive strength reduction is not normally a problem in practice. However, tensile strength reduction may require special consideration. It must also be kept in mind that cores that are macroscopically badly cracked are usually not used for strength tests. Consequently, test results on cores might present a slightly optimistic picture.

For unreinforced concrete cubes and prisms made in the laboratory and stored over water in sealed containers at 38°C, expansion and cracking of concrete caused by ASR do not cause a significant decrease in compressive strength, while the flexural strength decreases significantly. In structures where there is a well-detailed three-dimensional cage of reinforcement with good anchorage both of the main bars and the shear links, the detrimental effects of ASR are mitigated. The steel resisting the expansions puts the concrete into compression and, in a sense it becomes "pre-stressed" (selfstressed). Unfortunately, very few reinforced concrete structures are detailed and provided with anchorages appropriate to prestressed concrete. The additional bond stresses developed by ASR expansions can induce bond and shear failures between the concrete and the reinforcement. Where the reinforcement is confined to opposite faces of a member without any shear steel or links tying the main steel together, there is substantial risk of delamination leading to a severe loss of shear and flexural strength. It is not sufficient merely to consider the effects of ASR as a loss of concrete strength. The detailing requirements of BS 8110 and earlier codes are generally designed to ensure a ductile failure mode in normal concrete. This detailing may not be adequate for ASR effects which can produce an embrittlement of the structure. This renders it even more vulnerable to the redistribution of forces and moments arising from ASR expansions developing at different rates in different members.[10.29, 10.69]

In reinforced concrete structures, the degree of expansion is influenced by the amount of reinforcement. The mitigation of deterioration in the mechanical properties of concrete is insignificant at a reinforcement ratio of less than 0,1% but is significant where the ratio is more than 0,7%. [10.70]

When cores are taken from a structure to determine the influence of ASR on the physical properties of the concrete, it is important to keep in mind the influence of stress from dead and live loads and reinforcement detail on the cores taken from different directions in a particular element of the structure. It has been shown by Rigden et al[10.71] that in blocks subjected to high uniaxial stress, the amount of damage recorded in directions perpendicular to the uniaxial stress direction was increased and the amount of damage recorded in the direction of the uniaxial stress was decreased

Table 10.7: Lower bound residual mechanical properties as percentage of values for unaffected concrete at 28 days[10.29]

Property	Percentage strength as compared with unaffected concrete for free expansion (microstrain) as indicated				
	500	1 000	2 500	5 000	10 000
Compressive strength*	100	85	80	75	70
Uniaxial compressive strength**	95	80	60	60	-
Tensile strength	85	75	55	50	-
Elastic modulus	100	70	50	35	30

* Cube

** Core, length:diameter ratio = 2,5 or greater in comparison with an unstressed block. High levels of stress will increase the amount of damage recorded in the direction perpendicular to the uniaxial stress direction and decrease the amount of damage recorded in the direction of the uniaxial stress. Though not covered in their investigation, it is plausible that directional restraint due to reinforcing steel and loads, is likely to result in similar variations in core test results for elastic modulus, plastic strain, ultrasonic pulse velocity and compressive strength, to that reported by them for their particular test.

10.11.3 Structural severity rating

A procedure for the structural severity rating for each member of a structure has been drawn up by the Institution of Structural Engineers, UK, to assist with the management and monitoring of structures affected by ASR. The system is reproduced in Table 10.8. Terms used in the table are defined as follows:

Expansion index:

The expansion characteristics of the concrete are defined in terms of an expansion index in the range of I to V which is related to either the estimated current free expansion, for an assessment of the current state of the structure, or to the estimated future total expansion, for an assessment of the future state of the structure.

I Expansion less than 0,06%
II Expansion of at least 0,06% but less than 0,10%
III Expansion of at least 0,10% but less than 0,15%
IV Expansion of at least 0,15% but less than 0,25%
V Expansion equal to, or greater than 0,25%

Site environment:

Dry

- Reliably dry, always < 75% RH
- Currently dry, < 75% RH but at risk from change of use or leakage

Intermediate

- Intermediate, always < 85% RH
- Currently intermediate but at risk from change of use or leakage

Wet

- All buried parts, or those resting against soil
- All exterior elements in UK climates whether subject to rain or condensation
- Interior elements where frequent wetting can occur, eg washing, frequent showering, swimming pools, cooking, etc
- Where condensation can occur due to thermal gradients

Reinforcement detailing:

Class 1: A three-dimensional cage of very well anchored reinforcement
Class 2: A three-dimensional cage of conventionally anchored reinforcement
Class 3: A two-dimensional cage of reinforcement in one or two faces; no through ties, no links or low cover

Summarised management procedures based on structural severity rating:

Negligible

Routine inspections done at a frequency that is generally accepted for the type of structure.

Mild

Three-yearly engineering inspections may be appropriate as may the monitoring of cracks and laboratory testing of cores.

Moderate

Annual engineering inspections may be appropriate. Crack monitoring will be required. The use of other instruments should be considered. Laboratory and field testing of cores is probably required.

Table 10.8: Structural element severity rating[10.29]

Structural severity rating for combination of conditions as indicated											
Site environment	Reinforcement detailing class	Expansion index									
		I		II		III		IV		V	
		Consequence of failure									
		Slight	Significant	Slight	Significant	Slight	Significant	Slight	Significant	Slight	Significant
Dry	1	N	N	N	N	N	N	N	N	N	N
	2	N	N	N	N	N	N	N	N	N	Mi
	3	N	N	N	N	N	N	N	Mi	Mi	Mo
Intermediate	1	N	N	N	Mi	Mi	Mo	Mi	Mo	Mi	Mo
	2	N	N	Mi	Mo	Mi	Mo	Mo	Mo	Mo	S
	3	N	Mi	Mi	S	Mo	S	S	VS	S	VS
Wet	1	Mi	Mi	Mi	Mo	Mi	Mo	Mo	S	Mo	S
	2	Mi	Mi	Mo	S	Mo	S	S	S	S	VS
	3	Mi	Mo	Mo	VS	S	VS	VS	VS	VS	VS

Structural severity ratings:

N = Neglible; Mi = Mild; Mo = Moderate; S = Severe; VS = Very Severe

Severe

Three-monthly inspections may be appropriate. Extensive instrumentation and laboratory testing will be required. Structural strengthening may be necessary. Load restriction may be necessary. Water exclusion is important.

Very severe

Immediate action and specialised and detailed studies are required.

The above system for structural element severity rating and management procedures for concrete affected by ASR must not be applied rigidly, but be used as a guideline. For example, one must ask whether the Structural element severity rating of a prestressed concrete railway sleeper with an Expansion index of V can be rated as Mild if the site environment is Dry compared with Severe for the same sleeper in a site environment classified as Intermediate. One would expect that the dynamic load on such a sleeper in the line would be the major factor determining its serviceability irrespective of the site environment. It is important to realise that an element undergoing the above amount of expansion, and the resultant cracking, and which serves a structurally significant purpose, needs immediate attention to establish whether, for example, it can still perform satisfactorily for a predicted period of time; its lifespan can be increased by suitable rehabilitation measures; or it has reached a limit of deterioration where a critical reduction in the load-carrying capacity might be expected.

10.11.4 Appraisal of durability and serviceability

Factors that need to be considered when appraising a structure affected by ASR are whether its durability, ie safe performance for the designed life expectancy, taking into account normal maintenance, or serviceability, ie the capability to perform the function for which it was designed and constructed, [10.72] has been affected.

Appearance

Although ASR is structurally serious only in rare cases, the durability is invariably influenced because unexpected maintenance is required. Many owners are faced with the problem of carrying out expensive repairs to improve the appearance of structures that are cracked as a result of ASR. The potential corrosion of steel reinforcement as a result of access of moisture and oxygen through cracks caused by ASR also necessitates repairs. Remedial measures are expensive and, in many cases, not permanent.

Corrosion of steel reinforcement

While concern about the potential corrosion of steel reinforcement as a result of access of moisture and oxygen through cracks caused by ASR is often expressed, recent research results indicate that there is no general relationship between cracking and corrosion in most circumstances [10.73] and cracking at right angles to a reinforcing bar is often relatively unimportant.[10.74] Narrow cracks in a direction transverse to that of the reinforcement steel may not lead to serious corrosion except in severe environments, since the corrosion will probably be localised and shallow. Wider cracks, and particularly cracks parallel to the direction of the embedded steel, may provide greater access to corroding substances, ie water, oxygen, chloride and CO_2, and thus accelerate their attack while aggravating other causes of corrosion.[10.75] As a guide, where concrete quality and cover comply with current codes of practice and surface cracks do not exceed 0,3 mm, it is reasonable to assume that the effect of ASR on corrosion rates should not be significant.[10.29] Cracks in concrete affected by ASR generally run parallel to the reinforcing steel.

Because prestressing steel is more vulnerable to corrosion, prestressed structures affected by ASR should be investigated where crack widths of 0,15 mm and wider are present.

Deformations in structural members

Flexural deformations in beams and slabs will tend to be in the direction opposite to that caused by load and should not be a cause for concern with expansions up to 1,0 mm/m. In respect of overall expansion of members, the normal provisions for movement and expansion should cater for expansions up to 0,5 mm/m. Beyond this point, the consequences of expansion should be assessed, but it should be noted that estimated expansions from cores do not allow for the restraining effects of reinforcement.[10.29]

Mass concrete

Serviceability problems such as leakage in dams and reservoirs, jamming of sluice gates and other equipment and misalignment of sensitive equipment, such as power station turbines, are not uncommon and are known to have resulted in expensive maintenance and remedial measures and even replacement of structures or parts of structures.[10.76]

10.12 Repair procedures

In general the purpose of the repair is to restore or enhance one or more of the following:[10.77]

- Appearance
- Serviceability
- Durability
- Structural strength

Moisture is required for ASR to proceed. Therefore, to inhibit or ameliorate expansion, moisture has to be prevented or restricted from entering the concrete. At the same time, the concrete must be able to dry out and its internal relative humidity to drop below 75%.

10.12.1 No more reaction and expansion

If reaction and expansion have stopped, new cracks should not form and cracks that are grouted or coated should open very little or not at all. The method of filling, grouting or covering the cracks, and the material to be used, will depend on the purpose of the repair and the width, length and distribution of the cracks. Text books [10.77, 10.78] and information manuals of suppliers of repair materials [10.79] which give detailed instructions on materials and procedures, are available. Structures do not look attractive after cracks have been repaired; aesthetic considerations will dictate whether to coat the repaired surfaces.

10.12.2 Reaction and expansion continuing

Silane/siloxane surface treatment

It will serve no purpose to repair "live" cracks. Depending on how good the bond is between the material used to fill the cracks and the concrete, cracks may either open up again or new cracks may form. Structures or elements of structures, that can be treated as a whole with a hydrophobic substance such as a silane or siloxane should over a period of time dry out sufficiently for the reaction and expansion to stop[10.27, 10.28] provided that no part of the concrete is continuously in contact with free water or wet soil. Since the hydrophobic material allows free movement of water vapour in and out of the concrete, it will not dry out sufficiently for the reaction and expansion to stop if the mean annual ambient relative humidity is above a value of between 75% and 85%. Ventilated cladding, which prevents the concrete from being wetted directly, might also allow it to dry out. Only after the concrete has dried out sufficiently for ASR to stop (established by regular monitoring for dimensional changes over a period of at least two seasons) should cracks be repaired and the surface coated if required.

Several structures affected by ASR in South Africa have been treated with silane and the reaction successfully stopped.[10.28, 10.76, 10.80, 10.81] There is uncertainty about the effectiveness of treating for example the wing wall of a bridge with silane to stop further expansion (because water can enter via the fill). However, rehabilitation work done in Canada indicates that the application of a surface treatment such as silane may be an inexpensive and effective solution.[10.82]

Where there is concern about corrosion of reinforcing steel, a migrating corrosion inhibitor could be applied to the concrete surface prior to the application of the silane.

Assessment of a structure should always include a structural appraisal to determine whether it (or parts of it) needs to be strengthened[10.83] or replaced.[10.84] In a construction like a railway line, the assessment could comprise, for example, an appraisal of which of the sleepers:

- Should be treated to slow down or stop the reaction and usefully extend their service life;
- Should be strengthened as well, if economically feasible;
- Have deteriorated to such an extent that it is not economical to treat them and should be replaced in future or immediately.[10.82]

Lithium-based treatment

A number of field trials are under way in the United States to evaluate the effectiveness of lithium-based treatment of ASR-affected structures to inhibit expansion. These involve treatment of the concrete surface with an aqueous solution of a lithium salt or the use of an applied electric field to drive lithium ions into the affected concrete.[10.85-10.87]

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Chapter 11
Concrete mix design

Brian Addis and John Goodman

11.1 Introduction

The ingredients of concrete, namely cementitious material, water, sand, stone, and possibly admixture, must be proportioned in a mixture that :

- in the fresh state, can, without segregating, be transported, placed, fully compacted and finished if necessary;
- in the hardened state, achieves the required strength, adequate durability, and satisfactory dimensional stability;

at the lowest cost.

Because of the large number of variables involved in designing a mix theoretically, the use of approximations and short-cut methods is justified and undue refinements may be discarded.

According to McIntosh:[11.1]

> "Precise relationships have not been established between the properties of concrete, and even the more specific characteristics of the mix such as water: cement ratio, aggregate:cement ratio and grading let alone such elusive qualities as aggregate particle shape and texture. The data used in selecting mix proportions should be expected to do no more than serve as a guide. Mix design is a matter of trial and error, and any calculations based on design data are really only a means of making an intelligent guess at a starting point for the first tests to be made."

A competent concrete technologist can combine the science of mix design with experience and intuition and obtain proportions which will be so close to the final figures that only one or, at most, two trial mixes will be required.

Mixes are normally proportioned in the laboratory using samples of specific materials to be used for the concrete. Proper care during sampling is essential in order to obtain a representative sample, but clearly it is also important for the materials being used in the concrete to be as uniform as possible.

This chapter describes a method of mix design used by the Cement & Concrete Institute and gives guidelines for mixes for some special applications. Worked examples are also included.

Note on terminology

In this chapter, the word cement is used to indicate all the cementitious material in a concrete mixture. Cement may therefore be a "common cement" (see Chapter 1) or a blend of common cement and one or more cement extender.

11.2 C&CI method

This method is derived from ACI Standard 211.1-91[11.2] and is based on the following:

- The strength, at a given age, of fully compacted concrete, cured under standard conditions, is governed by water: cement ratio w:c and type of cementitious material used. Note: w:c means the ratio of total mass of the water in the mix, including water in the aggregates, to the total mass of cementitious materials.
- The amount of water required per unit volume of concrete for a given consistence and with given materials is substantially constant regardless of cement content, w:c or proportions of aggregates and cement. The main factors determining the amount of water are aggregate properties, cement properties, maximum size of the stone and the selected workability range. (This assumption is normally valid in the strength range of 20 to 30 MPa, but the water requirement of concrete of strengths outside this range should be determined.)
- For any particular concrete mix and combination of materials, there is an optimum stone content which depends on size, shape and compacted bulk density of the stone, fineness modulus of the sand and desired consistence of the concrete.
- The volume of compacted concrete produced by any combination of materials is equal to the sum of the absolute volumes of the cement and aggregates plus the volume of water and that of any entrapped or entrained air. The absolute volume of each material is calculated from the mass and the relative density of the material (D).

$$\text{Absolute volume, m}^3 = \frac{\text{Mass of material, kg}}{\text{D x 1 000}} \text{ ....(1)}$$

While all concrete mixtures contain some air, normally this may be ignored when calculating the concrete yield, except when air-entraining admixtures are used.

For practical purposes any discrepancies in the above assumptions are normally small enough to be of no consequence. The purpose of the trial mix is to verify the validity of the assumptions, and any significant inaccuracy would be revealed at this stage.

11.2.1 Applying the method

Practical mix design is a process involving the actions shown in Figure 11.1.

Actions in the mix design process can be categorised as follows:

- Specify concrete properties and characteristics – the starting points for the process.
- Select materials.
- Characterise materials.
- Proportion materials.
- Make and assess a trial mix.
- Specify mix proportions.

In the figure, arrows indicate the inputs required for each action. For the sake of simplicity, the use of admixtures is not included.

The following notes refer to numbered actions in Figure 11.1. Responsibilities are for conventional circumstances but could be otherwise in specific cases.

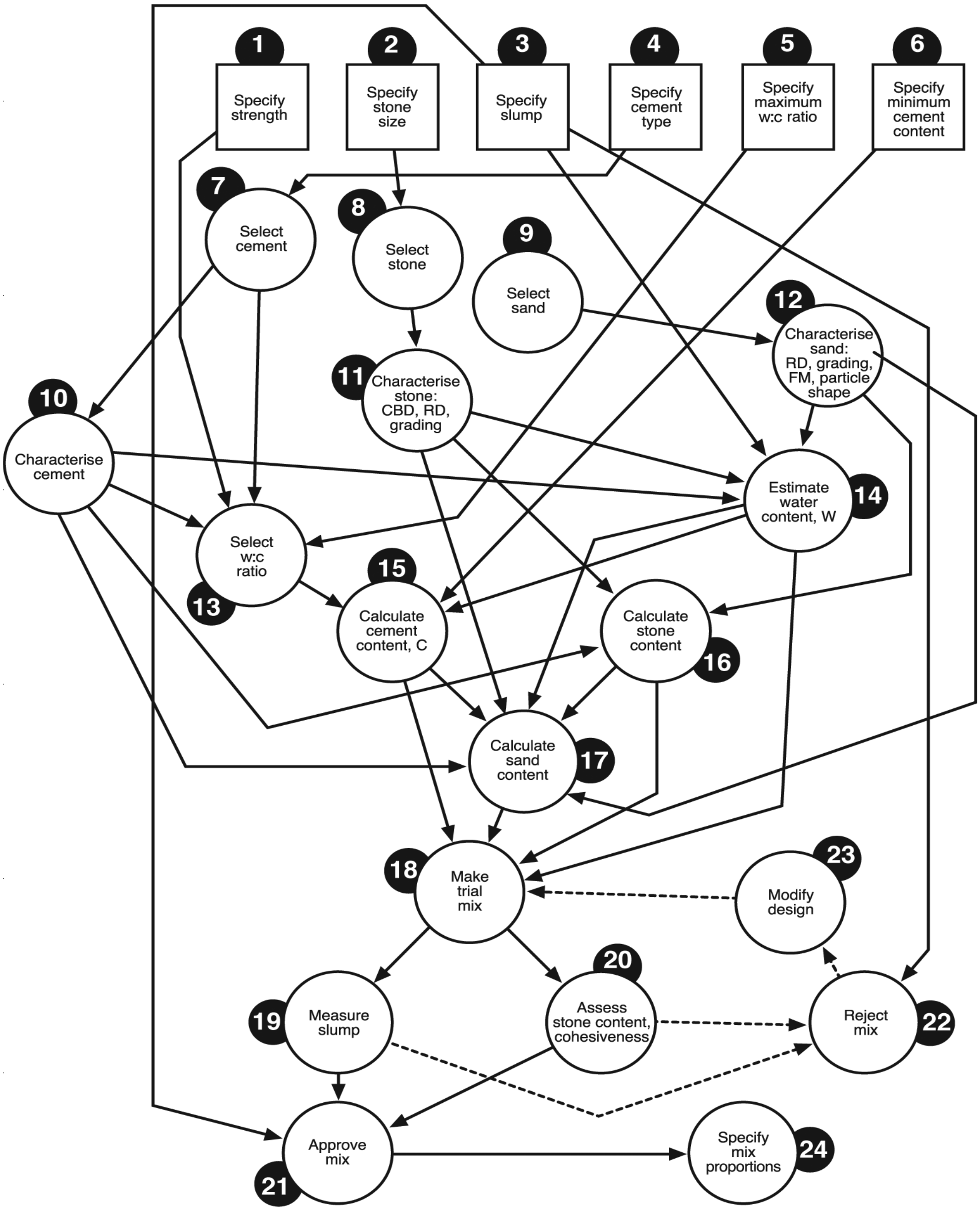

Figure 11.1: Mix design process: actions and inputs

1 and 2: Specify strength of concrete and stone size
By the designer of the structure. Optimum stone size depends on the dimensions of structural elements, depth of cover and spacing of reinforcement.

3: Specify slump
By the contractor to suit methods of transporting, placing and compacting.

4, 5 and 6: Specify cement type, maximum w:c ratio and minimum cement content
By the designer/consultant to meet durability requirements. Not done in all cases. Sometimes these items alone are insufficient to provide adequate durability and additional protective measures, eg a protective coating, may have to be used. Specification should be in line with the latest relevant code or technical literature.

7: Select cement type and source
By the mix designer for maximum economy but choice could be overridden by action 4 above. (See Table 11.7 for guidelines for selecting cements and blends for specific applications.) Consider all requirements for the concrete, eg any requirement for early strength.

8 and 9: Select stone and sand
By the mix designer for maximum economy of overall mix while still meeting any aggregate specification requirements. Use concrete technology knowledge to select appropriate aggregates for the application. In South Africa sands are often blended to achieve better concrete properties (see Chapter 3).

Note: The cheapest materials do not necessarily produce the cheapest concrete (see actions 14 and 15).

10: Characterise cement
This includes:

- Strength performance, needed to select w:c
- Particle relative density, needed to calculate solid volume
- Possible effect on workability of concrete

The strength performances of South African cements may be obtained from cement producers.

Particle relative densities of some cementitious materials are shown in Table 11.1.

Table 11.1: Particle relative densities of cementitious materials

Material	Particle relative density
CEM I	3,14
GGBS	2,9
FA	2,3
CSF	2,1

Effective particle relative densities of blends of different materials should be calculated as follows.

$$D = \frac{100}{\frac{\%X}{D_x} + \frac{\%Y}{D_y} + \ldots} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2)$$

where D is the effective particle relative density of the blend; %X and %Y are percentages by mass of materials X and Y respectively in the blend; and Dx and Dy are particle relative densities of materials X and Y respectively.

Values for factory-blended cements may be obtained from cement producers.

FA blends tend to reduce the quantity of water required for a given consistence and permit higher stone contents.

11: Characterise stone

CBD	– Compacted bulk density is a measure of packing capacity.
RD	– Particle relative density is needed to calculate solid volume.
Grading	– A sieve analysis is done to determine or verify nominal size.

12: Characterise sand

RD	– Particle relative density is needed to calculate solid volume.
Grading	– A sieve analysis is done to determine distribution of particle sizes.
FM	– Fineness modulus is calculated from the sieve analysis.
Particle shape	– Assess by eye, eg rounded, cubical, flaky, elongated.
Surface texture	– Affects water requirement of concrete.

13: Select w:c ratio
To achieve a target strength to satisfy a specified strength requirement, based on the performance of the specific cement. The selection may possibly be overridden by requirements of actions 5 and 6.

Note that the compressive strength of site-made, 150-mm cubes may be about 2 MPa lower than that of 100-mm laboratory specimens. This difference is small and may be ignored.

14: Estimate water requirement, W
Depends on:

- Particle shape and surface texture of sand
- Stone size
- Required slump
- Type of cementitious material (The inclusion of FA tends to reduce water requirement.)

An estimate of the water content of the mix, using the particular aggregate and cement combination selected, may be made in one of three ways:

- By using figures obtained from mixes previously made with the same materials
- By estimating, based on experience of similar materials
- By using average figures from Tables 11.2 and 11.3

The water contents given in Table 11.2 are for concrete made with an optimum quantity of 19-mm stone and having a slump of 75 mm.

For other sizes of stone, adjust the water content by the amount shown in Table 11.3.

A difference in water content of between 10 and 15 ℓ/m^3 will normally halve or double the slump.

Table 11.2: Water requirements of concrete mixes (19-mm stone, 75-mm slump)

Sand quality	Water content, ℓ/m^3	
	Natural	Crusher
Very poor	240	235
Poor	225	225
Average	210	215
Good	195	205
Excellent	180	195

Table 11.3: Adjustments to water content to compensate for stone sizes other than 19 mm

Maximum size of stone, mm	9,5	13,2	19,0	26,5	37,5
Correction, ℓ/m^3	+20	+10	0	-10	-20

15: Calculate cement content, C

C = W ÷ w:c from actions 13 and 14. May be overridden by requirement of action 6.

16: Calculate stone content

The formula for stone content is:

$$M_a = CBD\,(K - 0{,}1\,FM) \;\ldots\ldots\ldots\; (3)$$

where

M_a = mass of stone in one cubic metre of concrete, kg

CBD = dry compacted bulk density of stone, $\mathrm{kg/m}^3$, determined in accordance with SANS 5845 [11.3]

K = a factor which depends on the maximum size of the stone and the workability of the concrete (Values for K are given in Table 11.4.)

FM = fineness modulus of sand determined in accordance with SANS 201[11.4]

Note that there are limits to stone content for workable concrete. At M_a = CBD x 1,0, for example, concrete would be completely unworkable. Approximate maximum M_a = CBD x 0,8 for general-purpose concrete.

If a FA blend is used, calculated stone content may be increased as shown in Table 11.5.

Table 11.5: Increase of stone content when using FA

FA content as % by mass of total cementitious material	Percentage additional stone
15	3
25	4
30	5
40	6
50	7

Table 11.4: Values of K for determining stone content

Approximate slump range, mm	Placing requirement	K				
		Maximum size of stone, mm				
		9,5	13,2	19,0	26,5	37,5
75 - 150	Hand compaction	0,75	0,84	0,94	1,00	1,05
25 - 100	Moderate vibration	0,80	0,90	1,00	1,06	1,10
0 - 25	Heavy vibration	1,00	1,05	1,08	1,10	1,15
60 - 125	Pumped	-	0,83	0,86	0,87	-
25 - 50	Concrete roads*	-	-	-	-	1,2

* Calculated on CBD of 37,5-mm stone when using a blend of 37,5- and 19-mm stone

17: Calculate sand content

The sand content is calculated, assuming full compaction of the concrete, from the fact that volume of concrete is equal to the sum of absolute volumes of cement, sand, stone and water.

For 1 m^3 concrete (ignoring air content):

$$1 = \frac{M_c}{D_c \times 1\,000} + \frac{M_s}{D_s \times 1\,000} + \frac{M_a}{D_a \times 1\,000} + \frac{M_w}{1 \times 1\,000} \quad \text{......} (4)$$

where M and D refer respectively to mass, kg and particle relative density; subscripts c, s, a and w refer respectively to cementitious material, sand, stone and water.

$\therefore$ Ms =

$$D_s \times 1\,000 \left[1 - \frac{M_c}{D_c \times 1\,000} - \frac{M_a}{D_a \times 1\,000} - \frac{M_w}{1 \times 1\,000} \right] \text{......} (5)$$

The effective particle relative density of a blend of fine aggregates of different RDs should be calculated from equation (2) above.

18: Make trial mix

Done in a laboratory. A mix that yields 15 ℓ is normally enough.

19: Measure slump

20: Assess stone content and cohesiveness of concrete

This requires skill and experience. Some techniques are:

- Feel the resistance the concrete offers to the tamping rod when doing the slump test. In a workable concrete, if the slump is correct, the stones should move away easily from the tip of the rod. A harsh grating sound indicates that stone content is too high.
- After the slump has been measured, tap the base plate with the end of the tamping rod. If the concrete is cohesive, it will collapse in a solid mass. If it lacks cohesion, it will break apart as it collapses.
- Smooth the surface of the concrete with a steel trowel. A closed surface without voids should be achieved if stone content is not too high.
- Compact some concrete at least 250 mm deep, eg in a bucket. Scratch the surface of the concrete. Stones should be a millimetre or so beneath the surface. If they are deeper, stone content is too low. If stones protrude above the surface, stone content is too high.

21: Approve mix

Approve the mix if results of actions 19 and 20 are acceptable. Make cubes, preferably for crushing at seven and 28 days, to check that strength development is normal.

22: Reject mix

Reject the mix if results of actions 19 and 20 are not acceptable.

23: Modify design

Reproportion the mix after doing one or more of the following.

- If mix is too stony, reduce stone content by 100 kg/m^3.
- If mix is too sandy, increase stone content by 100 kg/m^3.
- If mix lacks cohesion, do one or more of the following:
  - Use a finer sand.
  - Use a sand with a higher percentage passing the 300-μm sieve. This may be achieved by substituting a finer sand for some of the original sand.
- Use a smaller stone. (If permitted. This will increase water requirement and hence binder content and increase material cost.)
- If mix is too sticky, use a coarser sand or a sand with a lower percentage passing the 75-μm sieve.
- If slump is incorrect, adjust water content. A difference of 10 to 15 ℓ in 1 m^3 of concrete will normally halve or double the slump.

  A practical short-cut method is to start with water content higher than estimated, and then, after measuring slump, effectively reduce water content in stages by adding predetermined increments of sand and stone. Thus the amount of water stays constant but the yield of the trial mix increases.

  In practice, initial amounts and aggregate increments can be easily calculated using a computer spreadsheet program.

  Water content for the required slump may be determined by interpolating between contents straddling the required slump.

No attempt should ever be made to improve workability simply by increasing the amount of mixing water.

24: Specify mix proportions for manufacture of concrete

Based on the outcome of the previous actions. If reactive aggregates have been used, check that adequate precautions are taken against deleterious ASR (see Chapter 10). To allow for moisture in the sand, the mass of sand is increased proportionally and the amount of added water decreased to keep w:c ratio the same.

To calculate the yield of a batch, add the total water content and solid volumes of cement, sand and stone.

11.3 Mix design for special applications

This section gives guidance on the design of mixes for the following:

- Concrete mixes for pumping
- Air-entrained concrete
- High-density concrete
- Low-density concrete
- Mixes for compaction by heavy vibration
- Mixes made with very fine sand
- Shaft linings
- Sliding formwork

11.3.1 Concrete mixes for pumping

The method of design is similar to that used for conventional concrete but, because of certain restrictions imposed by the pump and pipeline system, there are some limitations to the range of materials and concretes that can be used:

Cement content is usually in the range of 250 to 380 kg/m^3, however, concretes containing higher cement contents can be pumped using suitable admixtures, if necessary. All common cements are suitable for pumped concrete. The inclusion of FA may improve pumpability.

Water content is normally about 10 to 15 ℓ /m^3 higher than in conventional concrete when slump is 125 mm.

Sand of good particle shape should be selected. At least 20%, but preferably 30%, of the sand should pass the 300-micron sieve. Sand content, by mass of total aggregate, is generally about 3 to 8% higher than that for conventional mixes, and the fraction of sand by mass of total aggregate is usually in a range of 38 to 55%.

Stone of good particle shape is recommended. The size of stone for 100-mm delivery pipes should be limited to 26,5 mm but 19,0 mm is preferable. The stone content may be estimated using the K factor given in Table 11.4.

Admixtures are beneficial in some instances but they should be tested under site conditions.

A slump range of between 60 and 90 mm is regarded as optimal for piston pumps, and 100 to 125 mm for pneumatic pumps.

11.3.2 Air-entrained concrete (AEC)

It is preferable to use the proportions of a normal concrete mix, made with the same materials and having the same consistence in the fresh state and strength in the hardened state, as a starting point for the design of AEC. To calculate the proportions for the first trial mix of the AEC:

- Reduce the water content by 7%.
- Increase the cement content per m^3 by 20 kg for lean concrete (20 MPa or less at 28 days) to 40 kg for rich concrete (30 MPa or more at 28 days).
- Use the same stone content.
- Include the volume of air expected to be entrained in the calculation of absolute volumes.

The design of AEC may require several trial mixes to verify air content and consistence of the fresh concrete, and strength of the hardened concrete.

11.3.3 High-density concrete

Concrete with a high density may be produced by using high-density aggregates as discussed in Chapter 18.

The methods of design of mixes for high-density concrete are the same as for normal-density concrete.[11.2] High-density concretes are, however, more prone to segregation, particularly if there is a large difference in the particle relative densities of the coarse and fine aggregates and if mixes with a high slump are used. Often a number of trial attempts are required in order to produce a cohesive mix with minimal segregation and bleeding.

11.3.4 Low-density concrete

Because of the difficulty of estimating water contents of mixes containing low-density aggregates, conventional methods of mix design as described in this chapter are usually unsatisfactory. Low-density mixes are often designed on the basis of cement content rather than on w:c.

Useful data on methods of designing mixes for low-density concretes are given in the *ACI Guide for structural lightweight aggregate concrete*[11.5] and in the *FIP Manual of lightweight aggregate concrete.*[11.6]

11.3.5 Mixes for compaction by heavy vibration

Mixes of very low workability that are suitable for intense vibration are designed in the usual manner. Note that:

- Water content for very low workability will be about 20 ℓ/m^3 less than for medium workability.
- Stone content may be determined from equation (3) and Table 11.4.
- The Vebe test may be used to assess consistence.
- The mix should be tested under site conditions.

A method of designing concrete mixes for intense vibration was developed by Stewart and is described in reference 11.7.

11.3.6 Mixes made with very fine sand

In cases where the only available sand is very fine (FM < 1,7), a satisfactory mix may be designed by using one of the following two approaches.

The first approach is to blend 6,7-mm stone with the sand. The ratio of 6,7-mm stone to sand should be such that the FM of the blend is about 2,5. Use equation (3) to determine stone content.

The second approach is to use a blend of two sizes of stone instead of a single size. The smaller stone should be about half of the size of the larger stone. As a starting point, use a ratio of one part of smaller stone to two parts of larger stone.

Insert the CBD for the larger stone together with its K value from Table 11.4 in equation (3) to determine total stone content.

11.3.7 Mine-shaft linings

The use of 13,2-mm stone will in many cases be as economical as 19,0-mm stone in these very wet mixes in a slump range of 150 to 200 mm. Flaky or elongated stone should be avoided. Stone contents are similar to those used for pumped mixes. The sand should have a low water requirement, and sufficient material passing the 600-μm sieve to ensure adequate cohesion.

For these mixes, accelerators and superplasticisers or high-range water reducers are commonly used to reduce the water and cement contents to acceptable levels.

Usually, the strength requirement is an average of 5,0 MPa at eight hours. Suggested starting points for trial mixes are w:c = 0,55 and 0,65 when cements of strength classes 42,5N and higher, respectively, are used with calcium chloride as an accelerating admixture (at 2% by mass of the cement content). The calcium chloride flake should be dissolved as a 30% solution in water before being added to the mix.

The flow table test may be used to assess consistence.

11.3.8 Sliding formwork

The mix is designed using the normal procedure and the stone content is determined in the same way as that for conventional concrete.

The grading of the sand, however, has a significant influence on the success of the slide, and it is recommended that not less than 30% of the sand should pass the 300-μm sieve.

11.4 Designing a mix by eye

This method of mix design is an effective laboratory method requiring an experienced operator to produce a desired mix having characteristics not necessarily covered by conventional methods. It should preferably be done in a laboratory pan mixer having a watertight pan.

The method is especially suited to the design of mixes for:

- Exposed aggregate concrete
- High stone content and heavy vibration
- Special placing conditions such as pumping and placing by tremie
- High-strength concrete

Method

1 . Select w:c.
2. Weigh the water (usually 1,5 or 2,0 kg) and the cement and place them in the mixer.
3. Have ready sufficient sand and stone weighed in containers in easily handled quantities (5 or 10 kg).
4. Add about 5 kg of sand to the pan and start the mixer and add further sand until experience indicates that the mortar part of the mix is still slightly too wet.
5. With the mixer still running, add stone and finally correct the consistence with sand.
6. Record the quantities of sand and stone used.
7. Determine the slump or Vebe consistence, air content and wet density and make the appropriate test specimens.
8. Calculate the mix proportions per cubic metre using the recorded quantities of materials used for the mix and the measured air content and wet density.

11.5 Examples of calculations

Example 1: Determine the stone content of a mix

A concrete for hand compaction is to be made using a 19,0-mm stone with a compacted bulk density of 1 720 kg/m^3. The sand has a fineness modulus of 2,8.

From Table 11.4: K = 0,94

and substituting the values in equation (3):

M_a = 1 720 x (0,94 - 0,1 x 2,8) = 1 135 kg

Example 2: Calculate the particle relative density of a blend of two aggregates with different densities

The blend consists of 60% of aggregate A with a particle relative density of 2,65, and 40% of aggregate B with a particle relative density of 4,55.

From equation (1):

$$D = \frac{100}{\dfrac{60}{2{,}65} + \dfrac{40}{4{,}55}} = 3{,}18$$

(The result is very different from the intuitive [0,6 x 2,65] + [0,4 x 4,55] = 3,41.)

Example 3: Design a trial mix

Given	Cement	Type	CEM I 42,5N
		D_c	3,14
	Sand	D_s	2,65
		FM	2,30
		Quality	Average
	Stone	D_a	2,70
		Size	19,0 mm
		CBD	1 540 kg/m^3
	Degree of control		"Average"
Required	Characteristic strength at 28 days		25 MPa
	Consistence		75 mm slump for moderate vibration

Step 1: W:C
For "average" degree of control, strength margin is 10 MPa (see Table 16.1). For characteristic strength of 25 MPa, target strength is therefore 25 + 10 = 35 MPa. From cement producer,w:c = say, 0,67.

Step 2: Water content
The sand is of average quality.

Either from Table 11.2 or by visual examination of the sand, estimate the water content of the mix, say 210 ℓ/m^3. No adjustment is necessary for size of stone (Table 11.3) because the stone is 19,0 mm, or consistence because the required consistence is 75-mm slump.

Therefore water content = 210 ℓ/m^3

Step 3: Cement content

$$\begin{aligned} M_c &= \frac{\text{Water content}}{\text{w:c}} \\ &= 210/0{,}67 \\ &= 313\ \text{kg/m}^3\text{, say } 315\ \text{kg/m}^3 \end{aligned}$$

Step 4: Stone content
From equation (3) and Table 11.4:

$$\begin{aligned} M_a &= \text{CBD}\ (K - 0{,}1\ \text{FM}) \\ &= 1\,540\ (1{,}0 - 0{,}1 \times 2{,}30) \\ &= 1\,186\ \text{kg/m}^3\text{, say } 1\,185\ \text{kg/m}^3 \end{aligned}$$

Step 5: Sand content
From equation (5):

$$M_s = 2{,}65 \times 1\,000 \left[1 - \frac{315}{3\,140} - \frac{1\,185}{2\,700} - \frac{210}{1\,000} \right]$$

$$= 665\ \text{kg}$$

The quantities per cubic metre of concrete are therefore:

Cement	315 kg
Sand	665 kg
Stone	1 185 kg
Water	210 ℓ

Example 4: Design a trial mix

Given	Cement	Type	CEM I 42,5N
		D_c	3,14
	Sand	D_s	2,65
		FM	1,90
		Quality	Excellent
	Stone	D_a	2,74
		Size	26,5 mm
		CBD	1 460 kg/m^3
	Degree of control		"Good"
Required	Characteristic strength at 28 days		35 MPa
	Consistence		25 mm slump
	Maximum w:c for durability		0,43

Step 1: W:C
For "good" degree of control, strength margin is 8,5 MPa (see Table 16.1).

For characteristic strength of 35 MPa, target strength is therefore 35 + 8,5 = 43,5 MPa.

From cement producer: w:c for strength = 0,60.

But maximum w:c is specified as 0,43; this value must be used.

Step 2: Water content
The sand appears to be of excellent quality, therefore say 180 ℓ/m^3 (for 75-mm slump). See Table 11.2.

Adjustment for size of stone from Table 11.3 is minus 10 ℓ.

Adjustment for slump is minus 10 ℓ (roughly, slump halves for each 10 ℓ reduction).

∴ Water content = 180 - 10 - 10 = 160 ℓ/m^3

Step 3: Cement content

$$M_c = \frac{\text{Water content}}{\text{w:c}} = 160/0{,}43 = \text{say } 370 \text{ kg/m}^3$$

Step 4: Stone content
From equation (3) and Table 11.4:

$$M_a = \text{CBD (K - 0,1 FM)} = 1\,460\ (1{,}06 - 0{,}1 \times 1{,}9) = 1\,270 \text{ kg}$$

Step 5: Sand content
From equation (5):

$$M_s = 2{,}65 \times 1\,000 \left[1 - \frac{370}{3\,140} - \frac{1\,270}{2\,740} - \frac{160}{1\,000} \right]$$

$$= 685 \text{ kg}$$

The quantities per cubic metre are:

Cement	370 kg
Sand	685 kg
Stone	1 270 kg
Water	160 ℓ

The above mix contains 370 kg cement = 370/50 bags = 7,4 bags

The proportions by mass for a one-bag mix are therefore:

Cement	370/7,4	=	50 kg
Sand	685/7,4	=	93 kg
Stone	1 270/7,4	=	172 kg
Water	160/7,4	=	22 ℓ

and yield (per one-bag batch) = 1/7,4 = 0,135 m^3 = 135 ℓ

Example 5: Correct mix proportions for moisture content of aggregates
The steps described in section 11.2.1 are based on the use of dry aggregates. If the aggregates contain moisture this must be taken into account in the proportions for the mix.

The amount of moisture in the stone is usually very small and it is customary to ignore it.

Assume that the sand in Example 4 contains an estimated 4% of moisture by mass. Adjustments are made as follows:

Mass of dry sand = 93 kg
4% moisture = 93 x 4/100 = 3,7 kg, say 4 kg
∴ Mass of damp sand to be used = 93 + 4 = 97 kg
Say gross volume of water = 22 ℓ
Water in sand = 4 kg = 4 ℓ
Net water required = 22 - 4 = 18 ℓ

Example 6: Calculate volume proportions and quantities
If the loose bulk densities of the aggregates are known, in either the dry or damp state, the masses of the materials may be converted to volumes.

In Example 4 above, assume that the loose bulk densities are:

Stone: 1 410 kg/m^3
Sand: Dry 1 640 kg/m^3
Sand: Damp 1 360 kg/m^3 (this lower density is due to the bulking of sand caused by dampness)

The quantities of aggregates per cubic metre are therefore:

Stone: 1 270/1 410 kg/m^3 = 0,90 m^3
Sand: Dry 685/1 640 kg/m^3 = 0,42 m^3
Sand: Damp 1,04 x 685/1 360 = 0,52 m^3

These figures are used for calculating quantities or for ordering materials.

If the damp bulk density of the sand is not known, the dry density may be used and the volume so obtained increased by between 20 and 30% to allow for bulking:

Volume of damp bulked sand = (93/1 640) x 1,20 = 0,068 m^3

Table 11.6: Batching quantities for Example 6

Material	By mass	By volume, aggregates rounded to nearest 5 ℓ	
		Dry sand	Damp sand
Cement	50 kg	1 bag	1 bag
Sand	93 kg	93/1 640 = 0,055 m^3	97/1 360 = 0,070 m^3
Stone	172 kg	172/1 410 = 0,120 m^3	172/1 410 = 0,120 m^3
Water	22 kg	22 ℓ	22 - (93 x 0,04) = 18 ℓ

Table 11.7: Guidelines for selecting cements for concrete

Note: The guidelines given below assume appropriate proportioning of mixes and that good practice is followed.

Application	Comments
Conventional structural concrete in a non-aggressive environment	The cement is normally selected for economy. SANS 50197-1 common cements should be suitable. Site blends of CEM I cement with 50% GGBS or 30% FA have been extensively and successfully used in South Africa. A site blend of CEM I cement and about 8% CSF is technically feasible but there is relatively little local experience of its use. Blends with more than one extender have been used successfully.
Large placements where temperature rise, due to heat of reaction, is to be kept as low as possible	Best results are likely to be achieved with cements with extender contents in excess of 50% GGBS or 30% FA. See section 15.3.2.
Structural precast	Choice of cement will depend mainly on strength requirements at early ages. High early strengths, without steam curing, may be achieved most economically with cements of strength grade 42,5R and higher and with low extender content. Cements with high extender content are better suited to steam curing.
Precast bricks, blocks and pavers (See Chapter 22)	The cement should meet durability requirements, if any, and ensure sufficient strength to allow handling at an early age, typically the day after casting.
High-performance concrete (See Chapter 17)	Strength class should be 42,5N or higher. The inclusion of about 8% CSF is common practice in this application. Other cement extenders may also be used for technical or economic benefits. Superplasticiser is an essential ingredient in high-performance concrete. The compatibility of the specific cement and superplasticiser is important.
Floors, roads and pavements with sawn joints	Concrete for these applications must develop strength rapidly enough to permit joint sawing before the concrete cracks due to restrained drying shrinkage. The mature concrete must have good abrasion resistance. These properties are likely to be achieved most economically with cements of strength grade 42,5N or higher, with extender content not greater than 30%.
Reinforced concrete in marine environment	Based on research done with South African materials,cements containing 30% FA, 50% GGBS or 10% CSF are recommended. See Chaper 9.
Concrete made with alkali-reactive aggregate (See section 10.6)	The total alkali content in pore solution of the concrete should be maitained below limits (See Table 10.1) or the cement should contain not less than 40% GGBS, or 20% FA, or 15% CSF. However the use of 15% CSF usually results in a sticky concrete requiring the use of a superplasticiser.
Concrete exposed to sulphate attack	Caution should be exercised with concrete in sulphate environments and it is recommended that C&CI, specialist literature, relevant codes or a cement manufacturer should be consulted for guidance. Concrete made with 70/30 CEM II/FA blends normally gives better sulphate resistance than an equivalent CEM I concrete.
Extenders should meet the requirements of SANS 1491 parts 1-3[11.8] as relevant	

References

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Chapter 12

Manufacture and handling of concrete

John Kellerman

12.1 Introduction

The manufacture of concrete involves a number of activities. This chapter describes these activities, the sequence of which is shown in Figure 12.1. Formwork and reinforcement are dealt with in Chapters 13 and 14. Quality control of concrete which has to meet strength specifications is dealt with in Chapter 16.

12.2 Planning

Prior to actual manufacture, the principal documents, ie specification, drawings and bill of quantities, must be thoroughly checked. Methods of handling fresh concrete on site, together with any other construction requirements, eg early-age strength for post-tensioning, must also be considered at this stage. These requirements should be

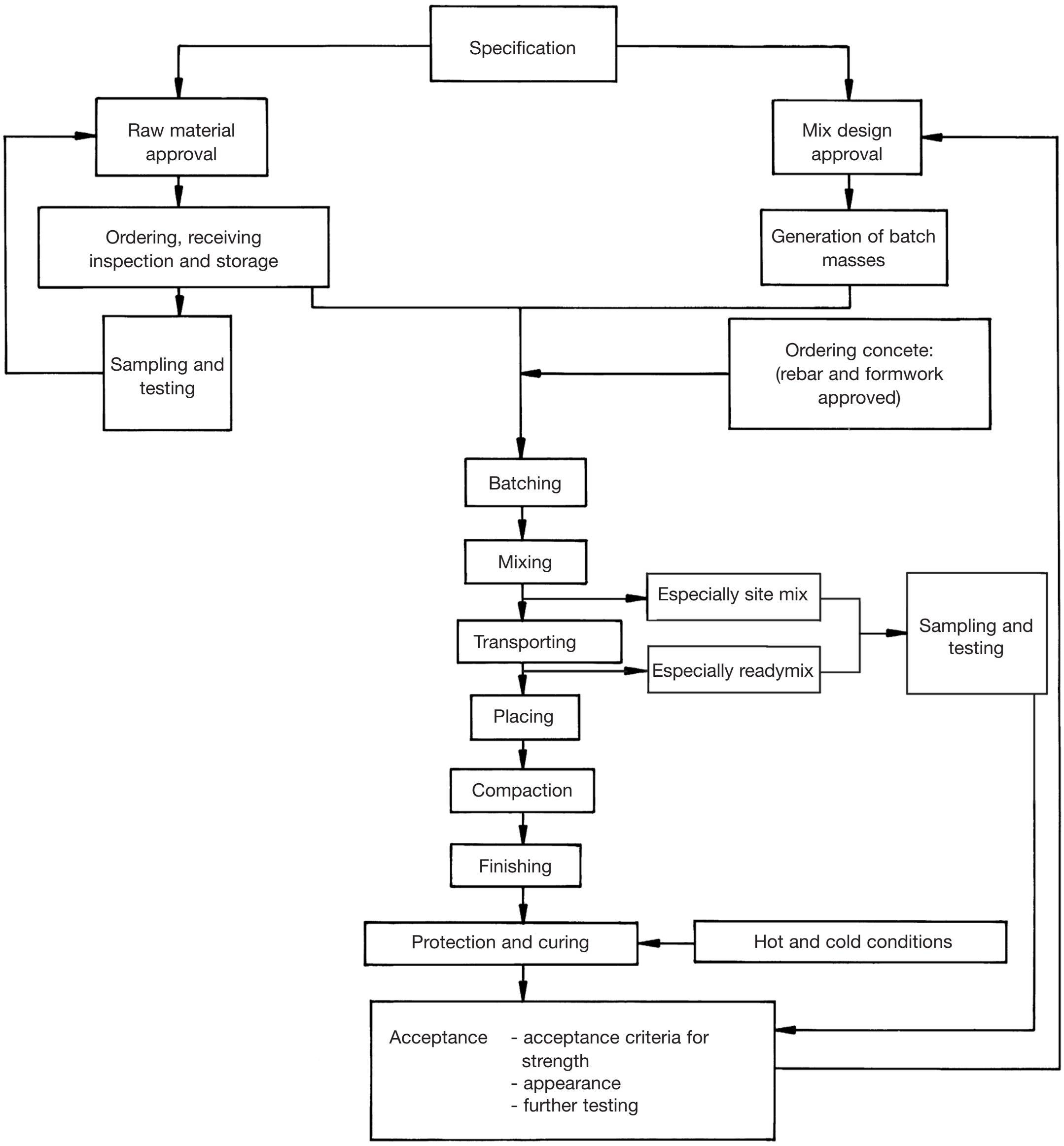


Figure 12.1: Flow diagram of main activities in concrete manufacture

defined by the contractor. Having identified the performance requirements of the concrete, suitable raw materials are selected, mix proportions calculated and trial mixes carried out.

The mix proportions and raw materials will normally require the approval of the client's representative. Samples should therefore be submitted in good time.

Adequate production planning must be done with regard to site layout, raw material handling, concreting methods, placement sizes and maximum outputs, safety, health and training. A suitable quality system must be documented, introduced and monitored.

12.3 Production of concrete

Concrete may be produced by batching and mixing on site or by purchasing ready-mixed concrete. Guidance on the production and delivery of ready-mixed concrete is given in SANS 878.[12.1]

Although, for a specific project, the differences between ready-mixed concrete and site-mixed concrete are negligible, the readymix producer, on a daily basis, has to be able to accommodate:

- A greater variety of plant and production technology
- A wide range and combination of binders, aggregates and admixtures
- Varying mix requirements in both the fresh and hardened states
- A proliferation of specifications
- Differing methods of transporting, placing and finishing
- Restricted delivery times
- Environmental/legal restraints

The advantages of batching and mixing on site are:

- A high degree of flexibility in site management is possible.
- The operation is completely under the contractor's control.
- Small quantities of concrete can be made at short notice.

The advantages of using ready-mixed concrete include:

- The supplier has the resources and technical expertise to provide a wide range of mixes.
- The supplier is able to meet changes to the construction programme.
- Space is made available on the site.
- Concrete can be supplied to several locations on site at the same time.
- High rates of delivery are possible.
- The supplier takes responsibility for quality control of concrete.

The following factors must be taken into account when costing site-batched concrete:

- Concrete materials, including wastage and possible theft
- Handling and storing materials
- Plant hire or depreciation
- Plant operation
- Labour
- Supervision
- Setting up and removing plant and reinstating the site
- Quality control, mix designs, scale calibrations, etc
- Transporting concrete to points on the site that would be accessible to readymix trucks

12.4 Control, handling and storage of materials

12.4.1 Receiving inspection

An appropriate sampling and testing programme must be introduced, together with a system of handling material which is considered to be out of specification when received. Personnel should be assigned to inspect received materials. They must be given details of approved suppliers and the checks that have to be carried out.

For all deliveries, the delivery note must be checked for type, source and quantity of material. Where possible, raw materials should be inspected in the trucks before unloading.

Aggregates should be inspected for cleanliness, size, colour and contamination.

Admixture drums must be clearly identified.

The seals on the inlet and discharge valves of tankers for cementitious binders must be intact, be the correct colour and carry the correct code/serial/delivery number. Care must be taken to discharge into the appropriate silo.

It is also essential to ensure that the vehicles are completely empty after discharge.

12.4.2 Material storage

Materials must be handled and stored in such a way that their quality does not deteriorate.

Aggregates

Aggregates may be stored in separate heaps on the ground, in ground storage bays or overhead bins. In each case it is necessary to control tipping and handling to prevent segregation and intermingling.

Large stockpiles should be built up in horizontal layers with minimal trafficking, not by dumping over the end of the pile.

Stockpiles placed close together require suitable partitions which should be high, long and strong enough to hold at least one full day's production requirements. If possible, a concrete floor, laid with a fall to facilitate drainage, should be provided.

It is advisable to display the various material sizes or descriptions (eg 19-mm stone) on large signs at each bay or bin.

Some form of shelter or protection for the aggregates may be advantageous. For example: where wind-blown dust or sand may contaminate the material; in hot areas to reduce aggregate temperature (combined with water spraying of coarse aggregate); and in cold areas to minimise frosting (combined with steam heating or other methods of raising the temperature).

Overhead bin systems should be equipped with a suitable signal level indicator to prevent running out or overfilling, thus contaminating adjacent bins. Systems in use include rotary paddles, nuclear signals, sonic or optic probes and tilting switches.

Cement

Cement may be delivered in bulk by road or rail tanker or in 50-kg bags.

The handling and storage of cement are discussed in section 1.4.

Note that dissimilar cementitious materials should not be stored in a split silo that has a common filter system. Silos should be clearly marked, especially when cement and cement extenders are used.

Admixtures

The handling and storage of admixtures are discussed in Chapter 5.

Water

Adequate storage facilities should be provided for water, especially when supply is intermittent. Requirements for mixing water are given in Chapter 4.

12.5 Batching

The materials for each batch of concrete must be measured with sufficient accuracy to ensure that the properties of the concrete in both the fresh and hardened state are consistent and meet the specification.

It is important that the batching operator and supervisors should at all times be vigilant for any changes in slump or workability which may indicate a batching error or change in the material properties. Batch operators must therefore be properly trained and adequately supervised.

Batching may be carried out by volume or mass, or by a combination of both. Mass batching is preferred.

12.5.1 Batching by volume

Volume batching is generally restricted to non-structural concreting such as house foundations and other small works not warranting the use of relatively costly mass-batching equipment, and where supervision is poor.

Aggregates

There are two possible sources of error when batching by volume: variations in the amount of aggregate contained in a specific volume; and errors in the measured volume.

- Bulking of sand
  The loose bulk density of a given sand depends on the amount of moisture contained in the sand. Loose bulk density is usually lowest at a moisture content of about 5% and is at a maximum when the sand is completely dry or completely saturated. Bulking, ie increase in volume relative to the volume of dry sand, typically ranges between 15% (for coarse sands) and 30% (for fine sands). A small change in moisture content can result in a disproportionately large change in bulk density.
- Compaction of aggregates
  Aggregates should be batched in a loose state but some compaction may occur inadvertently. The degree of such compaction may differ from batch to batch.
- Packing characteristics of the aggregate
  Rounder and more graded aggregate tends to pack more closely than poorly shaped and/or single-sized material.

Errors in the measured volume can be minimised by proper supervision and the use of gauges which are filled and struck off.

For low-density aggregates, volume batching is generally preferable to mass batching because of the large variation in absorbed moisture of such aggregates.

Cement
Volume batching of cement is subject to serious variation due to changes in its loose bulk density as the cement becomes aerated when handled. For example, while it is assumed that a bag of ordinary portland cement has a volume of 33 ℓ as packed at the factory, this can increase to 38 to 40 ℓ as the cement is poured from the bag or discharged from a silo into the measuring container.

Where nominal volumetric proportions are used to batch concrete, proportions should preferably be based on the use of whole bags of cement.

12.5.2 Batching by mass

When equipment is properly maintained and regularly calibrated, mass batching results in a high degree of uniformity of successive batches of concrete.

Factors that affect the choice of batching system include job size, production rate (both average and peak) and standards required.

The productive capacity of a plant is affected by material handling systems, bin size, hopper size, mixer size and type, and number of mixes.

Different types and layout of equipment are available for mass batching. Cementitious materials and aggregates usually have separate systems.

Figure 12.2 shows a typical batching arrangement using separate systems. Cement is fed by screw conveyor to a hopper and then by gravity to the mixer. Aggregates are fed by gravity into the aggregate hopper and by conveyor to the mixer. Aggregates may be batched individually or cumulatively, the latter being most common. Aggregate hoppers may be charged directly from overhead bins or belt conveyors, or by front-end loader.

Measuring hoppers may use balanced-mechanical-lever, load-cell or hydraulic systems. Irrespective of the system used, the accuracy of measuring equipment should be within the limits shown in Table 12.1. The batching operator should also be able to measure the materials to this accuracy. To assist the operator achieve these tolerances, a remote batching system is preferred.

Remote systems allow the batcher to operate from a more comfortable control cabin situated away from the hoppers. The control cabin should preferably be positioned and arranged so that the operator can see the measuring devices clearly during batching, as well as the charging, mixing and discharge of the mixes without leaving the cabin.

Table 12.1: Accuracy of batching equipment in accordance with SANS 10100-2: 1992[12.2] and BS EN 206-1:2000[12.3]

	Batching accuracy, %	
	SANS 10100	**BS EN 206-1**
Cement	2	3
Aggregate	3	3
Water	2	3
Admixture	2	5

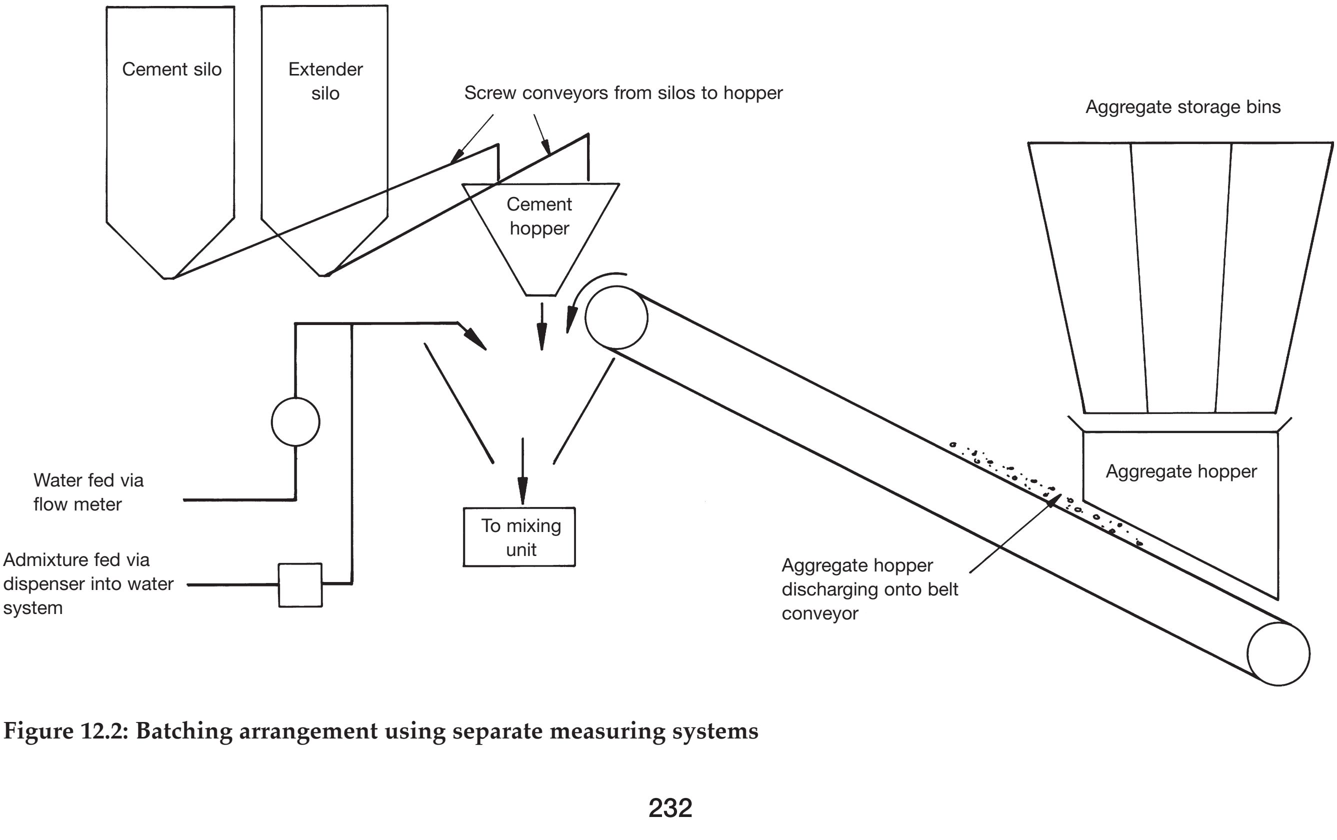


Figure 12.2: Batching arrangement using separate measuring systems

The batching operation may be manual, semi- or fully automatic.

Manual batching is suitable for low production rates, but with increased production, automated systems become justified.

In semi-automatic systems the operator opens aggregate and cement gates using remotely activated controls. Gates are closed either by the operator when the indicator reaches the correct mass or automatically when the required mass has been batched. Automatic systems have one or two starter switches, material flow being cut off when the required mass is reached. Interlocks are included to prevent simultaneous charging and discharging in semi-automatic systems, and to interrupt an automatic cycle when pre-set mass tolerances are exceeded. Effective stock control is possible with systems that record quantities of material batched.

12.5.3 Measurement and control of water

With given proportions of cement and aggregates, the water content will affect consistence. If consistence is well controlled by observation at the mixer, changes in water content will be small and concrete strength will be reasonably uniform.

Water content may be controlled by a combination of:

- Adjustment due to moisture in the aggregate, particularly the sand
- Observation of slump at the mixer
- Monitoring of mixer torque
- Carrying out of regular slump tests

Sand moisture meters may be used, sometimes in conjunction with compensators to take into account changing moisture contents.

Water added at the mixer may be measured by mass or volume, using vertical cylinders with centre syphon discharge, or flow meters. Because flow meters are more affected by dirt and sediment than vertical cylinders, they are less suitable for portable mixing systems.

12.5.4 Batching of admixtures

Admixtures are usually dispensed in liquid form in small quantities ie typically less than 1 ℓ/m^3 for most plasticisers. Inaccurate batching may result in ineffective action or undesirable side effects. It is therefore imperative that admixtures be accurately measured and correctly fed into the mix. The main methods of batching admixtures are:

- Positive displacement volumetric batching using a sight glass into which a measurable volume of the admixture can be pumped
- Positive displacement flow meters similar to water meters
- Timer controlled systems
- Mass batching (on large concrete batching plants). Generally, the admixture should be introduced with the water at the correct point in the loading cycle and there should be full interlocking of water and admixture dispensing to avoid dosage errors.

Dispensers that incorporate a sight glass have the added advantage that operator can see that the correct dosage is being dispensed.

12.5.5 Maintenance and calibration

It is essential that all batching equipment is suitably maintained and that measuring devices are regularly calibrated.

BS EN 206-1[12.3] requires weighing devices to be zeroed daily, checked for accuracy monthly and calibrated quarterly. SANS 10100-2[12.4] states that calibration and tests must be carried out at frequencies that will ensure that the batching system functions effectively and accurately, and requires that hoppers and cement containers are kept dry and clean.

In practice, all measuring devices should be checked at the start of production. Cement scales should be checked daily, other measures weekly. It is usually necessary to install a by-pass for checking liquid dispensers. Cement and aggregate hoppers should be checked for zero daily, and any build-up removed.

Checks on mass meters should be made by placing weights in the hopper over the complete scale range. Volumetric dispensers can be checked using measuring cylinders for small quantities, or by discharging into a tank and weighing for large volumes.

Reasons for inaccuracies of cement and aggregate measuring hoppers may include:

- Knife edges becoming clogged, blunt or unseated
- Incorrect use of counter-balances
- Bent connections binding on a part of the framework
- Dashpot damping devices using incorrect oil
- Spillage of aggregate accumulating under the hopper with integral systems
- Build-up of material in hopper and discharge chute
- Build-up of cement in breather pipe of cement batcher

Batch instructions

On small operations the batch operator may be making only one or two grades of concrete, all of the same batch size. At a readymix plant large numbers of different mixes of varying quantities will be batched. As the mix complexity increases, the necessity for a proper means of communication between user and batcher increases. A system must be developed by the supply operation to ensure that the correct quantities of the correct mixes can be produced and delivered to the correct destination.

It is also necessary to institute a system to check on stock-holding and material usage. An integral part of such a system is a check on the quantity of each mix produced throughout the day.

12.6 Mixing

Thorough mixing is essential for the production of concrete of uniform quality. Mixing must be such that the various sizes of aggregates are uniformly distributed throughout the mix, each aggregate particle being coated with a cement paste of uniform consistence. Mixing may be done by hand or machine.

12.6.1 Hand mixing

Normally this is carried out only on small unimportant jobs where properties of the hardened concrete such as strength, impermeability and durability are not critical. This method of mixing is relatively ineffective and produces variable concrete. It is suited only to concrete having a relatively high slump and is easier when smaller stone size is used.

Mixing should be carried out on a clean non-absorbent mixing platform such as a concrete slab, steel sheet or shutter board.

The specification may stipulate that the concrete be mixed three times dry and three times wet, turning over from one spot to another. Uniformity of colour and texture of the concrete are perhaps the best measure of thorough mixing.

12.6.2 Machine mixing

There are five main types of concrete mixers commonly in use. While their methods of operation differ considerably the following general principles are usually applicable:[12.5]

1. Cement, sand and coarse aggregate should ideally be fed into the mixer simultaneously and in such a manner that the flow of each extends over the same period. The concrete produced in this way is more uniform than that obtained when the ingredients are introduced one after the other.

   With most mixers it will be found that filling the loading hopper in more or less horizontal layers of stone, cement and sand, possibly with another layer of different size of stone on top, provides the best results. Placing of the stone at the bottom of the loading hopper is more likely to result in a self-cleaning process, so avoiding the formation of a hardened layer of material at the bottom of the hopper. The piling-up of any one size of material in the throat of the loading skip should be avoided. With large mixers (3 m^3 or larger), the loading sequence of materials into the mixer is of particular importance.

   Head packing may occur when the finer materials are fed into the mixer first and become lodged in the head of the mixer. If water or cement is fed in too fast or too hot, cement balling may occur.

2. The water should enter the mixer at the same time and over the same period as the other materials.

   When this is not possible, it is advisable to start the flow of water a little in advance of the other ingredients. If all the water is added before or after the other ingredients, the slump of the concrete is liable to vary within the batch. The direction of inflow of water may also influence the effectiveness of mixing.

3. Mixing should continue until the concrete is of uniform consistence, colour and texture. With drum mixers it is an advantage if the mixer driver can see into the drum so that be can observe the consistence and make any necessary minor correction to the water added.

4. The mixer should not be loaded beyond its rated capacity. Overloading results in spillage of materials and slower or incomplete mixing, in addition to imposing undue strain on mechanical parts.

5. The mixer should be set up accurately so that the axis of rotation of the mixer drum is horizontal except in the case of the tilting-drum type. Inaccurate setting may result in poor mixing. Horizontal water measuring tanks and some mass measuring systems are also affected by errors in levelling.

6. The mixer should be operated at the correct speed as stated by the manufacturer. The speed should be checked regularly.

7. Some mortar from the first batch of concrete mixed is left behind on the blades and inside the drum after the concrete has been discharged. To ensure that this first batch is not too stony, the stone content should be reduced slightly or the mixer lined with a mortar of similar proportions to that of the concrete.

8. Hardened concrete adhering to the blades and the inner surface of the drum reduces mixing efficiency. Regular cleaning at the end of each shift or where delays in mixing in excess of about 45 minutes are expected is necessary to prevent concrete build-up, especially if stiff mixes are produced. The mixer can be cleaned by loading it with a quantity of stone and water, running it until the drum and blades are clean and then emptying.

9. Badly worn and bent blades reduce effectiveness of mixing and should be replaced. Adjustable mixer and scraper blades must be kept set to the correct clearances. Wear of the inlet and discharge chutes eventually results in loss of materials. These chutes should therefore be kept in good repair.

10. Rubbing grease or oil over the mixer after cleaning prevents cement building up on the outside of the mixer. Layers of cement are particularly liable to build up in the nose of the loading hopper and should be chipped off regularly.

Uniformity of mixing

The proportions of constituent materials can vary considerably within a batch, but these variations are difficult to measure. Such variations may be due, in rotating drum mixers, to the lack of movement of material from one end of the drum to the other. ASTM C 94M-07C[12.6] requires samples to be taken from the first sixth and fifth sixth of the discharge, and that the differences in the properties of the two samples should not exceed any of the following:

- Density of concrete: 16 kg/m^3
- Air content: 1%
- Slump: 25 mm when average is less than 100 mm; 40 mm when average is 100 to 150 mm
- Aggregate retained on 4,75-mm sieve: 6%
- Density of air-free mortar: 1,6%
- Compressive strength (average of three cylinders at seven days): 7,5%

Mixing time

In order to use the mixer to maximum efficiency it is necessary to know the minimum mixing time necessary to produce a concrete of uniform composition.

The mixing cycle includes:

- The time taken to charge the mixer
- Actual mixing time (measured from when all the ingredients have been added to the mixer)
- Discharge time

Optimum mixing time depends on the type and size of mixer, speed of rotation and method of blending ingredients during charging of the mixer. While mixing and discharge are taking place, the hoppers should be charged with material for the following batch to save time.

The mixing cycle should never be reduced in order to increase production. Reducing the cycle shortens the actual mixing time and this may result in concrete with highly variable workability and strength properties. The time of mixing should not be less than that recommended by the manufacturer of the mixer or than that otherwise determined as being satisfactory. Prolonged mixing may lead to the following:

- Some mix water evaporates from the mix causing a decrease in slump.
- If the aggregate is soft, grinding occurs which produces a finer material and reduces workability.
- Concrete temperature increases due to friction.
- Some entrained air is lost from air-entrained concrete.

Mixer types

Batch mixers producing one batch at a time are most commonly used in South Africa. They include non-tilting drum, tilting drum, reversing drum, split drum and pan. Trough or continuous mixers are rarely used and are not considered in this section.

Non-tilting drum

This type has a single drum that rotates about a horizontal axis. The drum is mounted on rollers and driven via a rack and pinion drive or chain. The mixer is charged from one end and mixing is effected by cup-type blades that lift and drop the material as the drum revolves. The drum is discharged from the opposite end by inserting a chute that deflects the concrete out of the mixer as the concrete falls from the mixing blades.

Drum capacity typically ranges from 300 to 1 200 ℓ.

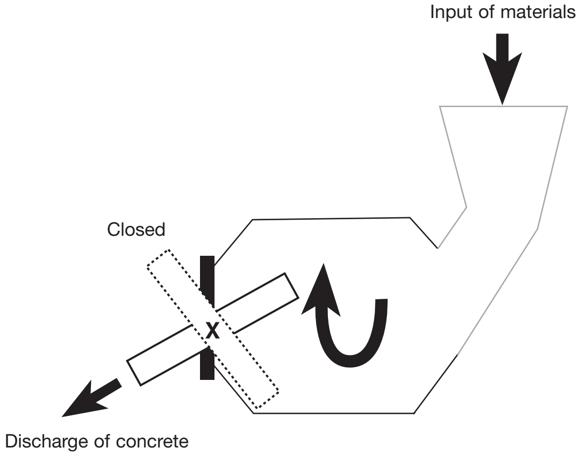


Figure 12.3: Cross section of a non-tilting mixer

Tilting drum

Small tilting drum mixers are commonly used because of their uncomplicated construction and ease of maintenance. Capacities typically do not exceed 200 ℓ, and gauge boxes are usually required for batching. Some larger models are fitted with hoppers for charging the mixer.

Mixing is by lifting and dropping the material, some assistance being given by the drum shape that moves the mass along the axis of the drum.

Tilting the drum discharges the concrete rapidly.

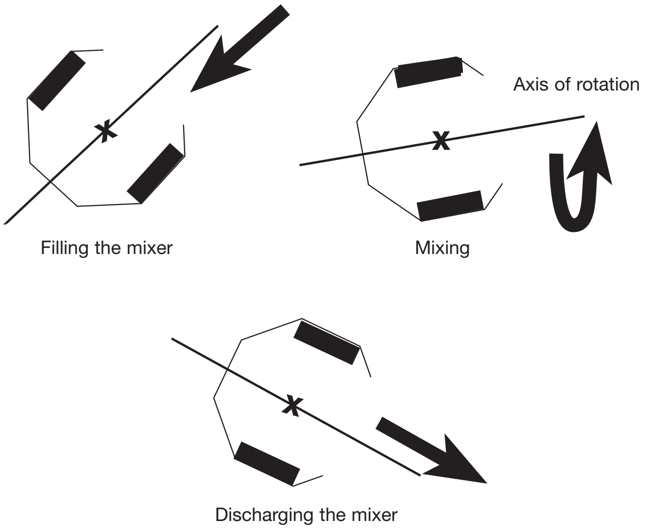


Figure 12.4: Cross section of a tilting mixer

Reversing drum

This is similar to the non-tilting mixer. It rotates in one direction for mixing and in the opposite direction to discharge the concrete. Typical capacity is 400 to 600 ℓ. The drum has two sets of fixed blades, one for mixing and the other for discharging. When the drum is reversed after mixing, the discharge is quick and complete. Mixing times are shorter than for tilting and non-tilting mixers. Very little build-up occurs on the blades.

Reversing drum mixers are usually fitted with integral loading mass-measuring hoppers.

A truncated cone is generally fitted to the discharge end of the mixer to channel the flow of the concrete into wheelbarrows, skips, dumpers, etc.

Truck mixers belong to this category of mixer except that the same opening is used to add constituents and discharge the concrete. The driver can control the speed of drum rotation for mixing, agitation and discharge.

Split drum

Split-drum mixers typically range in size from 500 to 8 000 ℓ.

The mixing drum consists of two halves that separate along a vertical plane, allowing the concrete to be discharged cleanly and rapidly. The two halves are closed during charging and mixing. The mixer is charged through a hole located at one end of the drum.

These mixers combine forced-action and free-fall mixing. They are able to mix all types of concrete and stone sizes efficiently, and are frequently used for projects requiring large volumes of concrete.

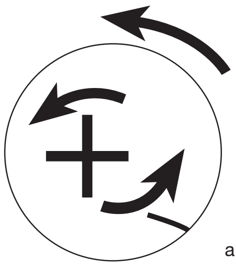
Pan

Pan mixers are available for site use in sizes typically ranging from 200 to about 2 000 ℓ. (Figure 12.5.)

These forced action mixers thoroughly mix even low-slump and lean concrete quickly, with mixing times often well under one minute. Because of this they are ideal for use in precast works.

Increasing use is being made of these mixers in the readymix industry. The capital cost of such mixers is high, and careful and frequent maintenance is needed to ensure efficient operation.

There are two types of pan mixers. One has a circular rotating pan with mixing spiders mounted eccentrically to the pan (Figure 12.5a). The spider may revolve in the same direction as, or counter to, the pan. The discharge doors are located centrally in the floor. The other (Figure 12.5b) has a stationary circular pan with central mixing arms extending from a central gearbox. The mixing blades plough through the materials at high speed. The discharge doors are located on the outer edge of the pan.


a. Counter-current motion

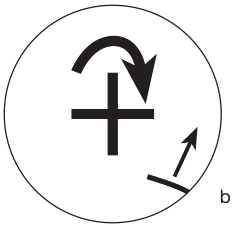

b. Centre shaft

Figure 12.5: Various configurations for pan mixers. The arrows indicate the direction of rotation of the pan, blades, and scraper.

12.7 Transporting

Various methods can be used to transport concrete from the mixer to the point of placing.

Whatever method is used, it is essential that the following points are considered:

- Method of transporting must be appropriate for the type of mix.
- Capacity, ie amount of concrete transported per hour, must be compatible with other operations such as mixing and placing. (A surge bin at the mixer may be used to compensate for differences in capacity.)

- Transport should be rapid enough to prevent drying out or loss of workability, and should be organised to minimise delays during the placing of a lift or section, thus preventing cold joints.
- There should be no contamination of the mix.
- Segregation, including loss of fine material, must be kept to a minimum.

12.7.1 Loss of workability

It is essential that concrete has the required workability at the point of placing to enable full compaction to be achieved.

Many factors that contribute to drying out of the fresh concrete, such as wind speed and temperature, are dependent on weather and are therefore very variable. Simply specifying a maximum time limit between mixing and placing of concrete does not take these variations into account.

The transporting system must be controlled so that transit time is as short as possible. The concrete may be covered to prevent evaporation; or more water may be added during mixing to allow for slump loss during transport. Alternatively, it may be preferable to use a suitable admixture to increase initial workability.

12.7.2 Contamination

Contamination may be caused by foreign matter, concrete left in the transporter from previous use, or wash-down water or rain. The concrete should be covered during rainy weather.

12.7.3 Segregation

The main factors contributing to segregation are lateral flow of the concrete and jolting during transportation. Loss of fines during transportation may result in stony concrete that is more prone to segregation.

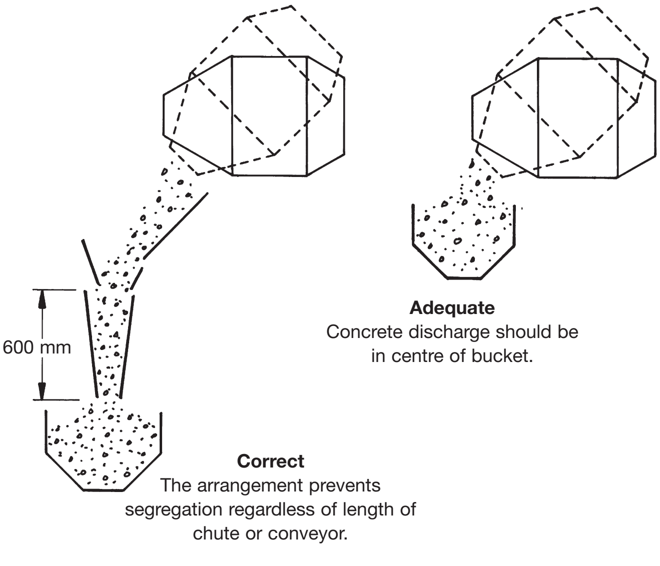


Figure 12.6: Control of segregation as concrete is discharged from mixers[12.7]

Jolting may be minimised using shorter and smoother haulage routes. Adequate maintenance of skip gates, conveyor belt wipers, tipper lorry tailgates, etc will minimise loss of fines. Figure 12.6[12.7] shows the preferred method of discharge from a mixer to minimise segregation at this point.

12.7.4 Selecting the method of transporting

When selecting a suitable method for transporting concrete on site, consideration must be given to the following:

- Site conditions: location of trees, overhead powerlines, adjacent buildings, effect of noise, terrain, gradient, access, size and layout of site
- Equipment availability: the use of equipment such as tower cranes for handling formwork and reinforcement as well as fresh concrete
- Continuity of concreting
- Volume and rate of largest placement
- Use of site-mixed or ready-mixed concrete
- Season of construction: requirements for hot or cold weather, and rain

12.7.5 Types of equipment

Transport equipment (see Table 12.2) can generally be categorised as follows:

- Linear: rope and pulley; hoists
- Two-dimensional: all wheeled transport, conveyors, aerial cableways
- Three-dimensional: cranes, concrete pumps, fork-lift trucks

Linear and two-dimensional systems often need a secondary handling method to complete a transporting process.

12.8 Placing concrete

A basic requirement of placing is to maintain quality and uniformity of the concrete. Operations should be planned so that the required amounts of consistent concrete can be placed. Equipment for placing should be well maintained, and arranged so as to deliver the concrete as close as possible to its final position without segregation, and without damaging or displacing reinforcement, stressing ducts, formwork, etc.

Segregation is particularly liable to occur when concrete, being discharged from a skip, chute or conveyor, is allowed to drop continuously and collect in a heap. It also commonly occurs when concrete is discharged from the ends of chutes or conveyors. Control of placing in these conditions is shown in Figure 12.7.[12.7]

Table 12.2: Methods and equipment for transporting and handling concrete[12.8]

Equipment	Type and range of work for which equipment is best suited	Advantages	Points to watch for
Truck agitator	Used to transport concrete for all uses in pavements, structures and buildings. Haul distances must allow discharge of concrete within three hours, but limit may be waived.	Truck agitators usually operate from central mixing plants where quality concrete is produced under controlled conditions. Discharge from agitators is well controlled. Concrete is homogeneous on discharge.	Timing of deliveries must suit job organisation. Concrete crew and equipment must be ready on site to handle large volumes of concrete.
Truck mixer	Used to mix and transport concrete to job site over short and long hauls. Hauls can be almost any distance if water is added and concrete mixed on arrival on site, or suitable retarding admixtures used.	No central mixing plant is needed. Concrete is completely mixed in truck mixer and only a batch plant is needed. Discharge is same as for truck agitator.	Control of concrete quality may not be as good as with central mixing. Slump tests of concrete consistence are needed on discharge. Timing of deliveries to suit job organisation must be accurate. Concrete crew and equipment must be ready on site to handle large volumes of concrete.
Non-agitating truck	Used to transport concrete on short hauls.	Capital cost of non-agitating equipment is lower than that of truck agitators or mixers.	Concrete slump should be limited. There is a possibility of segregation and leakage of fines.
Mobile continuous mixer	Used for continuous production of concrete at job site.	This is a combination of a materials transporter and a mobile mixing system for quick, precise proportioning of specified concrete. The system may be operated by one man.	Trouble-free operation requires good preventive maintenance programme on equipment. Materials must be identical to those in original mix-design proportioning.
Tower and mobile crane		Cranes can handle concrete, reinforcing steel, formwork, and sundry items in high-rise concrete-framed buildings.	Careful scheduling between concreting and other operations is needed to ensure both availability and high utilisation.
Skips or buckets	Used on cranes and cableways for construction of buildings and dams. They convey concrete directly from central discharge point to formwork or to secondary discharge point.	Skips and buckets enable full versatility of cranes and cableways to be exploited and have a wide range of capacities. Discharge is clean.	Select skip capacity to conform with size of concrete batch and capacity of crane and placing equipment. Discharge should be controllable.
Hoist	Used for small quantities of concrete. Passenger hoists can carry 0,5-m^3 skips at speeds of up to 85 m/min.	Hoists can handle concrete and other materials and are often used in conjunction with cranes. They may discharge into a wet hopper.	They require a secondary handling system, eg barrows, to distribute horizontally from discharge point.
Barrows and dumpers	For short flat hauls on all types of on-site concrete construction especially where accessibility to work area is restricted.	Barrows and dumpers are very versatile and therefore ideal on sites where placing conditions are constantly changing.	Barrows are slow and labour intensive.
Chutes	For conveying concrete to lower level, usually below ground level, on all types of concrete construction.	Chutes are inexpensive and easy to manoeuvre. No power is required and gravity does most of the work.	Slopes range between 1 to 2 and 1 to 3. Chutes must be adequately supported. Provide baffles and a downpipe at the discharge end to prevent segregation.
Belt conveyors	For conveying concrete horizontally or to a higher level. They are usually used between main discharge point and secondary discharge point and are not usually suitable for conveying concrete directly into formwork.	Sophisticated belt conveyors have adjustable reach, travelling diverter and variable speed, both forward and reverse. They can place large volumes of concrete quickly when access is limited.	End-discharge arrangements are needed to prevent segregation and mortar being left on return belt. In adverse weather (hot, windy) long reaches of belt need cover. Maximum slopes are about 30%.
Pneumatic guns (see also Chapter 23)	Used where concrete is to be placed in difficult locations and where thin sections and large areas are needed.	They are ideal for placing concrete in free-form shapes, for repairing and strengthening buildings, for protective coatings and thin linings.	Quality of work depends on operator skill. Only trained nozzlemen should be employed.
Concrete pumps and pneumatic placers	Used to convey concrete directly from central discharge point to formwork or to secondary discharge point.	Can be used to place concrete in inaccessible places. Pipelines take up little space and can be readily extended. Deliver concrete in continuous stream. Mobile boom pump can move concrete both vertically and horizontally. Use of distributing systems such as Krete-placer increases flexibility and reduces labour for changing pipelines on large slabs.	Constant supply of fresh concrete is needed at required consistence without any tendency to segregate. Ensure an even flow of concrete through the pipeline and clean out thoroughly at the end of each operation. Pumping distance is considerably reduced by pumping vertically, around bends and through flexible hose.

Important requirements to be checked during placing include the following:

- Concrete must be deposited as near as practicable to its final position and not deposited in heaps.
- Concrete must be deposited in horizontal layers of a depth that allows for full compaction to take place (typically less than about 450 mm). Each layer must be thoroughly compacted before the next layer is placed.
- Concreting must be carried out continuously to avoid cold joints and horizontal or nearly horizontal placing lines on vertical surfaces.

Figure 12.8 shows various methods of placing concrete under different conditions.

12.8.1 Placing concrete under water

Placing concrete under water is normally carried out using a tremie, as shown in Figure 12.9. Using this system, a plug or go-devil is put in the mouth of the hopper and concrete placed on top of it. The plug, made of plastic foam or wet cement sacks, is forced down the pipe by the mass of concrete, expelling water and air at the bottom. As the concrete flows out it forms a mound around the bottom of the pipe, preventing water from entering the pipe. It is important to keep the end of the pipe below the surface of the concrete as the lift proceeds to prevent the ingress of water. If water enters the pipe, another plug must be installed and concreting started again.

Concrete placed using this method must be cohesive and highly workable. The cement content of such mixes is normally in excess of 350 kg/m^3.

12.8.2 Grouted concrete

Grouted concrete is produced by placing coarse aggregate in a form and filling the inter-particle voids with a cement-sand grout, the grout being pumped into position through small-diameter pipes.[12.10]

Grouted concrete differs from conventional concrete in that the coarse aggregate particles are deposited with point-point contact, resulting in a concrete with substantially reduced shrinkage. It is therefore suitable for situations where its good dimensional stability can be exploited and where the placing of conventional concrete would be impracticable, ie underwater concreting, underpinning of foundations, repair and remedial work.

Materials

Coarse aggregate should be graded to have a voids content of less than 50% and a maximum size of 13,2 mm.

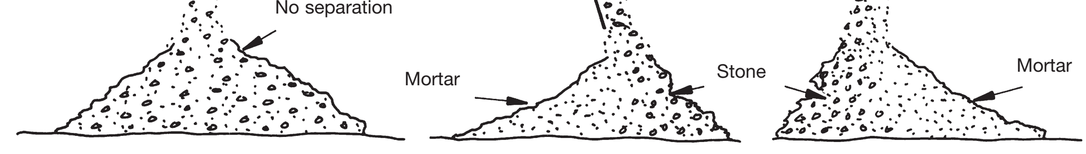
Fine aggregate should be well graded with 100% passing the 2,36-mm sieve.

Binders should be common cements complying with SANS 50197-1 or suitable blends of CEM 1 cements and FA or GGBS complying with SANS 1491.

Grout fluidising admixtures may be incorporated in the grout to reduce the accumulation of bleed-water under coarse aggregate particles, increase the w:c ratio at a given fluidity and retard stiffening, the latter providing additional handling time for the grouting cycle.

Grout properties

The w:c ratio usually varies between 0,40 and 0,50 by mass for normal-strength concrete. The mix should be



Correct
The arrangement prevents segregation of concrete whether it is being discharged into vehicles or forms.

Incorrect
Improper or complete lack of control at end of belt or chute. Usually a baffle or shallow hopper merely changes the direction of segregation.

Figure 12.7: Control of segregation of concrete at the end of conveyor belt or chute[12.7]

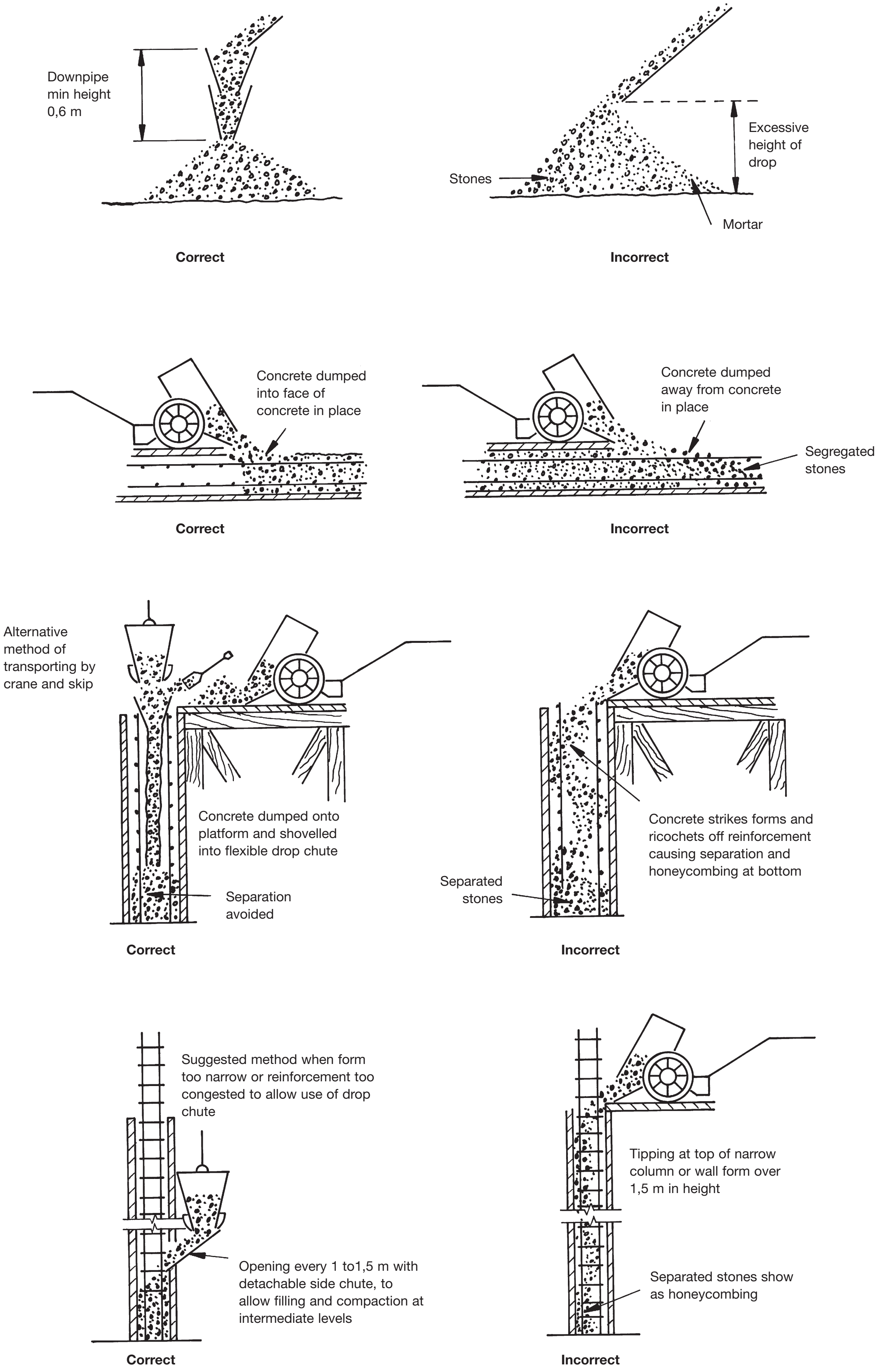


Figure 12.8: Methods of handling mixed concrete[12.9]

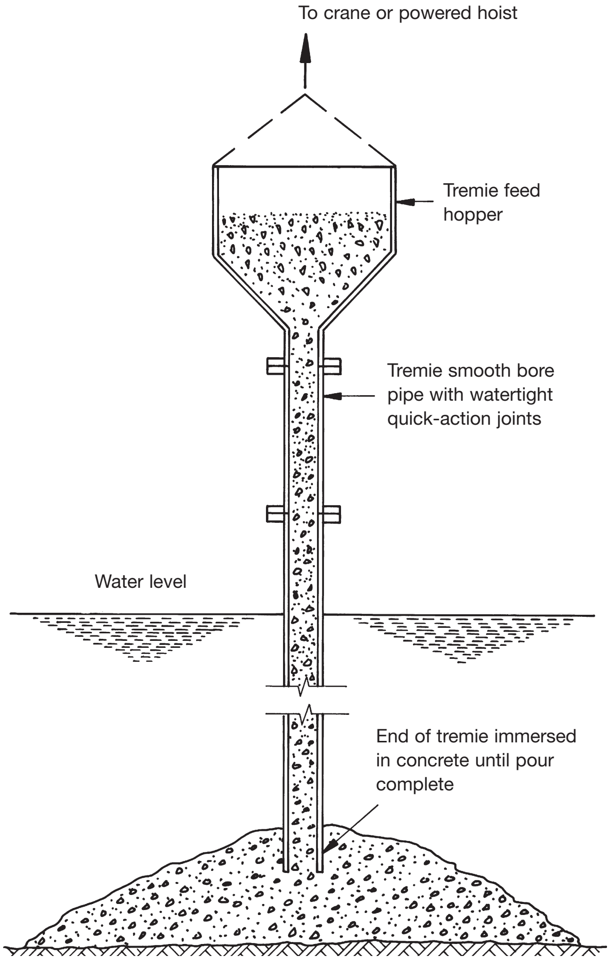


Figure 12.9: Underwater concreting

proportioned to produce a grout of the required consistence (typically a flow of 20 to 24 seconds) and strength after injection into the coarse aggregate. Bleeding should be limited to 0,50%.

12.9 Compaction

Concrete is compacted to expel entrapped air, thereby achieving maximum density, strength and impermeability. In properly cured concrete, full compaction ensures that the maximum potential strength will be achieved. Air voids include both entrapped air and voids formed by the drying out of excess water. For every 1% excess voids, the concrete strength is reduced by approximately 5 to 6%.

Full compaction will also result in minimum permeability (when the concrete is fully cured) and a smooth surface finish against well-assembled formwork.

During compaction, reinforcement must not be disturbed and the formwork should not be damaged or displaced.

The effort required to compact concrete depends on its workability. The workability of the fresh concrete must be compatible with the method of compaction used.

Compaction may be done without or with mechanical vibration.

12.9.1 Compaction without mechanical vibration

Methods of hand compaction include rodding, tamping or spading.

In order to fully compact a mix using these methods, workability must be high, with slumps in excess of 75 mm. To prevent segregation, the mix should have a higher-than-normal mortar content, ie less stone.

If the workability is high enough, concrete may be self-compacting and even self-levelling. The use of high workability concrete (slumps in excess of 200 mm) is now becoming more common in many horizontal and vertical applications on normal construction sites. Vertical forms can be filled without the need to limit each placement height prior to vibration (if vibration is necessary); rate of placement is greatly increased and less labour is required to level and compact horizontal surfaces. Correct mix design is essential for such concrete (see Chapter 20).

12.9.2 Compaction with mechanical vibration

Mechanical vibration, in most cases, is the most effective way of compacting concrete. Stiff mixes can be compacted using mechanical vibration. Such mixes have relatively low paste contents which result in improved dimensional stability of the hardened concrete.

With all methods of vibration the following precautions should be observed:

- Forms should be tight fitting to avoid grout loss.
- Placing increments should be shallow enough to allow complete compaction to be achieved, typically 350 to 450 mm deep.
- Vibration should not be prolonged in any one zone because this may bring excessive laitance to the surface; a vibration time of 10 to 15 seconds using an internal vibrator is usually sufficient for compaction.

Vibration is normally applied internally by poker vibrators, or in some cases externally by vibrating forms or tables and vibrating screeds.

Internal vibrators

Internal vibrators, also known as poker or immersion vibrators, are highly efficient because all their energy is transmitted directly to the concrete.

They consist of cylindrical vibrating units driven through flexible hoses, and are powered by petrol engines, electric motors or compressed air.

Head diameters range from 25 to 150 mm, with 60 mm being most common.

Work carried out by Latham[12.11] showed that, as a rule, head diameter should be increased with decreasing workability and increasing aggregate size. The radius of action and rate of concrete compaction increases with increasing head diameter, while frequency diminishes.

The following rules should be observed when compacting by internal vibration:

- Vibration must be systematically carried out.
- The vibrator should be inserted as close to vertical as possible, at points about 450 to 550 mm apart or not further apart than eight times the diameter of the poker.
- The whole length of the vibrator should be immersed. (This will also help to keep the bearings cool.)
- The vibrator should be immersed to the full depth of the concrete and should penetrate the layer of fresh concrete below.
- Vibration should continue only until compaction is complete. This is indicated by air bubbles ceasing to rise and the appearance of a sheen on the surface of the concrete.
- The vibrator should be inserted quickly so that the concrete is compacted from the bottom up to allow the air to escape easily.
- The vibrator should be removed slowly to ensure closing up of the concrete behind the head as it is withdrawn.
- The vibrator should never be used closer than about 100 mm to the form face.

Form vibrators

Form vibrators may be either table vibrators, used for small precast elements, or external vibrators rigidly clamped onto the forms. Their action transmits energy indirectly to the concrete and they are thus less efficient than internal vibrators. The concrete is compacted to only a limited distance from the form face. Forms must be strong enough to allow such vibration without leakage or deformation.

Vibrators should be spaced 1 to 2 m apart; deep elements may require vibrators at more than one level. Concrete should be compacted in layers about 150 mm deep. Blow-holes are likely to occur when using form vibrators, particularly near the top of a lift.

Portable vibrating hammers may also be used against formwork in areas where the concrete cannot be compacted by internal vibration.

Surface vibrators

Concrete roads and floor slabs may be compacted by vibrating beams applied directly to the surface of the concrete. Double vibration beams range in length from 4 to 12 m.

Concrete is placed between firmly fixed and levelled side forms with a surcharge of about 20% of the depth (see Figure 12.10). The surcharge should be maintained in front of the forward beam as shown to ensure that compaction is being achieved over the width of the slab.

These vibrators are suitable for compacting slabs up to 150 mm in depth. For deeper slabs, concrete should be compacted with a poker vibrator, or in layers not exceeding 150 mm using a notched beam vibrator.

Poker vibrators are often used in conjunction with a beam vibrator to fully compact concrete around the edges of the slab.

Vibrating plates and vibrating rollers may be used on very stiff mixes.

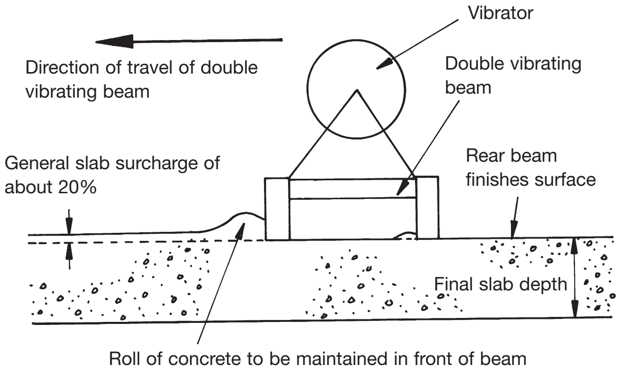


Figure 12.10: Maintaining a surcharge when using a beam vibrator

12.9.3 Revibration

Revibration, typically one to two hours after placing the concrete, is an effective way of eliminating defects caused by bleeding and restrained settlement. The compressive strength of the concrete is also likely to be increased slightly, especially of concrete subjected to excessive bleeding and settlement of the solids. Revibration is not harmful provided the concrete is still plastic enough to respond to vibration.

12.10 Working the surface of concrete in the "green" state

In the transition from a plastic, workable material to a strong, rigid material, concrete is said to pass through the "green" state. While concrete is green, ie a weak solid, its surface may be worked in different ways.

At a stage when the concrete is taking its final set, wearing surfaces of floors and pavements can be made skid-resistant by brushing, or be densified by means of trowelling, which improves the abrasion resistance of the surface. (See section 12.14.)

As the concrete hardens after final set, mortar can be removed from the surface by means of wire brushing or washing with water, or a combination of washing and brushing. In doing so, a rough texture with exposed coarse aggregate can be achieved. This technique is best suited to top surfaces (ie not formed by formwork or a mould) and may be used to prepare construction joints to create architectural finishes, especially in precasting. (See section 12.15.3.)

The timing of these activities is critical. The concrete must be at the correct stage of hardening to minimise damage while making it possible to achieve the desired effect. No rules-of-thumb can be given because timing is affected by a large number of variables. Trials should be carried out on the concrete at regular intervals to assess hardening.

12.11 Protection and curing

Once the concrete has been placed and compacted it is necessary to:

- Immediately protect it from rapid drying out and extremes of temperature, and from damage, eg by foot or vehicular traffic, or rain.
- Cure it to maintain a satisfactory moisture content and temperature in the concrete during its early stages so that desired properties may develop.

In practice, protection and curing may be a continuous process.

The section deals with protection and curing of conventional construction at normal temperatures. Protection and curing in cold and hot conditions are dealt with in sections 12.12 and 12.13, respectively. The control of temperature in mass concrete is dealt with in Chapter 15.

12.11.1 Protection

Factors influencing protection requirements

- Low relative humidity, high wind speed and high ambient and concrete temperatures result in rapid evaporation of water from the surface of the concrete. This phenomenon is especially troublesome in elements with a large exposed surface, eg concrete slabs. The loss of moisture causes shrinkage, which may result in cracking while the concrete is still plastic and so permanently impair the quality of concrete.
- Concrete containing CSF exhibits only minimal bleeding and requires early protection to prevent plastic-shrinkage cracking.
- Surfaces accessible to pedestrians and vehicles should be barricaded to prevent damage.

Techniques to prevent or reduce plastic-shrinkage cracking

- Erect windbreaks and sunshades when slabs are placed in exposed conditions.
- Protect concrete with moisture-retaining coverings during delays between compacting and finishing.
- Use a fog spray above the surface to maintain a water sheen on the concrete before final finishing and the start of curing.
- Carry out concreting operations during the cooler part of the day.

12.11.2 Curing

The need for curing

Since cementing reactions can take place satisfactorily only where capillaries are filled with water, it is essential to keep the mixing water in the concrete. For concrete with a water:cement ratio of less than about 0,5, additional water should be provided at the surface of the concrete. It is also important to initiate curing as soon as the concrete surface is exposed to the atmosphere since, if the start of curing is delayed, calcium hydroxide is deposited in the entrances of the capillaries by the evaporating pore water. This calcium hydroxide is then carbonated by carbon dioxide in the air, sealing the capillaries and making it very difficult to get water back into the concrete to replace the evaporated water. This reduces the degree to which cementitious materials can react. Curing by intermittent wetting has a similar effect.

Reducing the degree of cementing reactions in concrete results in a very significant loss of quality. This applies particularly in the important cover layer of concrete which has to resist chemical and physical attack and which must be as impermeable as possible to reduce the rate of carbonation of the concrete and protect the steel reinforcement from corrosion.

Figure 12.11 indicates the effect of the duration of damp curing on the strength of concrete.[12.12]

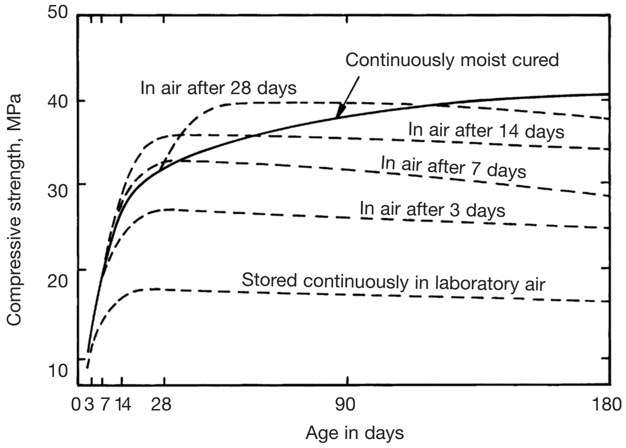


Figure 12.11: Compressive strength of concrete dried in laboratory air after preliminary moist curing[12.12]

The process of curing should continue until specified requirements are met.

Factors influencing curing requirements

- Cements containing extenders such as FA, GGBS and CSF react more slowly than portland cement on its own, particularly in cold weather. Concrete made with such cements should therefore be cured thoroughly and for a longer period than concrete made with CEM I cement. This is particularly true if the potential durability benefits which can be achieved with these extenders are to be achieved.

- Extremes of temperature have negative effects. The lower the temperature, the slower the rate of cementing reactions and the longer the concrete must be cured to achieve the required degree of reaction. When temperatures are high, the rate of reaction is high and relatively high early strengths are achieved, but the long-term strength and durability are reduced. If possible, concrete temperatures should be controlled to avoid these negative effects. Ambient temperatures of 15 to 25°C are generally considered to be most suitable for concreting operations.

Curing practice

Some curing methods are given in Table 12.3. Curing methods such as immersion, ponding or continuous spraying are effective but are seldom practicable. When water curing, care must be taken to avoid thermal shock, eg by putting cold water onto a warm concrete surface.

When a structural element is covered with polythene sheeting, the edges of the sheeting must be firmly secured to prevent wind blowing under it. Sheeting in contact with the slab surface, especially if the surface is pigmented, may cause local discolouration. The use of a layer of damp sand to cure slabs may also cause staining.

Formwork left in place reduces the loss of moisture from surfaces in contact with the forms. Surfaces not in contact with the forms, eg the tops of columns or walls, must be suitably covered to prevent moisture loss. If formwork has to be struck the day after placing concrete, further curing, eg the application of a curing membrane, should be carried out immediately.

Table 12.3: Guide to on-site concrete curing[12.13]

Type of construction	Curing material	Application
Road and airfield pavements	Pigmented resin-based curing compound with high efficiency rating	Immediately after finishing process is complete. (Concrete may have to be protected from rapid drying out before it sets to prevent plastic-shrinkage cracking and excessive heat build-up.)
Floor slabs	Curing compound or polythene or other impervious sheeting	As above
Tops of beams and columns	Polythene or other impervious sheeting material	Immediately after finishing process is complete
Concrete columns, beams, walls, etc in hot dry conditions	Resin-based curing compound	Immediately formwork is removed (Some curing compounds may cause debonding of subsequently applied finishes.)
	Polythene or other impervious sheeting	Apply in close contact with surface immediately formwork is removed
	Formwork itself	Leave undisturbed for at least seven days and longer in cold weather
Formed, permanently exposed concrete sections cast in cold weather	Insulation	As soon as concrete is placed, and maintain for at least seven days
	Delayed removal of formwork	
Large concrete sections with a minimum dimension exceeding 1m	Top surface insulation Delayed removal of formwork or replacement of formwork by insulating material	Maintain for at least seven days and until internal temperature gradient is less than 20°C
Some manufacturers of resin-based curing compounds recommend the re-wetting of formed concrete surfaces before application. To avoid thermal shock, the temperature of water used for this purpose must be close to the temperature of the surface of the concrete.		

Curing compounds are membrane-forming liquids sprayed onto the surface of the concrete to inhibit evaporation of water from the concrete. They are applied as soon as bleeding has stopped and bleedwater has evaporated from the surface.

The choice of curing compound is especially important when the concrete is to receive further treatment such as plastering or painting. Some compounds will prevent subsequently applied materials from adhering.

Curing methods to be adopted should be precisely defined in the specification and provision for payment made in the bill of quantities, preferably as a fixed extra-over rate per square metre.

Duration of moist curing

The period of curing cannot be simply prescribed; curing of concrete should be continued until the properties of a particular concrete are developed to the required extent. Unfortunately, there are as yet no simple in-situ tests to determine whether the required characteristics have been met.

ACI 308R-01[12.14] prescribes minimum times which are dependent on the cement type; BS 8110-1[12.15] also considers the influence of average temperature on curing periods; Table 12.4 gives suggested minimum moist curing periods for concrete, but does not differentiate between the various cement types.

Table 12.4: Suggested minimum moist curing periods[12.16]

Weather		Minimum moist curing period, days
Normal:	18 to 22°C 65% RH Low wind speeds	5
Hot:	With drying winds	7
Cold:	5 to 12°C	9

In practice, limited if any attention is paid to curing concrete. The inclusion of curing as a separate pay item in the bill of quantities could contribute to an increased awareness of the importance of curing and encourage good curing practice.

Steam curing

The rate of strength gain of concrete can be accelerated by increasing the curing temperature. Curing in steam at atmospheric pressure is considered as a special wet-curing process.

Steam curing is used mainly in the precast industry to obtain high early strength to enable products to be handled soon after casting. Steam curing is normally done in special chambers or tunnels, or covers may be placed over the members and steam supplied under the covers.

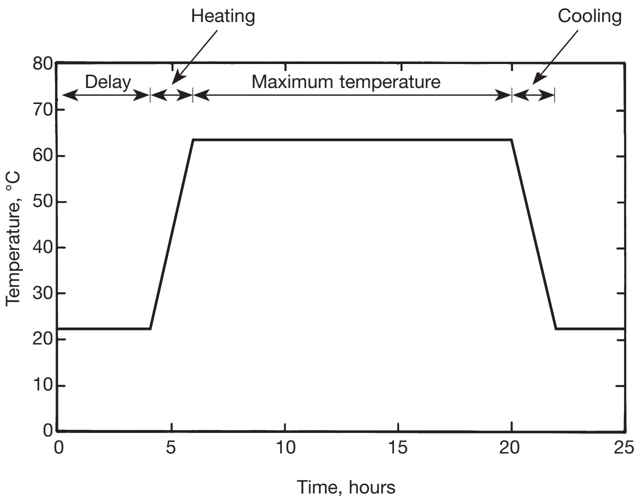


Figure 12.12: Typical steam-curing cycle

Curing cycles are a compromise between early and late strength requirements and time. A typical curing cycle is indicated in Figure 12.12.

A delay of three to five hours before increasing the temperature is required to prevent excessive expansion due to the high pressures set up in the pores by the rising temperature while the concrete is too weak to resist this pressure. This expansion results in excessive pore sizes and weaker and less durable concrete. Surface cracking can also occur. In addition, the gypsum in the cement is less soluble at high temperatures and forms an impervious shell around the cement grains. This reduces hydration and the benefit from the high temperature.

The temperature is typically increased to between 60 and 70ºC, at a rate not exceeding 25ºC/hour, and maintained for the required period. Elements should then be allowed to cool to within about 20ºC of ambient temperature at a rate not exceeding 25ºC/hour.

Each mix, type of cement and concrete element has its own optimum regime, which needs to be determined experimentally.

Autoclaving, ie high-pressure steam curing, is outside the scope of this book.

12.12 Concreting in cold weather

Weather is described as cold when the ambient temperature is 5ºC or less. Cold weather is seldom experienced in southern Africa and expensive preventive measures such as heating aggregates are seldom necessary.

The effect of concrete freezing at early ages depends on whether the concrete has set, and what strength the concrete has attained when freezing occurs.

If concrete which has not yet set is allowed to freeze, an increase in the overall volume of the concrete occurs due to the expansion of water, especially in capillary pores. When thawing takes place, the concrete will set with an enlarged volume of pores, which will reduce strength and durability.

If freezing takes place after the concrete has set, but before it has gained sufficient strength (about 3 to 5 MPa), expansion associated with the formation of ice will cause disruption of the microstructure and irreparable loss of strength and durability.

Once the concrete has achieved a compressive strength of at least 3 to 5 MPa, it can resist a freezing cycle without damage. This is because it has a higher resistance to the pressure of ice and because a large part of the mixing water will either have become combined with the cement or will be located in gel pores and will be unable to freeze.

Consideration should be given to the following when concreting in cold weather.

12.12.1 Cement type

Because of their slower setting and rate of strength gain, the use of highly extended common cements or the partial replacement of CEM I cement with GGBS or FA is not recommended. It may be advantageous to use CEM I 42,5R or 52,5N cement in preference to 42,5N.

12.12.2 Aggregate protection

Water in aggregate may be prevented from freezing by covering stockpiles with tarpaulins. If aggregates are likely to become frozen, or contain ice or snow, they may have to be heated. This may be achieved using steam injection or hot air blowers. When using steam heating, adequate drainage must be provided. Typically aggregate should be heated to between 10 and 20ºC.

Aggregate stockpiles should be oriented in order to be exposed to the warming effect of the morning sun.

12.12.3 Lagging of water pipes

All water pipes must be adequately lagged to prevent supply pipes from freezing or even bursting.

12.12.4 Heating the mixing water

The most economical and easiest way to heat concrete is to heat the mixing water, but care must be taken not to exceed 60 to 70ºC. At these temperatures, flash setting of the cement and reduced workability may occur.

12.12.5 Batching and mixing

Adequate protection must be given to the batching and mixing plant, as the concrete temperature can drop significantly if the equipment is very cold.

12.12.6 Concrete temperature

The minimum concrete temperature as mixed should be higher in colder conditions but may be reduced for concrete placed in larger sections.[12.17] For air temperatures between -18 and 0ºC recommended concrete temperatures vary between 7 and 18ºC.

The temperature of concrete at the time of placement should however not exceed the minimum recommended temperatures by more than 11ºC, as concrete placed at low temperatures (5 to 13ºC), protected against freezing and given adequate long-term curing develops a higher ultimate compressive strength.

12.12.7 Transporting and placing

As significant heat losses occur during these processes, they must be carried out quickly. Unless the concrete is adequately protected, methods of transport such as conveyors and chutes are not recommended.

Care must be taken to ensure that the fresh concrete is not deposited against frozen surfaces.

12.12.8 Protection and curing

The main requirement is to prevent heat loss of the freshly placed concrete. Under no circumstances should water-curing methods be used. Heat may be retained by using insulated forms, covering exposed surfaces with insulating material or the erection of covers with internal heating.

Formwork and props must be left in place longer than in normal weather.

Pedestrian and vehicular traffic will also have to be kept off slabs for longer periods. It is important that the heat generating process does not give rise to high carbon dioxide concentrations in the vicinity of the fresh concrete and cause rapid carbonation of the concrete.

12.13 Concreting in hot weather

While ambient temperatures above 32ºC are considered to constitute hot weather, conditions can also be treated as hot weather when the temperature exceeds 25ºC and any of the following factors are present:

- Low relative humidity and high wind velocity
- Solar radiation
- High concrete temperatures

These tend to impair the quality of fresh and hardened concrete.

During hot weather the rate of slump loss and the incidence of plastic-shrinkage cracking tend to increase, setting time decreases, water requirement for a given consistence increases; while early-age strengths tend to be higher, long-term strengths and impermeability are significantly lower. Figure 12.13 indicates that water content needs to be increased to maintain a given consistence as concrete temperature increases. Figure 12.14 shows the effect of elevated concrete temperature on both the early and long-term strength gain. Cements containing extenders such as FA or GGBS tend to be less affected than PC mixes.

The following steps can be taken to control concrete temperatures:

- The time of day can have a significant effect and, where possible, concreting should be confined to the cooler parts of the day.
- Aggregates should be kept as cool as possible. Stockpiles should be shaded from direct sunlight and coarse aggregate sprayed with water to assist cooling but drainage must be adequate. (Spraying fine aggregate is ineffective and may lead to problems in controlling the moisture content.)
- Because cement has only a limited effect on concrete temperature, cement delivered hot is not a serious problem. It probably is worthwhile painting silos white to reduce temperature rise.
- Mixing water should be stored in such a way as to be as cool as possible. Storage may be below ground or in tanks painted white and shaded. In some instances it may be necessary to further cool the water. This may be done by refrigeration or by injecting liquid nitrogen into water tanks. Liquid nitrogen can also be injected directly into the concrete mixer or truck mixer to reduce concrete temperatures very rapidly. Alternatively, crushed ice may be used as part of the mixing water to reduce concrete temperatures.
- The batching and mixing plant should be shaded as far as possible and painted white. The plant should be designed to minimise raw material handling times; fast and efficient mixers are preferred.
- Transport time must be kept to a minimum. The concrete may be covered with damp material, and readymix vehicles or pump pipelines sprayed with water to cool them. Pipelines may also be shaded.
- A suitable retarder may be used in the mix. Many retarders also reduce the water content of concrete and so may permit either a reduction in cement content or a higher initial slump for the same water content.
- Aggregates should be selected and the mix designed to minimise the water content and thus the cement content. Some of the cement may also be replaced with an extender such as FA or GGBS.
- As far as is practicable, the area to be concreted should be sheltered from high winds and direct sunlight. Formwork may be cooled by occasionally spraying it with water.

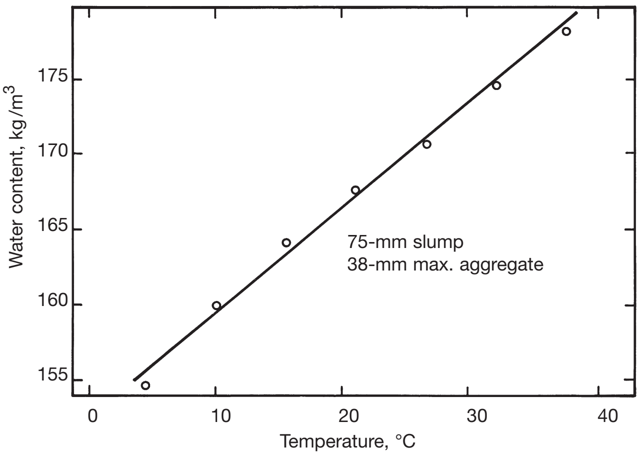


Figure 12.13: Effect of temperature increase on the water requirement of concrete[12.18]

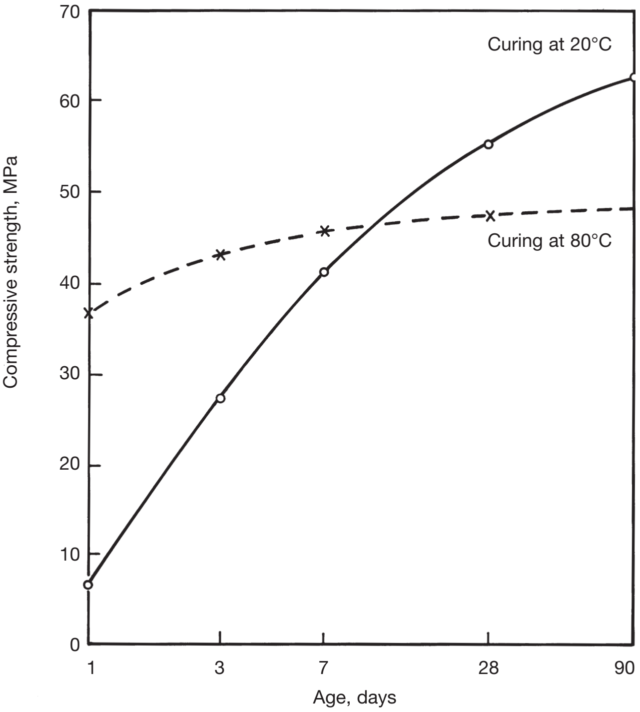


Figure 12.14: Effect of curing temperature on strength of concrete at ages of 1 to 90 days (for concrete with a w:c ratio of 0,47)[12.19]

- Placing and compacting must be carried out carefully and systematically to minimise the formation of cold joints. Adequate in-use and stand-by equipment must be available.
- Curing and protection must be done in such a way as to minimise heat gain.

12.14 Finishing concrete

When concrete is placed below ground or encased by other materials, its appearance may be of little importance. Where the concrete is to be visible, however, it must be aesthetically acceptable and, in some cases, have required surface texture and adequate abrasion resistance. This requires close co-operation between the architect, engineer and contractor. Many unsightly features result from uncontrolled cracking, poor placing especially at construction joints (see section 12.15), poor formwork, patching and lack of cover to reinforcement. With care during all phases of design and construction, the above defects can be avoided.

Smooth finishing of horizontal surfaces such as power-trowelling of industrial floors is considered in detail in reference 12.20.

Alternatively it may be necessary to provide the surface of a flat slab with a suitable non-slip finish. This may be done by wood floating, by brushing with a stiff broom, by dragging hessian over the surface of the concrete or by exposing the aggregate. These techniques should be carried out after bleeding has stopped and surface bleed water evaporated but while the concrete is still plastic enough to receive the finish. Wetting the concrete in order to apply the finish will substantially reduce the quality of the surface.

Some of the more common types of special or architectural finishes may be classified as follows:

- Off-shutter finishes. These rely entirely on the form face for shape and texture. Form faces include steel, smooth timber, rough boards, absorbent linings such as soft-board and hessian, and profiled or sculpted form liners made of rubber or plastics. No further treatment of the concrete surface should be undertaken other than the making good of small blemishes such as blow-holes.

- Indirect or secondary methods. These methods include exposing the aggregate by such methods as:
  - Pre-placing the coarse aggregate
  - Washing and brushing
  - Tooling
  - Water- and grit-blasting
  - Surface retardation

With pre-placed aggregate, the stone is densely packed in a bed of sand and a thin layer of backing mortar is applied to embed the particles to about three quarters of their depth. After the mortar has stiffened for 30 to 40 minutes the backing concrete is applied and compacted. When the form is removed, the sand is brushed away from the embedded stone.

Washing and brushing is done with a fairly stiff broom, on concrete cast face-up, and is carried out when the concrete has stiffened sufficiently, typically 2 to 4 hours after placing. Care must be taken to ensure an even exposure of the coarse aggregate. If a special aggregate is to be used, this may be spread over the freshly placed concrete and pushed into the surface. When the concrete has been cured, the wet concrete surface should be washed with a 10% solution of hydrochloric acid to remove any lime bloom or cement from the aggregate, and then thoroughly rinsed to remove all unreacted acid.

Tooling is done on hardened concrete. The simplest form of tooling is by hammer and chisel. Usually, however, it is done with a needle gun or bush hammer. The mortar is removed to the required depth with some damage to coarse aggregate surfaces. It is advisable to leave a margin around edges and corners of a treated area to prevent spalling. Tooling does not mask inadequacies caused by poor formwork joints or honeycombing and in many instances it accentuates them.

Water- and grit-blasting may be used to remove the surface of hardened concrete. The degree of removal is normally classified as brush, light, medium or heavy. As with all types of surface removal, great care must be taken to obtain an even surface texture. These techniques normally cause roughening, and a consequent change in appearance, of the coarse aggregate.

In some cases it may be preferable to paint a suitable surface retarder directly onto the formwork. This retards the setting of the concrete for a few millimetres from the surface, which can be washed and brushed after removal of the formwork. Timing is critical for all indirect or secondary finishing methods. It is advisable to carry out trials to determine optimum timing.

12.15 Construction joints

Joints are provided in structures either to afford a convenient stopping point during construction, or to allow relative movement between various parts of the structure. They are a major cost factor during construction and, if poorly designed, detailed or constructed, may affect the strength, serviceability and durability of the structure.

This section considers construction joints where no relative movement is allowed between the adjacent sections. For construction and other joints in floors, see reference 12.20.

Construction joints or day-work joints are necessary because there is a limit to the amount of concrete which can be economically placed in one operation. Construction is therefore carried out in lifts, panels or bays.

Lift, panel or bay size is influenced by availability and re-use of formwork, location of joints for aesthetic purposes, the amount of concreting that can be carried out in one day and other construction considerations. Interruptions in the concreting operation, if long enough to prevent knitting together of the fresh and previously placed concrete, also require the formation of a construction joint.

12.15.1 Positioning

Positions of construction joints are normally as follows:

- Junction between top of column or wall and slab or beam: at level of soffit of slab or beam
- Junction between top of base or slab and column or wall: at level of top of base or slab; and/or 75 mm to 150 mm above top of base or slab
- In beams and slabs: at cross sections where bending moments and shear stress are low
- In high walls and columns: at convenient height-intervals

12.15.2 Featuring joints

If the concrete is to remain visible, construction joints in walls should preferably be featured. Suitable methods of featuring these joints are shown in Figure 12.15 but care must be taken to provide adequate cover to reinforcement (see Chapter 14).

12.15.3 Forming

A construction joint is a potential plane of weakness in a structure and every effort must be taken to ensure maximum continuity across the joint. The main principles of constructing suitable construction joints include:

- Simple butt joints should be used. Joggled or keyed joints should be avoided except in floor slabs on the ground.
- The coarse aggregate must be exposed on the surface to ensure the best possible bond to subsequently placed concrete. This is best achieved on horizontal surfaces by brushing off surface laitance and mortar about two to four hours after compaction of the concrete. At this stage it is easily done, and does not loosen nor damage the coarse aggregate. As the concrete becomes harder, other methods of surface removal will be necessary. Wire brushing can be used on concrete less than about 18 hours old. Methods such as scabbling or grit-blasting must be used on older concrete, but care must be taken to avoid cracking or spalling the concrete.

On vertical stop-ends use may be made of a paint-on surface retarder to assist in roughening the concrete surface when the stop-end is removed. Alternatively, using expanded metal or chicken wire allows a proportion of mortar to pass through, providing a rough surface which requires no further treatment.

- Irrespective of the method of surface roughening it is necessary to remove all loose particles and dust by water or air jetting.
- Research by Brook[12.21] has shown that, irrespective of surface preparation, best results are obtained by placing concrete against a dry joint surface.
- The application of a layer of mortar to the joint just prior to concreting is not recommended. The mortar would need to be well brushed into the surface and this is nearly always impossible to do. Pre-placing buckets of mortar in a column or wall section is not generally recommended although it can sometimes reduce the risk of honeycombed concrete at horizontal construction joints.
- Particular attention must be paid to correct placing and compaction procedures to ensure the soundest possible concrete at construction joints. Reducing the stone content of the layer of fresh concrete in contact with the joint by about 25% may help.

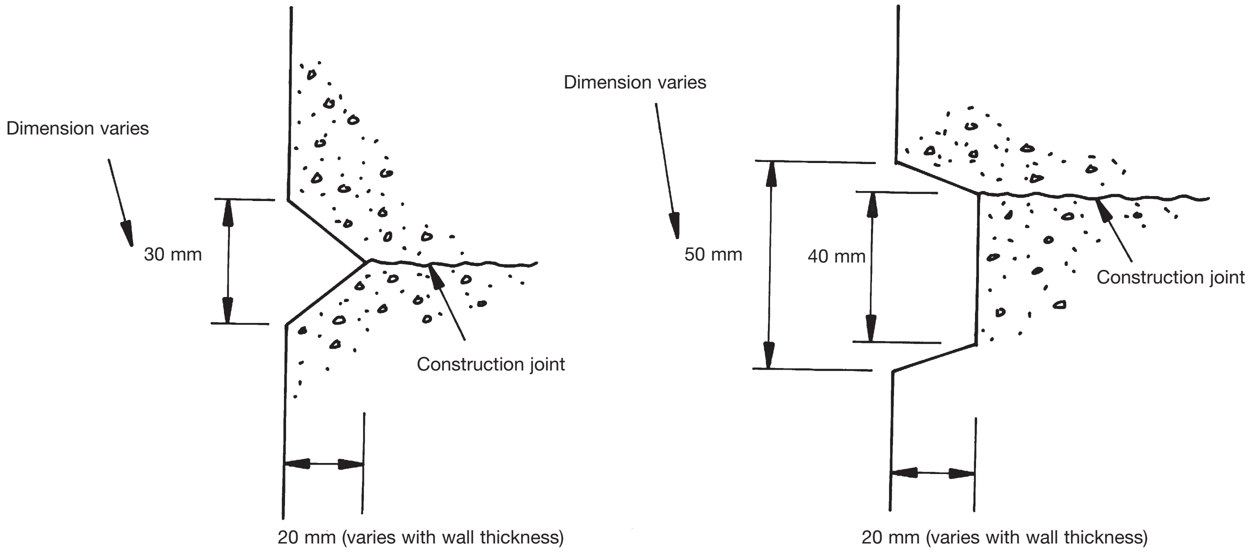


Figure 12.15: Methods of featuring construction joints

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Chapter 13

Formwork

Callie Middel

13.1 Introduction

Concrete is a material which, at the time of placing, has no shape of its own but can be moulded into a wide variety of shapes. At this stage, the concrete is "plastic." In most applications, formwork is used to shape the concrete.

Formwork is defined in BS 6100-9 *Building and civil engineering. Vocabulary. Work with concrete and mortar*[13.1] as a "structure, either temporary or permanent, provided to contain fresh concrete and support it in the required shape until it has hardened."

The focus of this chapter is on temporary formwork which is dismantled to reveal the cast concrete. Since many terms used in formwork are specific to the subject, a glossary of terms is included as section 13.10.

The scope of this chapter does not include design. It is of critical importance that formwork be designed so that it meets the requirements of the structure. All the major formwork suppliers offer a comprehensive design service and it is strongly recommended that they be consulted.

13.2 Purpose and requirements of formwork

The purpose of formwork is to:

- Contain freshly placed and compacted concrete until it has gained sufficient strength to be self-supporting.
- Produce a concrete element of the required size and shape.
- Produce the desired finish to the concrete.

Formwork (materials and labour) accounts for 35 to 60% of the cost of in-situ structural concrete. The speed of erection and striking fundamentally affect the rate of construction, thus the contractor must use considerable skill in the design, fabrication and handling of formwork.

Where the formwork is supported from below, the structural support is called falsework.

The general design and construction requirements of formwork involve:

- Shutter strength. Soffit shuttering has to withstand both dead loads and live loads. Wall and column shuttering must resist hydrostatic pressure and possibly wind loads
- Accuracy. Dimensions must fall within specified tolerances.
- Rigidity of shutter. This must be maintained within specified dimensional tolerances.
- Concrete should be contained without contamination occurring, eg through the use of excessive or inappropriate release agents.
- Formwork durability should be such that the formwork produces satisfactory concrete surfaces for the intended number of formwork uses.
- There must be no loss of the constituents of the concrete through joints in the formwork.
- Uneven absorption by the form face may be unacceptable for high quality finishes.
- Panel sizes should allow for safe and easy handling using equipment available on site. The design should permit an orderly and simple method of erection and striking, to prevent damage to both the formwork and the concrete.
- Panel arrangement should be such that panels are not trapped during striking; eg beam side forms must be able to be removed without disturbing the soffit formwork.
- Formwork should (normally) be reusable.

13.3 Types of formwork

A wide variety of modular formwork systems is available with specific applications for walls, columns or suspended floor slabs. Most of these systems are generic and skills can easily be transferred from one system to the next. A resin-coated timber plywood lining mounted inside a modular steel frame is commonly used by most contractors.

The main objective of the formwork manufacturer is to provide a system which complies with standards of accuracy and requires fewer man-hours per square metre to erect. The contractor aims to use as little equipment as possible with the shortest cycle time, thereby reducing the time-related costs of the formwork.

13.3.1 Permanent formwork

Permanent formwork remains in position and becomes part of the structure. Examples of permanent formwork include precast concrete elements, natural stone, masonry and glass-reinforced cement. Ribbed interlocking galvanised decking systems, which become part of the structure, also constitute permanent formwork.

13.3.2 Temporary formwork

Temporary formwork is used for most concrete construction. A large variety of materials including steel, timber and wood-based materials, plastics and fabrics may be used in the manufacture of formwork. Other materials such as treated polystyrene, fibre board, fabric and rubber may be used as form-face materials.

Most temporary formwork systems consist of a modular matrix of panels that can be assembled in a variety of combinations. These combinations of panels, together with accessories for connecting, tying and supporting them will form the planned construction. Timber or steel panels are used for special manufacture of non-modular in-fills and for patching on walls or suspended slabs. Tables 13.1 and 13.2 give a brief summary of vertical and horizontal formwork systems.

13.3.3 Slip-forming

Slip-forming or sliding formwork has been used since the 1930s and evolved from the use of timber forms with manual screw jacks to steel systems with hydraulic jacks. The term slip-forming applies to vertical shaft construction as well as horizontal paving operations. Only vertical shaft formwork is considered in this chapter.

Slip-forming becomes viable for silos and shafts over 25 m high. It involves the use of equipment similar to that for wall formwork with two 1,2-m high steel panels attached to a yoke connected to two strong-backs. Steel rods of approximately 25 mm in diameter run vertically up the construction with hydraulic jacks connected to each rod. Each jack contains two sets of tungsten steel wedge bits which catch and release the rods with every adjustment. The yoke and strong-backs with their attached platforms

Table 13.1: Vertical formwork systems

Vertical formwork			
Type	**Framed panel wall formwork, Figure 13.1**	**Steel panel wall formwork, Figure 13.2**	**Variable girder wall system, Figure 13.3**
Application	All types of walls for continuous adjustment on site from one pour to the next.	Gang-forming for walls with few adjustments from one pour to the next	Special shapes or repetitive use with architectural finishes.
Form-face material	Timber ply 18 mm thick.	Steel plate 4 mm thick.	Hardwood ply 21 mm thick.
Finish	Fair-face finish with frame joints visible on concrete face. Joints can serve as a feature.	New panels provide a high-quality finish, but older panels become dented with a resultant rough finish.	Smooth fair-face finish.
Assembly	Purpose-made wedge clamps hold panels together. Quick adjustment of panel combinations.	Panels are connected by tees and wedges, with channel walers and strong-backs transferring tie forces to the formwork face.	By a skilled carpenter in a workshop or on site on a flat level surface.
Design	On-site layout by experienced user or by the formwork designer for more complex structures.	Formwork design required for concrete pours over 3 m high.	By the formwork designer taking into account the pour height and rate of rise.
Handling	Some units can be manhandled, but are mostly moved by crane.	Manhandled for foundations and walls not exceeding 3 m high.	Crane handling only.

Table 13.2: Horizontal formwork systems (beams and slabs)

Horizontal formwork			
Type	**Panel slab formwork, Figure 13.4**	**Conventional steel slab formwork, Figure 13.5**	**Variable girder slab formwork, Figure 13.6**
Application	Suspended floor slabs from 150 to 900 mm thick. Very well suited to parking decks.	Suspended floor slabs and coffer or trough slabs.	Suspended floors from 150 mm to 1m thick. Easy adjustment for different slab thicknesses by the expansion or contraction of the grid layout.
Form-face material	9-mm resin-coated ply which beds into a steel or aluminium frame.	Steel decking pans of 2,5 mm thick. Fibre glass or plastic moulds for coffers and troughs.	Resin-coated ply 21 mm thick.
Finish	Varies from gloss to matt with new and old panels respectively, with modular panel joints visible.	New panels provide a high-quality finish, but older panels become dented with a resultant rough finish.	Fair and smooth with resin-coated ply, and timber veneer for boards without resin coating.
Erection	Fast up to any height with little labour required.	Decking onto birdcage scaffold with a larger labour component.	A mobile tower is required for heights exceeding 3,5 m.
Striking	Quickstrip system with drophead lowering beam and panel.	Quikstrip system with double-headed jack and spindle combination that lowers beams and decking panels.	Non-quickstrip system with temporary props inserted before striking.

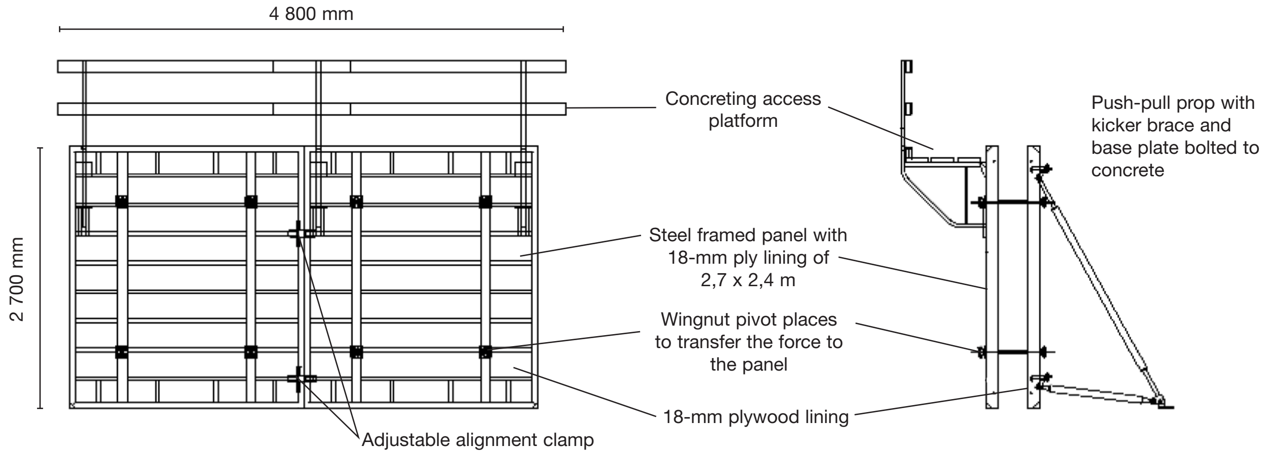


Figure 13.1: Framed panel wall formwork system

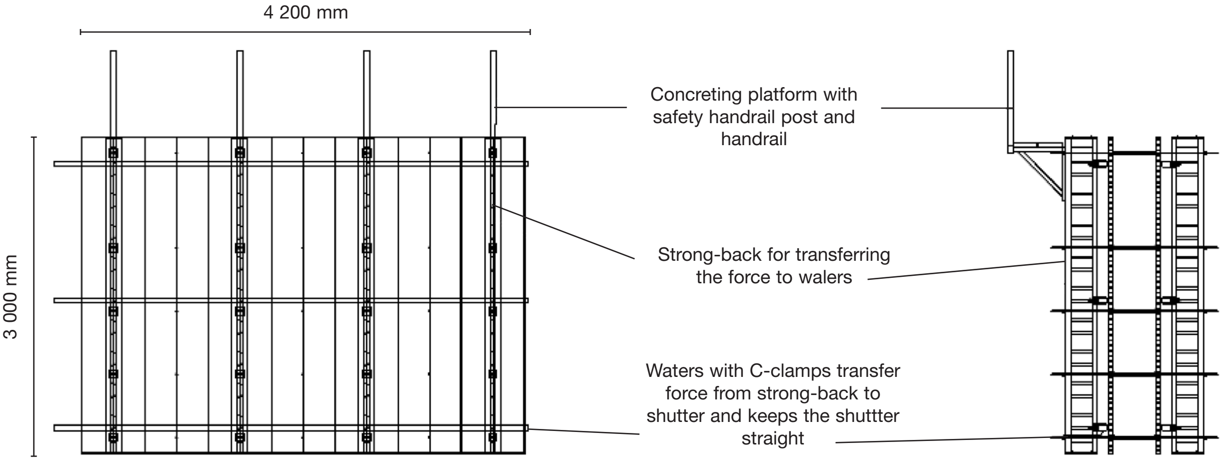


Figure 13.2: Steel panel wall formwork

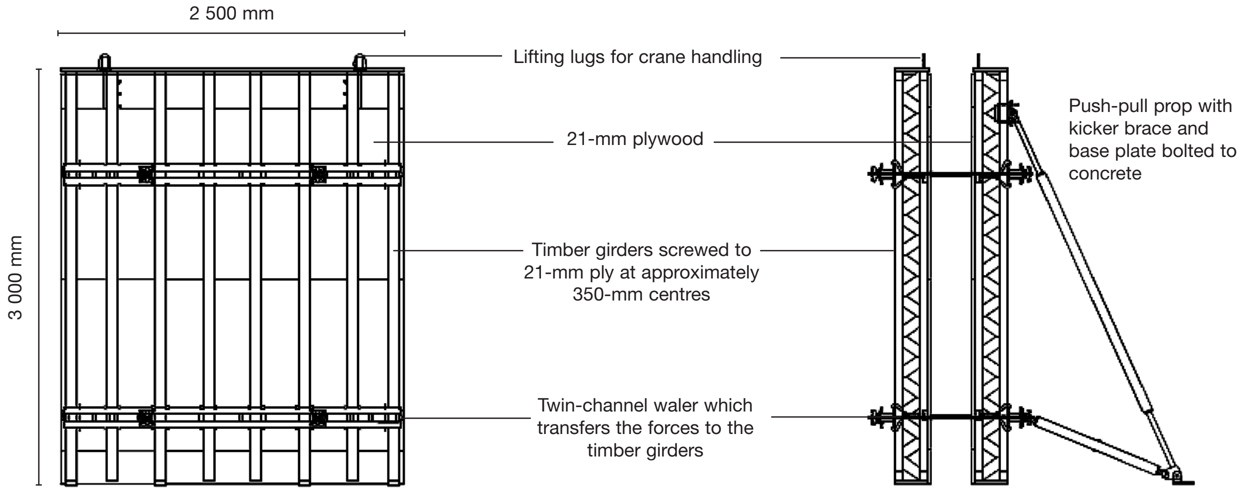
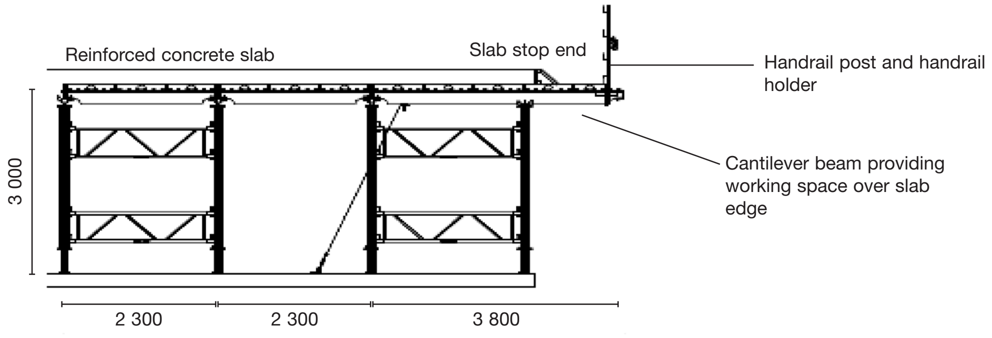


Figure 13.3: Variable girder wall formwork system



Panels supported by beams (panels are 1 500 x 750 mm)

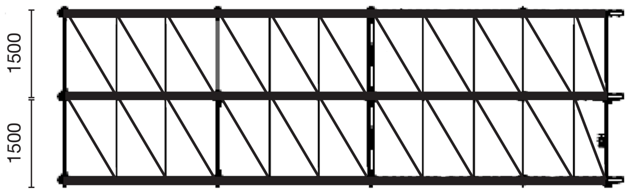


Figure 13.4: A modular decking system (all dimensions in mm)

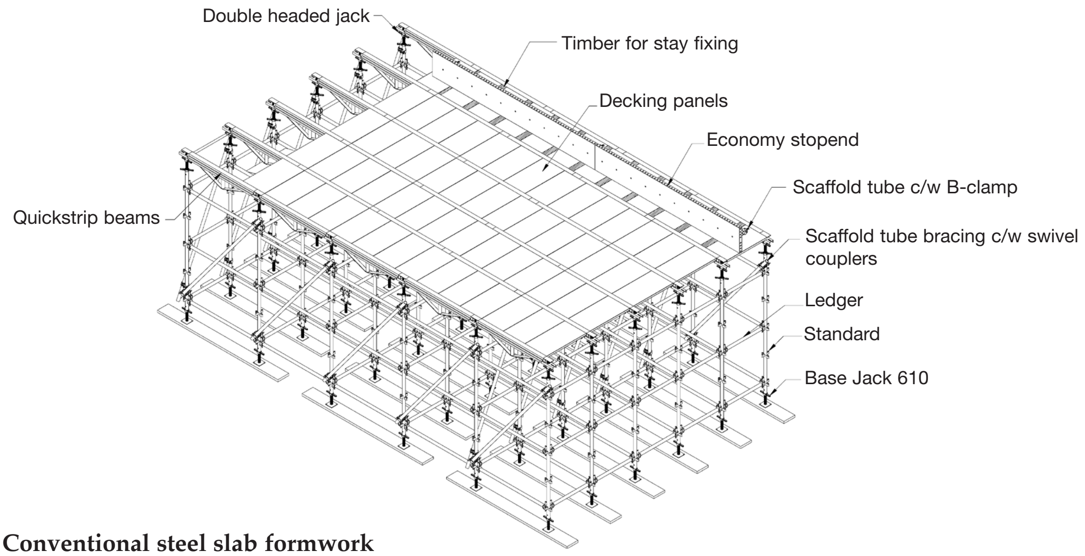


Figure 13.5: Conventional steel slab formwork

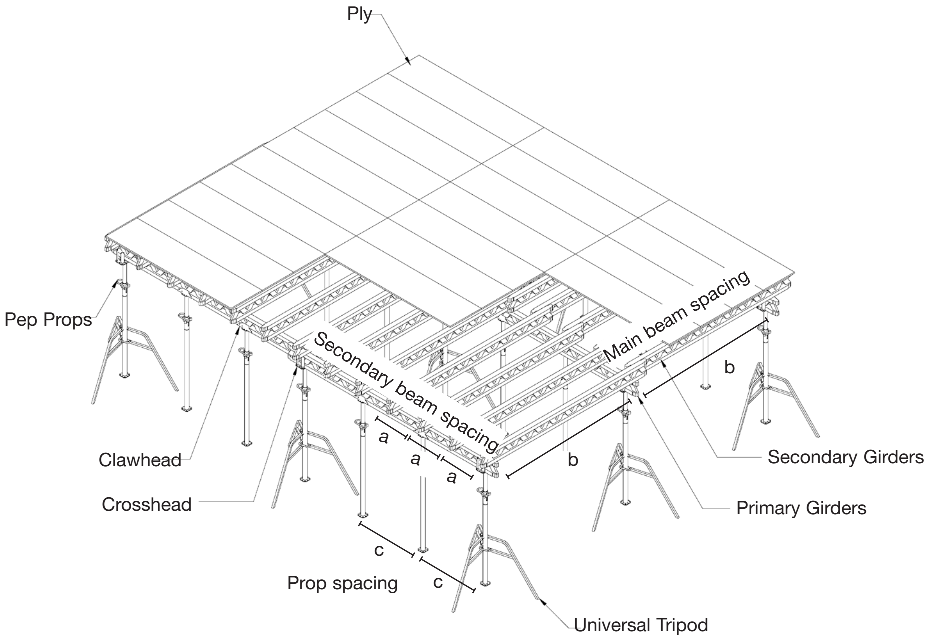


Figure 13.6: Variable girder slab formwork with (a) Secondary beam spacing, (b) Main beam spacing and (c) prop spacing

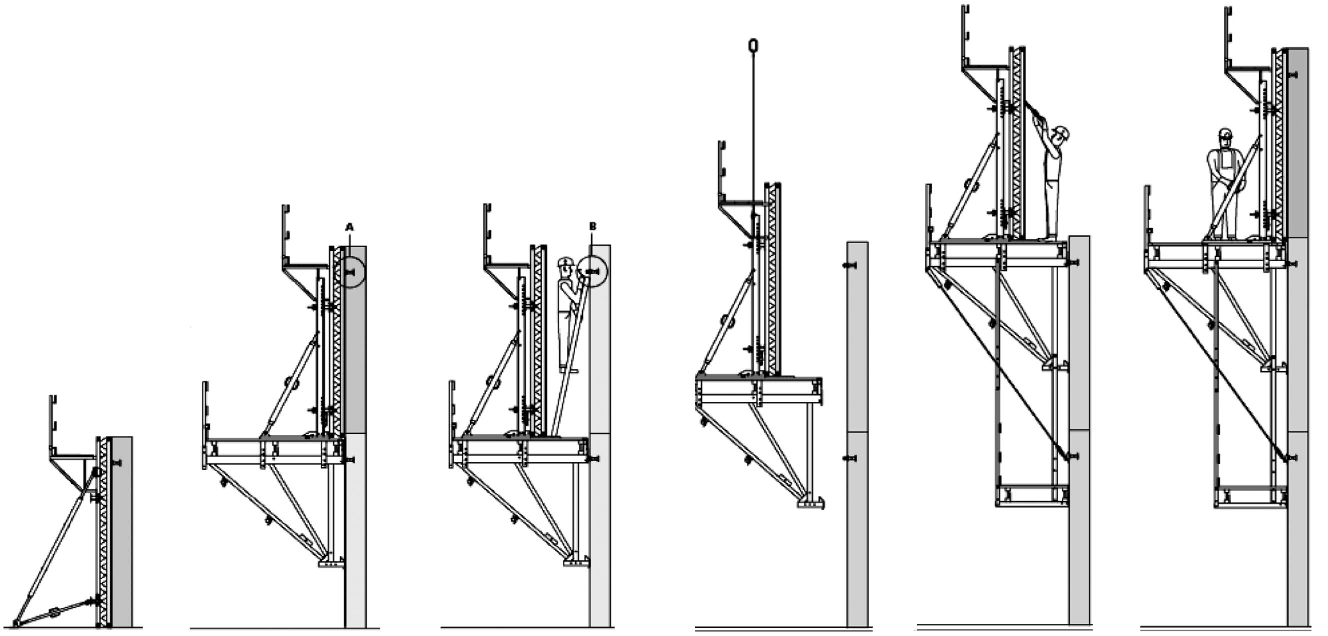


Figure 13.7: Climbing formwork sequence

travel up the steel rods with synchronised adjustments to the hydraulic jacks. The speed of construction is around 230 mm per hour.

This method of construction demands a cohesive concrete mix with a high fines content. The shutter is set to a taper with the bottom of the shutter tapering away from the wall. The required wall thickness is set to the centre of the shutter. The concrete coming out at the bottom of the shutter will be between four or five hours old.

13.3.4 Climbing formwork

Climbing formwork is used for structures of similar proportions to those of sliding formwork. High walls with multiple lifts and shafts of any shape and size can be built with climbing formwork. A monolithic structure of continuously cast concrete is not formed as it is with slip-forming. The workings of climbing formwork are as shown in Figure 13.7.

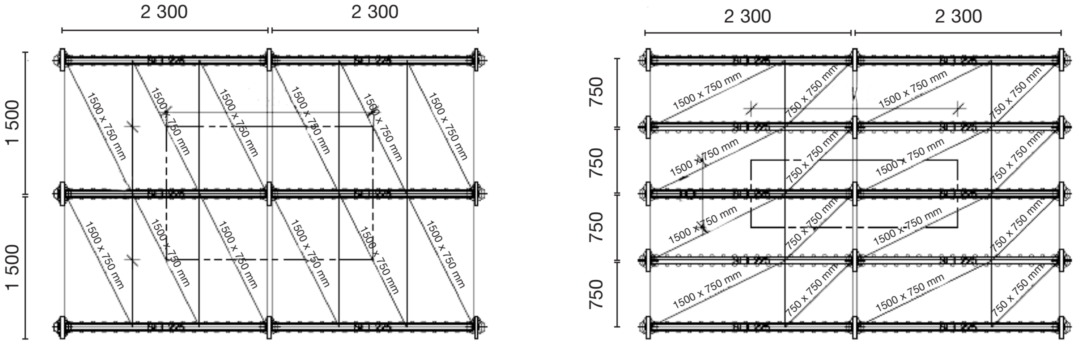
The first pour requires the normal arrangement of shutters with the pour height predetermined to be anything from 3 to 4 m high. The only addition to the first pour is the fixing of anchor screws onto the formwork face at a predetermined position.

The second pour requires the assembly of working platforms which will consist of two climbing brackets per platform. The two climbing brackets are connected by timber beams. The brackets of the climbing platform hook onto the anchor screws by means of bolts and scaffold mounting rings. The shutter is now lifted by crane and connected to the climbing platform. A second set of anchor screws is mounted on the shutter face. The formwork is closed and the second pour can commence.

The third pour requires the finishing platform to be connected to the underside of the climbing brackets before the formwork and platform are lifted to the third lift. The whole combination of the shutter, climbing platform and finishing platform is now lifted by crane to the third pour.

13.3.5 Slab formwork principles

The design layout of a decking system is preceded by the selection of the appropriate system. This is normally facilitated by involving the formwork supplier. Factors to



Slabs up to 400 mm thick

Slabs from 400 to 950 mm thick

Figure 13.8: Changes made to grid for slabs of different thicknesses (all dimensions in mm)

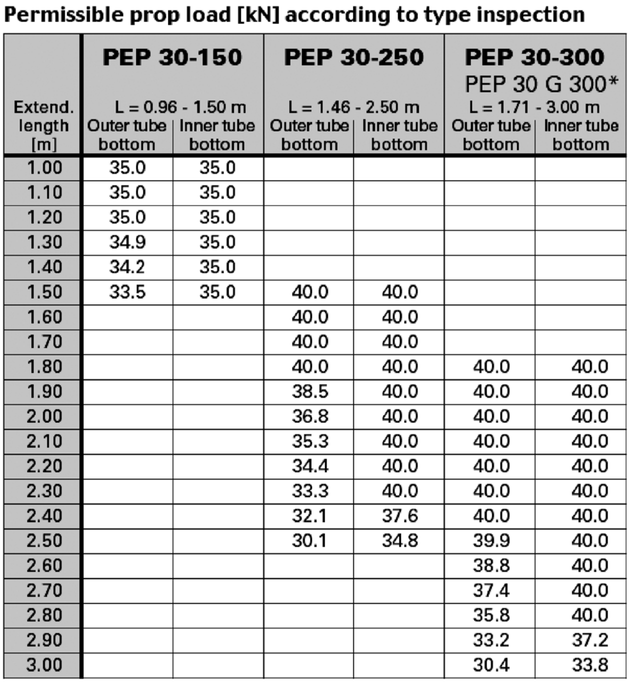
consider will be slab thickness, soffit height, early striking and site conditions.

Slab thickness

A higher density of equipment/m^2 will be required to cast a slab 500 mm thick than will be necessary for a slab 250 mm thick. For example, a modular panel decking system may have a fixed support grid, with props at 2,30 x 1,50m spacing (supporting 3,45 m^2 of concrete) allowing concrete of up to 400 mm thick to be cast. The grid of panels is turned to 2,30 x 0,75 m to support 1,73 m^2 per prop in order to cast slabs from 400 to 950 mm thick. This is illustrated in Figure 13.8.

Soffit height

The soffit height will determine the extended length of the prop or "prop extension". This is an important design consideration as the allowable load on a prop decreases up to maximum extension due to the slenderness ratio. For example, a prop with a maximum extension of 3 m will support 30 kN at maximum extension, but can support up to 40 kN when retracted to 2,7 m. Table 13.3 illustrates the fact that props become weaker with further extension and that the allowable load is different for having the outer tube at the bottom as opposed to having the inner tube at the bottom. The difference in strength relates to the lateral force, which can be as much as 20% of the dead load at the point where the decking system transfers force to the prop.



Permissible prop load [kN] according to type inspection

	PEP 30-150		**PEP 30-250**		**PEP 30-300** PEP 30 G 300*	
Extend. length [m]	L = 0.96 - 1.50 m Outer tube bottom	Inner tube bottom	L = 1.46 - 2.50 m Outer tube bottom	Inner tube bottom	L = 1.71 - 3.00 m Outer tube bottom	Inner tube bottom
1.00	35.0	35.0				
1.10	35.0	35.0				
1.20	35.0	35.0				
1.30	34.9	35.0				
1.40	34.2	35.0				
1.50	33.5	35.0	40.0	40.0		
1.60			40.0	40.0		
1.70			40.0	40.0		
1.80			40.0	40.0	40.0	40.0
1.90			38.5	40.0	40.0	40.0
2.00			36.8	40.0	40.0	40.0
2.10			35.3	40.0	40.0	40.0
2.20			34.4	40.0	40.0	40.0
2.30			33.3	40.0	40.0	40.0
2.40			32.1	37.6	40.0	40.0
2.50			30.1	34.8	39.9	40.0
2.60					38.8	40.0
2.70					37.4	40.0
2.80					35.8	40.0
2.90					33.2	37.2
3.00					30.4	33.8

Table 13.3: Example of an allowable load table in kN for various props with the outer tube and inner tube at the bottom. Formwork suppliers can provide these tables for their products.

13.4 Form-face materials

The original formwork material was timber, which is still used extensively today, both as support and as form-face material. Metals, rubber, plastics, etc are also commonly used as form-face materials.

13.4.1 Wood-based materials

Timber may be used as boards, planks, or sheets of wood-based material, to produce a "sawn-board" or "board-marked" finish.

The use of untreated pine timber tends to severely retard the surface of concrete cast against it when the timber is used for the first time.

Sawn-board formwork

A wide range of concrete finishes may be obtained from sawn-board formwork, from smooth to deeply-grained markings. Saw marks may be used as a decorative feature; the timber may be sandblasted to accentuate the grain, or different thicknesses of timber may be used.

Untreated timber can normally be re-used between one and ten times, depending on care taken during erection, striking and storage.

Timber should not be too dry or it will swell when absorbing moisture and make removal difficult, or the timber may distort in hot weather, making alignment difficult.

Sheet materials

Sheet materials include plywood, particle board or fibre building boards, but plywood is most commonly used.

Plywood may be backed by timber struts, particularly for soffit formwork, but it is often made into panels backed with a steel frame, as shown in Figure 13.9.

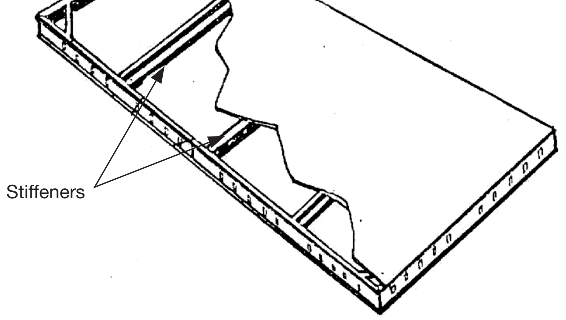


Figure 13.9: Ply-faced panel with steel frame backing

13.4.2 Metal form-face materials

Steel

Steel forms are made up in a variety of sizes with different amounts of stiffening. Figure 13.10a) shows a standard steel panel with internal stiffeners, while 13.10b) shows a heavy-duty panel with walers built in.

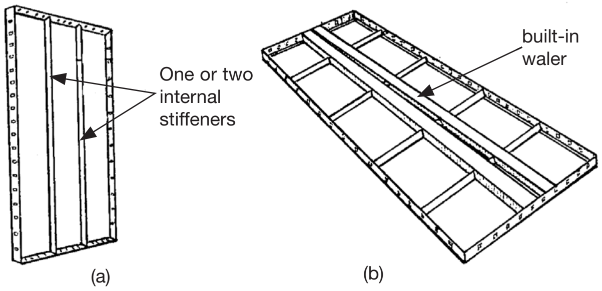


Figure 13.10: Standard steel panels

The surface of concrete produced by steel shutters is smooth, often variable in colour and of eggshell surface texture. Steel is dimensionally stable but liable to rust. The rust tends to come off against the concrete even if it is not severe enough to corrode the form face. Any other marks detectable on the form face, such as dents or tack welds, will also be reproduced.

The material is durable and has a high number of re-uses, provided it is handled carefully.

Aluminium

Aluminium is rarely used as a form-face material as there are problems due to the reaction of alkalis in the fresh concrete with aluminium in the presence of moisture. This can be minimised by suitable pre-treatment and the use of selected release agents.

The metal is more often used in proprietary systems for structural framing in the form of walers and soldiers, to reduce the mass of large sections.

13.4.3 Plastics and rubber

Plastics

Plastic materials are by-products of oil refining and their cost is usually high. A further disadvantage is that the elastic modulus or stiffness is low. Thus either a considerable thickness or other adequate backing is required to minimise deflections.

Glass-reinforced plastic (GRP) is a strong, low-density material and may be used for circular column forms, column heads and waffles. It is also useful for making highly complex shapes for architectural purposes. However, as the form face is impervious, a high proportion of blowholes may be experienced, and there may be a tendency for the concrete surface to craze.

With careful use, a GRP form can be used more than 100 times.

Expanded plastics may be used as void-formers and, properly treated, may be used to form sculptured surfaces.

Rubber

Rubber or elastomeric materials enable complex shapes to be cast in concrete. With elastomeric mould materials it is not necessary to provide a taper for the removal of the formwork. The materials are available as pre-formed mats that are fixed to formwork with a suitable adhesive or with staples. Where a deep undercut is involved, alternative means of fixing may be required so that the liner may be removed or peeled from the concrete separately, after the formwork has been struck.

13.4.4 Permanent formwork

As the name implies, the form material remains in place when the supporting formwork has been removed. Typical uses of permanent formwork include troughs and coffers, tunnel linings, bridge parapets and bridge decks, with panels spanning the gap between the beams or girders.

Materials used as a permanent formwork include precast concrete, glass-reinforced cement (GRC), profiled metal, brickwork and blockwork. Concrete and GRC products can be made in a variety of profiles and surface textures. Furthermore, GRC can be made in two- or three-dimensional profiles. An example of a three-dimensional profile is a coffer.

13.4.5 Controlled permeability formwork (CPF)

CPF is designed to be permeable to air and water, but not to cement particles. The concept of using formwork to remove excess water from cast concrete originated in the work of John Earley in the 1930s. Earley manufactured precast architectural facings using dry plaster moulds that absorbed water from the concrete and produced a better surface finish than he had obtained with coated formwork.

During compaction, air and some mix water escape through the form, leaving concrete in the cover zone with a lower water:cement (w:c) ratio. Figure 13.11 shows a textile form of CPF system.

The textile combines a filter and drain laid over conventional formwork, pierced by a grid of holes. The cast concrete surface is imprinted with the texture of the textile which helps mask minor surface variations, as shown in Figure 13.12.

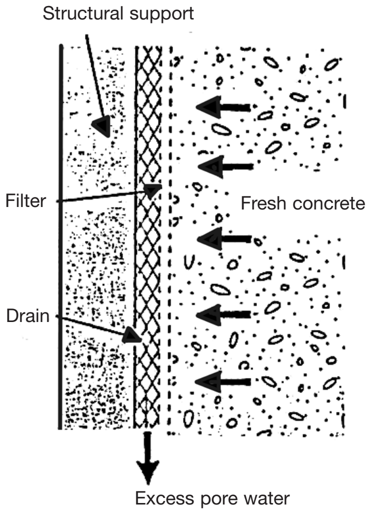


Figure 13.11: A textile CPF system

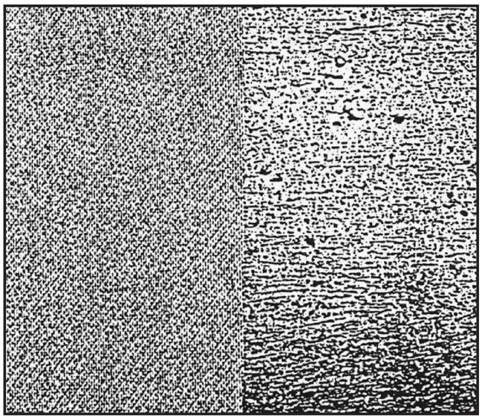


Figure 13.12: Concrete cast against a textile form (left) helps mask surface blemishes

Other systems of CPF include a silk form which uses a polyester filter and polyethylene drainage cloth tensioned and stapled to plywood panels. The panels do not contain drainage holes.

Superabsorbent polymer sheeting (SAP) is a material that absorbs up to 200 times its weight of water. The sheeting is stapled to the form edge.

The principal benefits derived from CPF include:

- Lower w:c ratio in the cover zone (see Figure 13.13) resulting in slower rates of carbonation, reduced rates of chloride penetration, and reduced permeability
- Improved surface finish
- Textured surface which improves bond for subsequent tiling or plastering
- Higher initial surface strength allowing earlier formwork removal

13.5 Formwork ties

Formwork to vertical members such as walls usually requires ties to resist pressure exerted by the concrete. Ties

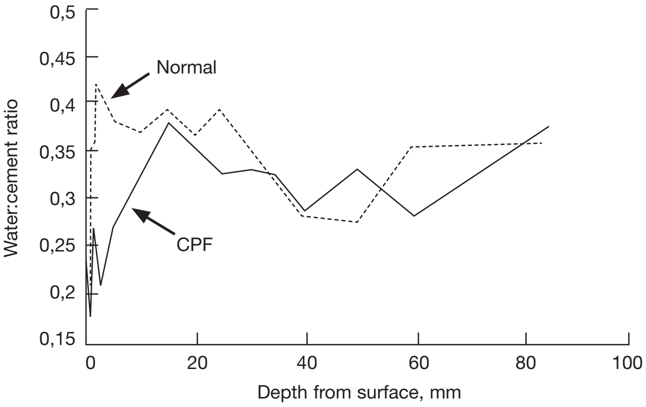


Figure 13.13: Lower water:cement ratio at concrete surface

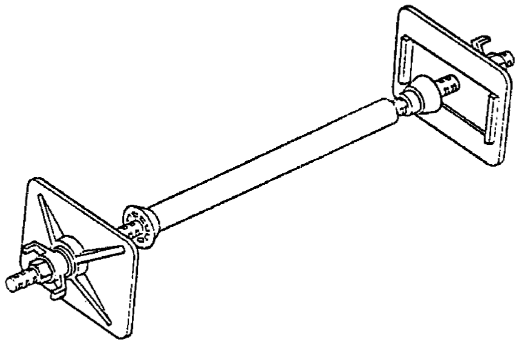


Figure 13.14: Dividag tie with sleeve and cones to act as a spacer

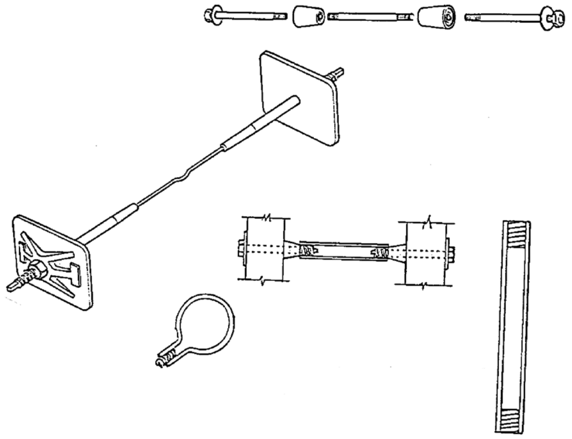


Figure 13.15: She-bolt, rod-and-cone and coil ties

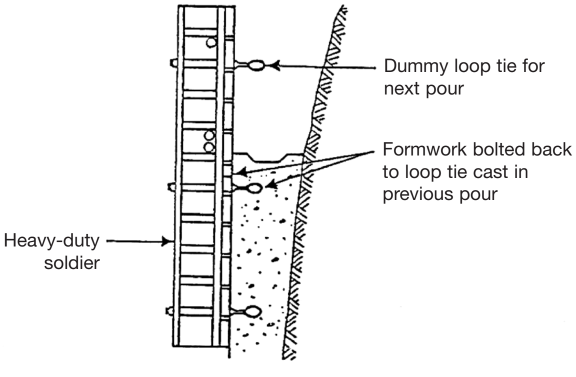


Figure 13.16: Use of coil loop ties with climbing formwork

connect the outer framing members of one panel to those of the opposite side of the wall so as to balance the forces. Ties must hold each pair of forms in the correct position, neither allowing the formwork to be squeezed, nor to bulge during concreting.

Two types of ties are used: those totally withdrawn after use, and those in which the centre portion remains permanently in the concrete.

Ties may be extracted by using a sleeve to prevent concrete touching them, or by using tapered ties.

So-called "Dividag" ties (see Figure 13.14) use a sleeve of plastic tubing, often with special plastic end pieces. These ends allow the tie to be drawn up tightly, ensuring that the wall width is correct.

Leave-in ties include she-bolts, coil ties and rod-and-cone ties, in which the centre portion of the tie is left in place (see Figure 13.15). In many cases, particularly with she-bolts and coil ties, spacers are needed to maintain form-faces the correct distance apart.

For single-sided climbing formwork, coil loop ties (see Figure 13.16) may be used to secure formwork to previously cast concrete.

13.6 Formwork release agents

Most form materials require the application of a release agent to the surface that is in contact with the concrete. Use of the correct type of release agent is important to minimise blemishes such as staining, blowholes, poor surface durability and colour variation. Under-application may result in the concrete surface plucking, ie patches of the surface adhering to the formwork and being pulled off the face during removal. Over-application usually results in staining and softening of the hardened concrete surface.

Table 13.3 gives suggestions for the selection of release agents to use with various types of form-face materials. Surfaces to which the release agent is applied should be clean. The agent may be applied by rag, brush or spray, but the application should be thin and even. Manufacturers' recommendations should be followed.

Release agents should not be allowed to contaminate construction joints or reinforcing steel. Treated surfaces should also not be allowed to become contaminated with dust or have the release agent washed off by rainwater.

13.6.1 Types of release agents

Release agents (also incorrectly known as form oils or mould oils) are generally of the following types:

Table 13.4: Suggested use of release agents[13.2]

Form-face type	Surface finish required			
	Plain		Special	
	Pre-treatment	Subsequent applications	Pre-treatment	Subsequent applicationsvccccccccc
Absorbent surfaces				
Timber: rough sawn	Neat oil with surfactant	Repeat applications	Impermeable coating (paint, lacquer, wax, etc)	Neat oil with surfactant Chemical release agent
Timber: planed	Mould cream emulsion	Repeat applications		
Plywood: unsealed	Chemical release agent. Impermeable coating (paint, lacquer, wax, etc)	Repeat applications		
Concrete	Wax	Neat oils with surfactant. Chemical release agent Impermeable coating		
Non-absorbent surfaces				
Plywood sealed	None required	Neat oils with surfactant Chemical release agent	None required	Neat oil with surfactant
Plastics: GRP, polypropylene plain or textured	None required	Neat oil with surfactant. Chemical release agent	None required	Chemical release agent
Steel	None required	Neat oil with surfactant. Chemical release agent	Impermeable coating (chemical release agent can be used to prevent rust)	Neat oil with surfactant. Chemical release agent

Notes:

1. Paints, lacquers, waxes and other impermeable coatings are pre-treatments only.
2. Neat oils and water soluble emulsions are not recommended.
3. Pre-treatment where specified is a once-only operation required to overcome the differential absorption characteristics of timber due to the varying density of the material. If not pre-treated, the formwork may give a grain-patterned finish. This is most marked on the spring and summer growth rings found in soft plywood.
4. Patches of barrier paint or varnish may wear off with use and cannot be renewed because of the absorption of release agent into the form-face material, resulting in a blotchy appearance to the concrete.

- Neat oils: neat oils are usually mineral oils which tend to produce blowholes and are thus not generally recommended.
- Neat oils with surfactant: neat oils with the addition of a small amount of surface activating or wetting agent minimise blowholes and have good penetration and resistance to weather.
- Mould cream emulsions: emulsions of water in oil tend to be removed by rain but they minimise blowholes and are good general-purpose release agents.
- Water-soluble emulsions: emulsions of oil in water used as release agents produce a dark porous non-durable concrete surface. They are not recommended for high quality work and are now seldom used.
- Chemical release agents: small amounts of various chemicals suspended in a low-viscosity oil distillate. These chemicals react with cement to produce a form of soap at the interface. They are recommended for all high-quality work, and should be applied lightly by spray to avoid retardation. Their relatively high cost is compensated for by better coverage.
- Paints, lacquers and other surface coatings: these are not strictly release agents, but are sealers that prevent release agent being absorbed into the form-face material. Wax treatments also fall into this category.
- Wax emulsions: a recent development uses a stable wax suspension that acts as a release agent. Advantages claimed are that it dries off completely and is resistant to removal by the weather.
- Vegetable extract release agents: non-toxic biodegradable vegetable extract release agents, which provide new and safe release agents suitable for use in confined spaces and with all types of formwork. Disposal after use is also easier as most are biodegradable and nontoxic.

Careful consideration needs to be given to the choice of release agent. The following information is required to select a suitable release agent:

- Delayed demoulding, immediate demoulding, or special demoulding
- Workability of the concrete: stiff, plastic or self-compacting for pumping
- Cement type: grey cement or white cement.
- Curing used: normal or steam
- Mould type used: wood, metal, plastic
- Subsequent work to be done on the exposed surface eg painting, tiling etc.

The supplier's technical information and safety information leaflets must be available to ensure correct application and safety precautions are adhered to on site.

The supplier's recommendations should always be followed especially in applications where aesthetics are important.

13.7 Care of formwork

Formwork frequently accounts for over a third of the cost of the finished concrete structure, so it should be handled with care. The life of forms can be extended considerably by careful treatment, thus decreasing the overall cost of the job. Rough treatment may make timber and plywood forms useless after one pour, whereas eight or more uses may be obtained by following good site practice.

All formwork should be cleaned as soon as it has been struck. Timber and plywood forms are best cleaned with a stiff brush to remove dust and grout; stubborn grout can be removed using a wooden scraper. A wax-based release agent may be applied to the edges of boards. This will serve as a bond breaker and also reduce water absorption. With GRP and other plastics, a brush and wet cloth are all that should be needed. Steel scrapers are not advisable for cleaning plywood or faced plywood.

Steel forms should be lightly oiled to prevent rusting if they are not to be re-used immediately; similarly, timber and untreated plywood should be given a coat of release agent for protection.

Any formwork surface defects such as depressions, splits, nail holes or other unwanted holes should be repaired and made good. A slow-setting epoxy can be used to fill damaged areas. The epoxy is then ground flush with the form face.

Formwork should be properly stored and protected. Panels and plywood sheets are best stored horizontally on a flat base, clear of the ground, so that they lie flat without warping. They should be stacked face-to-face to protect the surfaces. Large panels are usually best stored vertically in specially made racks. Stored formwork should be protected from the weather by tarpaulins or plastic sheeting. Once release agent has been applied, stack panels face-down to avoid exposure to dust, etc, before erection.

13.8 Safety measures

Safety measures associated with formwork, support work and falsework are of great importance as failures of these elements are responsible for more fatalities in the construction industry than all other types of accident together.

Accidents are usually due to lack of guard-rails, nets or other safety measures, but can also be due to major collapses of support work.

Possible causes of unsafe formwork, falsework or scaffolding include:

- Faulty design
- Loss of support (eg settlement of fill material, insufficient supporting capacity of the ground, adjacent excavations,vibration from plant or traffic, washout by rain or burstwater main)
- Use of sub-standard or damaged materials
- Inadequate, incorrect type of fastenings or improperly secured fastenings (eg nails instead of bolts to take tension)
- Eccentric or shock loads
- Failure of support (eg inadequate diagonal bracing, props out of plumb)

Formwork, falsework and scaffolding should be inspected before and during concrete placement to ensure safety.

13.9 Striking formwork

The time which must elapse between placing concrete and removing formwork (ie the striking time) will depend on factors which include:

- Type of member: column, beam side, slab, etc
- Type of cement and admixture used
- Average temperature conditions
- Curing conditions

Recent recommendations have been made that, for suspended slabs, the consultant should specify the strength at which the props may be removed and that it is the responsibility of the contractor to prove that this strength has been achieved.

Methods of determining striking times are discussed in the following sections.

13.9.1 Reference to standard tables

SANS 10100-2:1992.[13.3] Clause 9.5 of the code deals with removal of formwork. Table 13.5 is taken from Table 7 of the code.

CIRIA report No 67 *Tables of minimum striking times for soffit and vertical formwork* . [13.4] In these tables, striking times depend on:

- Grade of concrete (20, 25, 30, 40 and 50 MPa)
- Cement types (CEM I 32,5 N or 32,5 R and CEM I 42,5N or 42,5R on the basis of equal characteristic strengths)
- Mean air temperature (taken as the average of daily max/min temperatures)
- Concrete placing temperature (taken as the mean air temperature or 5°C, whichever is the greater, if the concrete temperature has not been measured)
- Formwork type
- Minimum dimension of freshly placed concrete

Table 13.5: Removal of formwork: minimum times in days

Type of structure member of formwork	Minimum time, days								
	Type of cement used								
	Ordinary portland cement and portland cement 15			Rapid-hardening portland cement2) and rapid-hardening portland cement 153)			Cements containing more than 15% of GGBS or FA		
	Weather								
	Normal	Cool	Cold	Normal	Cool	Cold	Normal	Cool	Cold
Beam sides, walls and unloaded columns	0,75	4)	1,5	0,5	4)	1	2	4)	4
Slabs with props left in place	4	4)	7	2	4)	4	6	4)	10
Beam soffits with props left in place, and ribs of a ribbed floor construction	7	4)	12	3	4)	5	10	4)	17
Slab props including cantilevers	10	4)	17	5	4)	9	10	4)	17
Beam props including cantilevers	14	4)	21	7	4)	12	14	4)	21

Notes:

1.) The time of removal of formwork allowing for the surface finish is to be decided by the designer. This should be regarded as a guide only.

2.) Shorter periods may be used for sections having a thickness of 300 mm or more.

3.) Only if rapid-hardening cement is used as a direct replacement for ordinary portland cement. Shorter removal times for formwork do not apply if mixes are designed with rapid-hardening cement and achieve the same 28-d strength.

4.) In cool weather, stripping times must be determined by interpolation between the periods specified for normal and cold weather.

13.9.2 Methods to determine concrete strength before striking formwork

Similarly-cured cubes (cubes cured alongside)

In this case, standard cubes are made from samples of the concrete used for the structure, but the cubes are cured as far as possible in conditions similar to those existing in the structure. This is usually done by keeping the cubes alongside or on top of the structure. If insulation is used to cover the element, the cubes should be placed under this insulation. The cubes should be prevented from drying out.

Cubes cured alongside the element will generally have a lower maturity than the in-situ concrete, due to such factors as size of member, insulating effect of the formwork etc. The method is suitable for sections (eg walls and slabs) up to 300 mm thick. In thicker sections the difference in maturity between the concrete and the cubes becomes significant and the method will indicate unduly conservative striking times.

Temperature-matched curing

The limitations of the previous method can largely be overcome using a temperature-matched curing system as depicted in Figure 13.17'.

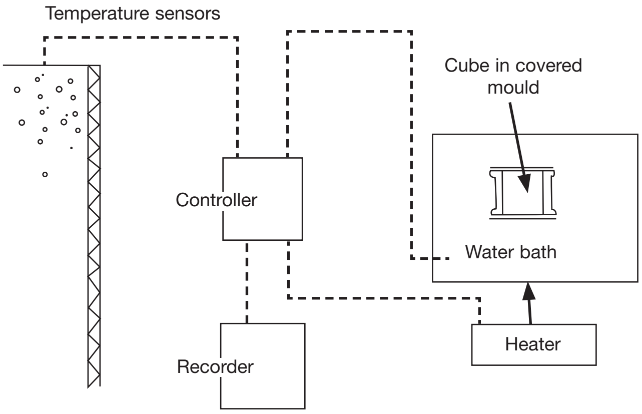


Figure 13.17: Temperature-matched curing

Cubes are made from samples of the concrete placed in the element. A steel or glass plate is placed on the exposed cube surface, and the covered moulds are submerged in the curing bath.

A temperature sensor is placed at a pre-determined position within the element. The temperature of the water in the bath is then matched to the in-situ temperature, thus the maturity of the two concretes remains the same.

The system can be used with accelerated curing techniques and in precast production. It is likely to indicate the shortest safe striking times.

A comparison of temperature and strength development between similarly cured cubes and temperature-matched cubes is shown in Figure 13.18.

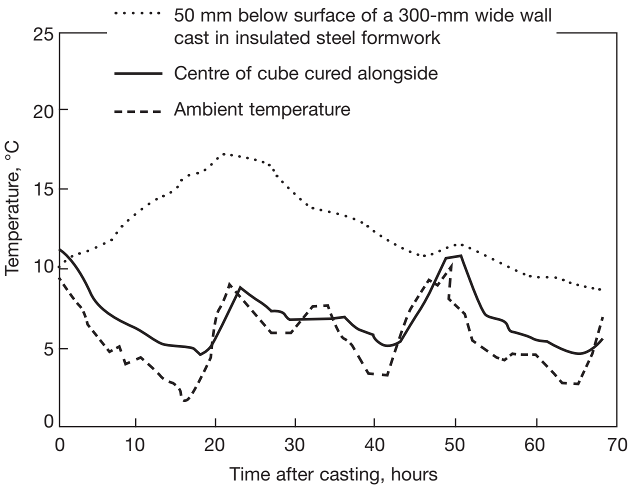


Figure 13.18: Comparison of cubes cured alongside and temperature-matched cubes

Non-destructive tests

Non-destructive tests may be carried out on the hardened concrete at early ages to estimate when the concrete is strong enough for formwork removal. Such tests include rebound hammer tests, penetration tests, pull-out/pull-off tests and break-off tests.

One of the main limitations of using any of these methods is in establishing a reliable correlation between the test value and in-situ strength. The use of different mix proportions or materials requires a separate correlation to be established.

Maturity measurements

Maturity is the summation of the product of temperature and the time during which it prevails. It is usually expressed in °C hours, or °C days.

The concept of maturity is that concrete from the same mix at the same maturity will have approximately the same strength whatever combination of temperature and time go to make up that maturity.

Maturity is usually calculated from a datum temperature, this value being the lowest temperature at which any appreciable increase in strength takes place. Datum temperatures derived by researchers commonly range from minus 10°C to minus 16°C. Limited data for local materials indicate a datum temperature of about 0°C.

Calculation of maturity

An example of a maturity calculation is given below. There are number of other formulae that can be applied.

Maturity = $\Sigma\ (T + 10) \times t$

where t is the time interval in days or hours in which the temperature was T, as measured from a datum temperature of minus 10°C.

For example:
If a concrete mix achieves a strength of 15 MPa in 3 days when cured at 20°C, how long will it take to achieve the same strength when cured at 35°C?

The maturity of the concrete cured at 20°C =
(20 + 10) x 3 = 90°C days

Since the strength of the two concretes is to be the same, the maturity of both concretes must be the same. Hence, for the concrete cured at 35°C:

$$90 = (35 + 10) \times t$$
$$\therefore t = 2 \text{ days}$$

Maturity meters
Maturity meters directly measure the maturity of the concrete. Some meters consist of a temperature sensing probe placed in the concrete and coupled to a processor.

The COMA meter is a small disposable meter that requires no sophisticated electronic equipment.

The meter is a liquid-filled capillary tube fixed to a maturity scale, encased in a glass container. The end of the tube is broken off, allowing the liquid to evaporate. The rate of evaporation is temperature dependent. The capillary tube and scale are then screwed back into the glass container and the whole COMA probe is pushed into the fresh concrete. The maturity is checked periodically until the desired maturity for striking formwork is attained.

13.10 Glossary

Back-propping – Props or staging arranged in a specified grid with the function of supporting additional loading on suspended concrete slabs over and above the weight of the slab itself. This will normally be the weight of the slab and equipment directly above.

Cantilevered formwork – A deck constructed over a slab edge (normally 750 mm for working space) without support from ground level, but instead the supporting main beams are chained down or bolted to the concrete floor from which the formwork is supported.

Column head – A thickening of the concrete slab located at the top of a column with the function of containing reinforcing and counteracting shear force in the area around the top of the column.

Consumable items – A formwork component which is cast into the concrete and can therefore not be used again, such as plastic spacers, ferrule tubes and ferrule cones.

Cycle joint or construction joint – This is an interruption of concrete with the purpose of allowing reinforcing to continue through two separate concrete pours. (The location of such joints must be carefully considered in suspended slabs, because of higher shear force around columns.)

Cycle time – This is the full rotation in days for the use of a set of formwork equipment, from the point of erecting a set to the point of erecting the same set of equipment, for the next pour.

Downstand beam – A reinforced concrete beam hanging under the slab soffit with the function of providing additional structural support to the slab by working in tension.

Fixed cost – Items of formwork to be purchased which are in many cases non-standard components manufactured specially for a particular project which is also referred to as "a special manufacture".

Gang-form – An arrangement of wall form panels pre-assembled on a flat level surface which is lifted by crane into its working position.

Girders – A wooden beam with specified shear capacity and bending moment resistance.

Hydrostatic pressure – This is normally expressed as the kN-/m^2 on the face of the form lining which results from the formwork containing wet concrete in a state of rest.

Isolated beam or roof beam – A beam supported by columns with the function of supporting structural steel or roof trusses at the top of a building or structure.

Max list – A list of the maximum number of formwork components required on a particular construction sequence, allowing all the variances of shape to be constructed from the first concrete cycle to the last.

Patching – The use of mostly plywood, cut to size in order to fill up the gaps between a modular decking system and the neighbouring structures like columns and walls. This also provides a form of lateral bracing during concreting operations.

Prop extension – The total length to which a prop under a decking system needs to be set in order to support the decking system from the floor up to the underside of the concrete soffit.

Push-pull prop – A telescopic steel prop with threads at both ends anchored to a base plate on the ground and the top of a wall or column shutter. The function is to plumb wall and column formwork and resist wind forces with the capacity to push and pull simultaneously.

Rate of pour – The cubic metres per hour of cemente poured into a mould. This becomes critical with columns and walls as it has a direct influence on the rate of rise.

Rate of rise – The metres per hour rise in a mould which relates to the hydrostatic pressure.

Re-propping – With specific reference to quickstrip systems; this relates the release of props from a newly cast suspended slab, normally at 60 to 70% of 28-day strength, to re-engage the same props into the working position under the slab, which allows the suspended slab to deflect and support its own weight.

Soil plates - Scaffold planks (230 mm x 38 mm x 6 m) bedded on a 20-to 30-mm thick layer of river sand which fills all voids caused by uneven ground conditions, with the purpose of transferring prop load to the ground.

Strong-back – A vertical steel beam, used in traditional steel formwork, with the task of transferring tie forces to the walers.

Slenderness ratio – This principle determines that a prop becomes progressively weaker with further extension.

Striking or stripping – This is the systematic removal of formwork or shutters from the hardened concrete to reveal the concrete face.

Suspended slab – A reinforced concrete slab supported by columns or walls.

Time-related cost – Cost to the contractor which relates to the hire of plant or equipment, and more specifically for standard components of formwork.

Up-stand beam – A concrete beam located on top of a concrete slab with the function of providing additional structural support in compression.

Waler – The horizontal members (normally a twin channel) with the function of transferring tie forces to the vertical girders or transferring load from strong-backs to the formwork.

Acknowledgements

Information in this chapter and Figures 13.1 to 13.7 are reproduced by courtesy of Wiehahn Formwork and Scaffolding (Pty) Ltd.

Material was drawn from the C&CI School of Concrete Technology's course notes.

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13.3 SANS 10100-2:1992, **Code of practice for the structural use of concrete. Part 2: Materials and execution of work,** Pretoria: South African Bureau of Standards, 1992, table 7.

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Chapter 14
Reinforcement

Gary Theodosiou

14.1 Introduction
Reinforcement in the form of steel bars, rods, strands or welded mesh fabrics, selectively positioned and of the required quantity, is embedded in concrete in such a manner that the reinforcement and concrete act together in resisting forces to provide tensile strength in the concrete-steel composite known as reinforced concrete.

Reinforcement can also be used to increase the compressive strength in elements such as columns. In addition to strengthening concrete in tension and compression, it is used to control cracking.

In order for stresses to be transferred from the concrete to the steel, adequate bond of the concrete to the steel is essential (see section 7.6).

During concreting operations, full compaction of concrete around the steel is essential. Excessive bleeding and settlement will contribute to a reduction of bond, particularly in the case of horizontal bars. The presence of loose rust, paint or oil on the bar will also reduce bond.

Conventional types of reinforcing are covered briefly as they are extensively used and their properties well known. Alternative reinforcing materials are discussed in some detail as the reader may be less familiar with their composition and properties.

The design of reinforced concrete elements is outside the scope of this book.

Fibre-reinforced concrete is covered in Chapter 21.

14.1.1 Terminology
Steel bars for reinforcing concrete are correctly termed “reinforcing bars”, sometimes shortened to “rebar” or “reinforcement”.

Formerly the word “deformed” together with the words “Type 1” or “Type 2” described a bond performance rather than an appearance, but with the phasing out of “Type 1” (square twisted) bars, “deformed” has now become synonymous with “ribbed.”

It is not universally understood that initially all bars are hot rolled after the basic steelmaking process. So plain round mild steel with a tensile strength of 250 MPa or high-yield bars with a minimum tensile strength of 450 MPa are both “hot rolled.”

Ribbed bars for cold working are also “hot rolled” initially but subsequently they are “cold worked” by twisting to become high-yield bars.

Fabric, not mesh, is the correct term for hard-drawn wire welded into sheets. Fabric comprises a mesh of wires welded at their intersections.

14.2 Types of reinforcement
The standard types of steel for reinforcement are plain, mild steel bars, deformed high-yield bars and steel fabric, as specified in SANS 920:2005 *Standard specification for steel bars for concrete reinforcement*,[14.1] and SANS 1024:2006 *Welded steel fabric for reinforcement of concrete*.[14.2] SANS 920 uses the symbol “R” (round bars) to designate mild steel bars, and “Y” to designate high-yield bars. R bars are smooth, while Y bars have ribs or deformations to improve bond.

14.2.1 Mild steel bars
Hot rolled plain surface mild steel bars have a minimum tensile strength of 250 MPa. They are easily cut, bent and welded, and are most commonly used for links or stirrups.

In general the use of mild steel is not economical but for small bars where small radii of bending are required, they can be useful.

14.2.2 High-yield bars
High-yield bars with a minimum tensile strength of 450 MPa may be formed by hot rolling, during which process ribs are formed on the surface of the steel, or by cold working where deformed mild steel bars are twisted after hot rolling.

Most high-yield bars in South Africa are formed by hot rolling, and have the resultant ribs on the surface of the steel.

Hot rolled bars must not be heated without authorisation because at high temperatures the properties of the steel can change. The tensile strength of cold worked bars can be reduced to that of mild steel bars.

Bars should not be cut using oxyacetylene torches, and any welding done at junctions and splices must be done only by an experienced welder under carefully controlled technical supervision.

Hot rolled bars may be considered weldable if the carbon equivalent percentage (CEV) is less than 0,51% and not weldable if the CEV is greater than 0,51%.

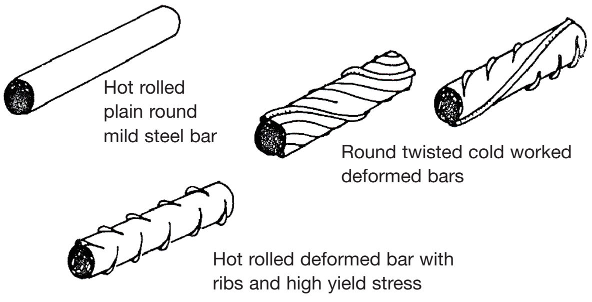

Figure 14.1: Types of reinforcing steel

14.2.3 Steel fabric

Steel fabric reinforcement, covered by SANS 1024:2006,[14.2] is made from hard-drawn plain steel wire, welded to the required pattern. It is mainly used as reinforcement in slabs and precast concrete elements.

Used in walls, it provides the benefits of reduced labour costs in handling and fixing, but these must be balanced against the higher material costs.

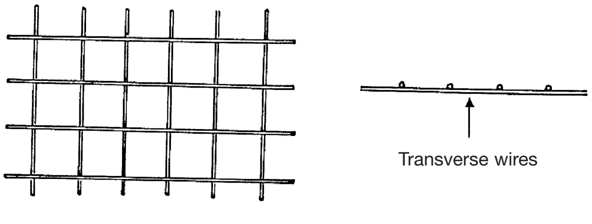

Figure 14.2: Plan and section of fabric reinforcement

14.3 Alternative reinforcement for concrete

When correctly specified and constructed, reinforced concrete is a cost-effective and durable construction material. However, the specification and/or use of inappropriate materials can lead to poor durability, particularly in severe exposure conditions and with poor levels of workmanship. The major problem is the corrosion of the embedded steel reinforcement. The principal approach for achieving durability is the selection of appropriate concrete materials and mixes.

Another approach, and one that is becoming more widespread, especially under conditions of extreme exposure, for example concrete exposed to de-icing salts or in a marine environment, is the use of alternative reinforcement materials that will be more durable.

These corrosion protection systems include coating the steel or using stainless steel reinforcement. The reinforcing steel can be coated with either epoxy or zinc. ASTM A775-07b *Standard specification for epoxy-coated steel reinforcing bars*[14.3] and ASTM A767-05 *Standard specification for zinc-coated (galvanized) steel bars for concrete reinforcement*[14.4] address the requirements for these coatings. Stainless steel reinforcing is covered by BS EN 10088-1:2005 *Stainless steels. Part 1: List of stainless steels.*[14.5]

14.3.1 Fusion-bonded epoxy-coated steel reinforcement

Fusion-bonded epoxy-coated reinforcement (FBECR) has been developed over the past 25 years to combat the huge cost of deterioration of reinforced concrete structures caused by reinforcement corrosion. The reputation of the first FBECR was doubtful, with many failures reported in the USA. The critical processes in the manufacture of FBECR are now better understood, and the coating must comply with ASTM A775-07b.

It has been recognised that with good manufacturing techniques, the quality and performance of FBECR bars can be improved significantly. It is also recommended that no cracking be allowed on any part of the coating when a bar is subjected to the specified bend test. If aggressive substances can reach the exposed steel because of damage to the coating, corrosion will be concentrated at these points and the cross-section will be reduced correspondingly.

There are two types of epoxy coatings available in South Africa; powder-coated epoxy and epoxy paint. In practice, the problem with FBECR is that the coating is almost always scratched during handling. Any rupture (called a holiday) allows ingress of water and oxygen, and in service, inevitably results in premature failure due to under-film creep.

An important structural requirement that may be affected by the presence of the epoxy coating is the bond strength between the steel and the concrete, normally determined by pull-out tests. Epoxy coatings can cause a certain degree of slippage between the coated bar and the concrete.

14.3.2 Galvanised steel reinforcement

Hot-dipped galvanised steel is produced in accordance with SANS 121:2000[14.6] and ISO 1461:2002[14.7] by dipping clean and fluxed steel into a bath of molten zinc.

Zinc is a highly reactive metal that coats the steel and also reacts with steel to form hard iron/zinc alloys. The zinc forms a sacrificial anode and is itself consumed to protect the steel but its reaction products are not as expansive as those of steel and they are soluble. Under-film corrosion of galvanised steel is not possible.

It must be emphasised that the specification of galvanised reinforcement should not be considered to be an alternative to good concrete practice, but rather its use should be regarded as complementary to good concrete practice.

The main advantages of galvanised steel are:

- It offers additional protection
- Where the risk of corrosion is high (ie inland where carbonation is a problem as well as in coastal regions where chloride ingress is the dominant problem)
- Where cover is less than ideal (eg in precast units)
- Where rust staining cannot be tolerated
- Where steel is partly embedded in concrete (eg holding down machine bolts)
- It has longer life in cracked carbonated concrete than uncoated bar
- It delays the initiation of corrosion and cracking
- It tolerates higher chloride migration levels than uncoated steel

Some misconceptions of hot-dip galvanising are that:

- Bars cannot be bent after galvanising
- If the coating is scratched, protection fails
- The heat of the molten zinc reduces the strength of high-tensile bars
- High-tensile bars will suffer hydrogen embrittlement (galvanising is not generally recommended for steels with a tensile strength above 800 N/mm^2)
- Hot-dip galvanised bars will accelerate corrosion
- Hot-dip galvanised bars cannot withstand the high pH of concrete

Hydrogen evolution

When unpassivated zinc comes into contact with fresh concrete, a chemical reaction may take place which may form hydrogen gas. Many writers have argued that this gas will cause the concrete surrounding the reinforcement to become spongy, with a resultant loss of bond.

Combating hydrogen evolution

It has been suggested that this can be overcome by adding chromate to the concrete mix or giving the bars a chromate passivation treatment. On the other hand, zincates produced – which are less expansive and more soluble than iron corrosion products in the concrete environment – could diffuse into the pores of the concrete and make the concrete more dense locally, increasing the bond strength beyond that of uncoated bar.

The use of a soluble chromate as an additive to the concrete is not recommended. Soluble chromates (such as sodium dichromate which is commonly used as a passivation medium in most hot dip galvanising facilities) may be carcinogenic. Therefore, exposure to these chemicals should be minimised. By adding them to concrete, health is potentially compromised since all the concrete is being affected by a toxin that is only required to function at the zinc/concrete interface. A better solution would be for specifiers to include a requirement that all galvanised reinforcing be passivated in a suitable chromate solution following the galvanising process.

With reference to corrosion resistance in chloride-contaminated concrete, the distinction must be made between cast-in chloride and that which penetrates from outside. Cast-in chlorides may attack the zinc coating before and during the formation of the passive calcium hydroxyzincate, whereas chlorides penetrating from the outside will find the passive layer already formed and may therefore be less harmful.

Though zinc can be depassivated and attacked in the presence of chloride ions, the tolerance of galvanised steel to chloride is greater than that of uncoated steel. Galvanising protects the steel against chloride ingress because it is more tolerant to chlorides, requiring a higher concentration for depassivation, and it corrodes more slowly in chloride-contaminated conditions.

14.3.3 Stainless steel reinforcement

Historically used in the preservation of old structures and in severe environments, stainless steel reinforcement is not produced in South Africa and the cost to import is currently so great that its use has become impracticable. In addition, 3CR12 which was produced locally is no longer available.

Stainless steels are iron-based alloys, containing at least 10,5% chromium, whose corrosion resistance increases with alloying metal contents. Some of the most expensive grades also contain nickel.

The alloys are commonly identified by their microstructure: martensitic, ferritic, austenitic and duplex (with a mico-structure of ferrite and austenite). Although these terms relate to the microstructure of the alloys, they also define the ranges of chemical composition as the microstructures of these alloys tend only to be stable within specific composition ranges. Interest in the use of stainless steels as concrete reinforcement is attributable to their increased resistance to corrosion particularly in chloride-containing media.

Stainless steel depends upon oxygen to oxidise the highly reactive chrome which forms an impermeable protective barrier against corrosion. There is always enough oxygen in concrete for this to occur.

Since the publication of BS 6744[14.8] in 1986, there have been many developments which have meant that updating was required to cover the latest chemical and physical properties of the stainless steels available. The BS 6744 classification system has thus been replaced by BS EN 10088-1: 2005[14.5]

Properties
Concrete bond to stainless steels is inferior.

Although 3CR12 conforms to the requirements of SANS 920,[14.1] the strength can be very high (600 MPa) and this could result in brittle failure of reinforced concrete elements.

It has been suggested that the absence of a yield point might improve reinforced concrete ductility if the strain hardening of the steel can offset concrete strain weakening.

Applications
Stainless steel reinforcement is ideal:

- In conditions where there may be high chloride concentrations and/or cover depth may be compromised.
- Where the risk of failure due to corrosion cannot be permitted.
- Where the service life of a structure needs to be considerable and maintenance is not an option.

Stainless steel reinforcement is unsuitable:

- Where there is no oxygen present.
- Where a less costly alternative would suffice.
- Where the reinforcement will incur high fatigue.

Stainless steel has been used successfully:

- In many parking garages and road bridges in Europe and North America (austenitic grades).
- 3CR12 was used successfully in the construction of four pedestrian rail bridges at Warner Beach on the South African south coast.
- 3CR12 rebar was used for part of the Saldanha ore jetty and at Coega Harbour in the Eastern Cape.

14.4 Cover to reinforcement

Cover is the distance from the surface of the concrete to the nearest reinforcing bar surface, including links.

When detailing reinforcement it must be remembered that the reinforcement cannot be fixed and bent with great accuracy, particularly when larger bar sizes are used. Even with pre-assembled reinforcement sections, difficulty is often experienced in practice in accurately aligning the bars.

The requirements for minimum cover for various conditions of exposure are shown below and are taken from SANS 10100-2:1992.[14.9]

Table 14.1: Minimum cover for various exposure conditions

	Minimum cover, mm				
	Conditions of exposure				
Concrete	Mild	Moderate	Severe	Very severe	Extreme
Normal-density concrete	20	30	40	50	60
Low-density concrete	20	40	50	60	70

Mild conditions – The concrete is:

- Exposed generally to unpolluted air.

  For example: Indoors (but not including industrial areas) or out of doors in arid rural areas (Karoo)

Moderate conditions – The concrete is:

- Sheltered from severe rain
- Buried in non-aggressive soil
- Subject to polluted air (but not corrosive fumes)

  For example: Indoors in industrial areas or out of doors in rural Highveld areas.

Severe conditions – The concrete is exposed to:

- Wet conditions in which the water is mildly to fairly aggressive
- Corrosive fumes
- Salt-laden air

  For example: Out of doors in industrial areas; out of doors in marine atmospheric conditions (up to 15 km from the sea); or out of doors in the Cape winter-rainfall area.

Very severe conditions – The concrete is exposed to:

- Wet conditions in which the water is mildly to fairly aggressive
- Abrasive actions under any wet conditions
- Highly corrosive fumes

Extreme conditions – The concrete is exposed to wet conditions in which the water is highly aggressive.

Conditions are considered wet if the concrete is exposed to water continuously or intermittently. The effect on the concrete of exposure depends on the aggressiveness of the water, the period of time during which the concrete is wet, and the frequency of the wet-dry cycling. It is not possible to propose definite limits in this regard. However, it should be kept in mind that highly aggressive water can have a seriously detrimental effect on concrete even if the period during which the concrete is exposed to such water is short.

Figure 14.1 shows graphically the importance of adequate cover to steel in combating reinforcement corrosion

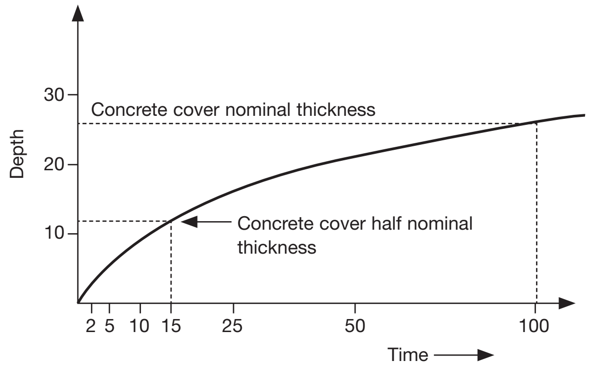

Figure 14.3: Reduction of initiation time of corrosion due to local reduction in concrete cover

Spacers

Adequate impermeable spacers of the correct size must be used and all sections must be firmly fixed to prevent displacement during concreting. Spacer blocks are generally less than 75 mm thick and should be made of impermeable material such as a high quality mortar, asbestos cement or plastic.

Mortar spacer blocks can be made from a 1:1 mix of cement and coarse sand. These blocks should be cured in water for at least seven days. The wire used for fixing the blocks in position must not extend more than halfway into the block to avoid the risk of corrosion of the wire.

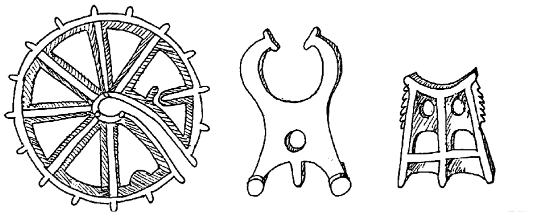

Figure 14.4: Examples of plastic spacer blocks

14.5 Control of reinforcement on site

Reinforcement should be inspected on delivery for signs of damage and excessive or loosely adhering rust or millscale. Bar diameters should be checked and bundles should be correctly labelled and compared with the bending schedule.

The reinforcement must be stored in such a manner as to prevent contamination by mud, sea spray, oils, paint, etc. This may necessitate it being stored off the ground or under cover.

Bars must be firmly fixed using the correct tie wire and tie method. The supervising engineer should check the position of and support given to the reinforcement prior to concreting. Cover to reinforcement should be checked, using a suitable covermeter, after concreting.

14.5.1 Fixing of reinforcement

For reinforcement to function effectively it must be:

- Correctly cut and bent
- Correctly positioned, using suitable supports and spacers
- Securely tied to prevent movement during construction and concreting

14.5.2 Splicing reinforcement

Splicing is the method of transferring load from one bar to the next. Splicing methods include lapping, welding (not generally recommended) and mechanical methods such as screwed connections.

14.5.3 Surface corrosion

According to SANS 10100-2:1994[14.10] "All reinforcement, at the time of placing of the concrete, shall be free from rust, scale, oil and other coating that may reduce the bond between the steel and the surrounding concrete, or initiate corrosion of the reinforcement." SANS 10100-2 is ambiguous as to whether firm rust is allowed on reinforcement prior to embedment in the concrete.

With prolonged exposure to the atmosphere all steel reinforcement corrodes. However, according to other sources including the previous structural code CP110,[14.11] "Normal handling prior to embedment in the concrete is usually sufficient for the removal of loose rust and scale from reinforcement."

For small sizes of bar and fabric, severe corrosion can reduce the effective cross-sectional area and thus its structural performance.

The presence of firm rust or scale improves the bond of plain bars, as the bond of this type of bar is largely dependent on friction.

14.6 Prestressed concrete

The patent for prestressed concrete was taken out in California in 1886. However, Eugene Freyssinet of France, regarded as the "father" of prestressed concrete, registered his first patent in 1928.

Prestressing techniques have allowed new types of structures to be built, such as very long-span bridges, offshore oil platforms, harbours and nuclear power stations.

14.6.1 Principles

In prestressed concrete, a compressive stress is put into the concrete member before loading to pre-compress it. Under working-load conditions pre-compression diminishes, but as no tensile stresses develop, the concrete remains crack-free. If the working load is exceeded and tensile stresses overcome the prestress, cracks will develop. However, removal of the load results in complete closing of the cracks, which do not reappear under working-load conditions.

Pre-compression is applied by tensioning steel tendons that are incorporated in the member. The tendons are usually in the form of high-strength wires, strands or bars, used singly or made up into cables. The two basic methods of prestressing using tendons are:

- Pre-tensioning
- Post-tensioning

Pre-tensioning

In this method steel is first tensioned between fixed abutments and the concrete is cast in moulds around the steel. When the concrete is strong enough, the steel is released from the abutment and compressive forces are transferred through bond to the concrete.

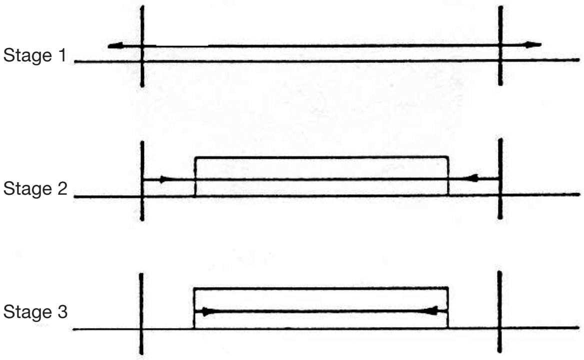

Figure 14.6: Pre-tensioning

This method is mainly suitable for factory production when a large number of similar units, such as railway sleepers or lintels, are required. The most effective method is the long-line system using permanent stressing beds.

Steel tendons are tensioned between anchor plates at opposite ends of the stressing bed. At one end, the anchor plate bears directly onto supporting steel joists and is referred to as the fixed abutment. At the other (jacking) end, temporary steel struts are placed between the anchor plate and supporting joists.

The tendons are drawn down the full length of the stressing bed and anchored off at the fixed abutment. When all tendons have been positioned, jacking commences from the other end. Slack is taken up, the load applied and the tendons are then anchored off.

After stressing, reinforcement is fixed in position and the moulds assembled. Care must be taken to ensure that tendons do not become coated with any material such as mould oil which would reduce bond.

In order to fully utilise the system, accelerated curing is often used to achieve rapid production turn-around.

When the concrete has gained sufficient strength, the steel is cut at the ends of the units, putting the concrete into compression.

Post-tensioning

In the post-tensioning method, tendons are stressed after the concrete has hardened sufficiently. Tendons are usually encased in a metal sheath, often of preformed circular ducting, to ensure free movement on stressing. The sheath is accurately positioned and fixed to the reinforcement, as shown in Figure 14.8.

As end-anchors transfer substantial forces over small areas, end-anchorage areas are heavily reinforced. Special care must be taken in these areas to ensure thorough compaction of the concrete.

With moulds assembled and reinforcement and sheathing fixed in position, concrete is placed. It is important to ensure that paste from the mix cannot enter the duct, and that

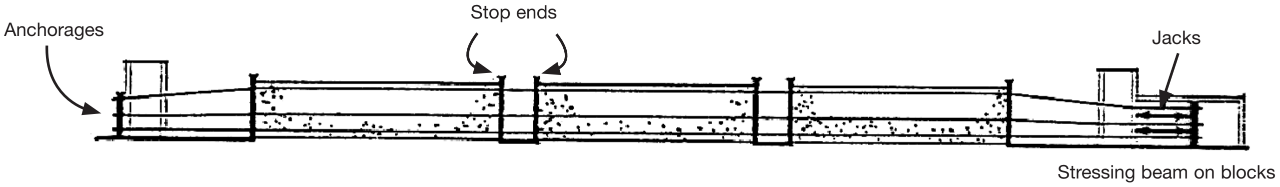

Figure 14.7: Long-line production

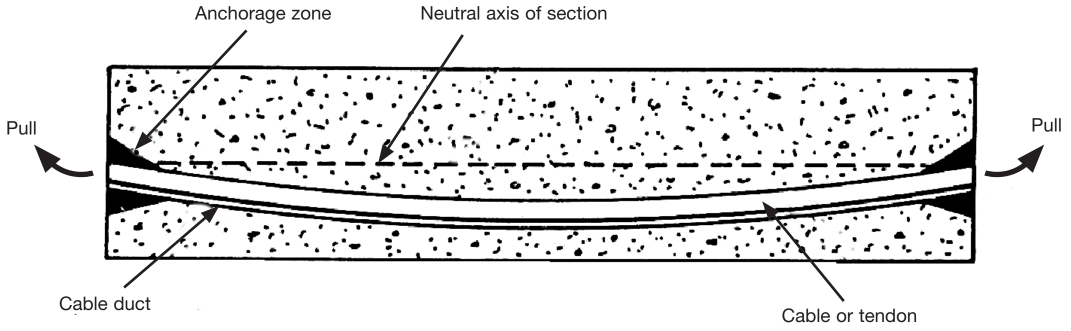

Figure 14.8: Post-tensioning

sheathing is not able to "float" out of position. The concrete must be thoroughly cured.

When the concrete has gained sufficient strength, the tendons are tensioned using hydraulic jacks. Tendons are tensioned by anchoring one end of the tendon and jacking against the face of the anchorage at the other end. Tendons are anchored off using barrel and wedge assemblies.

Tendons in each duct may be tensioned individually or simultaneously. During tensioning both applied load and extension are noted. If a tendon becomes "locked" in the duct, extension will decrease, indicating a fault.

When the correct load and extension have been reached, the tendon can be anchored off.

With all tendons stressed and anchored off, ducts are normally filled with a cement grout under pressure to prevent corrosion.

Tendons

The steel used in prestressing work is usually in the form of cold-drawn, high-tensile wires or alloy steel bars. The strength of the steel is four times that of high-yield reinforcing steel (although the elastic modulus is about the same).

Wires can be used singly, or twisted together to form a strand. Wires are typically 5 to 7 mm in diameter. Strands, consisting usually of seven wires are 12 to 15 mm diameter. Bars are 16 to 36 mm diameter.

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Chapter 15

Thermal properties of concrete and temperature development at early ages in large concrete elements

Yunus Ballim and Peter Graham

15.1 Introduction

The heat liberated by the cementing reactions in concrete is primarily of interest because of the potential for thermally induced cracks to develop in the concrete. This cracking potential is determined by the:

- Stresses induced by strains resulting from transient temperature and shrinkage gradients in the concrete
- Ability of the concrete to respond to these stresses without cracking
- Nature and degree of restraint

The complexity of this process lies in the fact that the ability of the concrete to deform without cracking depends on its tensile strength, relaxation capacity and stiffness; parameters which are both time and temperature dependent. As the cementing reactions progress, concrete increases in stiffness and therefore attracts more stress, while at the same time the tensile strength increases and relaxation capacity decreases. It is necessary to have a complete understanding of the processes by which the thermal and mechanical properties of the concrete change during the early stages of reaction in order to reliably predict the stresses that will develop and whether or not the concrete will crack as a result of such stresses.

An accurate representation of the time-space distribution of temperature in a concrete element at early ages is essential for an understanding of the likely stress distribution within the element. The focus of this chapter is on the development of temperature within concrete elements as a result of the heat of cementing reactions. Such information is also necessary for an understanding of the development of the mechanical properties of concrete at various locations within the structure and, at a more practical level, for the contractor to plan activities related to joint grouting, which must take place when the concrete has cooled sufficiently. To provide guidance at the design and construction stages, a numerical model for predicting time-temperature profiles in large concrete structures is discussed and included with this chapter.

A discussion of the thermal properties of concrete, which are of significance in temperature development is also presented here. A full treatment of the cracking potential of concrete due to early temperature development is beyond the scope of this chapter and interested readers should consult references 15.1 and 15.2 for more detail.

15.2 Development of thermal stresses in concrete structures

15.2.1 Free thermal movement of concrete

The coefficient of linear thermal expansion indicates the magnitude of strain in a material in response to a unit change in temperature. In the case of concrete, quoted values of this coefficient vary between 7,5 x 10^{-6}/°C to 12,2 x 10^{-6}/°C for concretes made with typical South African aggregates,[15.3] as shown in Table 15.1.

SANS 10100[15.4] recommends a value of 10 x 10^{-6}/°C, which is suitable for most structural applications.

Table 15.1: Effect of aggregate type on the coefficient of thermal expansion of concrete containing typical South African aggregates[15.3]

Aggregate type	Coefficient of thermal expansion of saturated concrete, x10^{-6} per °K
Witwatersrand quartzite	12,2
Granite (Jukskei, North of Johannesburg)	9,4
Dolomite (Olifantsfontein)	8,6
Malmesbury hornfels (Cape Peninsula)	10,9
Limestone (50/50 by mass Lichtenburg/Ulco)	9,7
Namaqualand onyx	10,3
Dolerite (Orange-Fish Tunnel)	7,5
Felsite (Witbank)	9,2

However, in stress or deflection-sensitive structures, a more accurate value of the coefficient of thermal expansion should be obtained by direct testing. The moisture content of concrete has a significant effect on thermal expansion and the lowest thermal expansion characteristics are obtained for concretes with low water absorption characteristics.

15.2.2 Restraint of thermal movement and development of stress

The development of thermally induced stresses in concrete is directly related to the degree to which potential free thermal expansion and contraction of the structure are restrained. The form of restraint is usually classified as either external or internal.

External restraint occurs when concrete is cast against a rigid structure or foundation and the chemical and mechanical interlock between the new concrete and the rigid surface is such that movement of the concrete close to the interface is restrained. As an example, Figure 15.1 shows a concrete wall cast onto a previously cast, mature base slab.[15.2] During the rising temperature phase of the wall, the expansion of the concrete near the base is restrained and a small compressive stress is developed in the wall. This stress is small because of the low stiffness and high relaxation capacity of the young concrete. However, during the cooling phase of the wall from the peak temperature, the contraction of the wall is restrained by the base slab and the induced stress changes from compressive to tensile. Cracks will occur in the wall when the tensile stress so developed equals the tensile strength of the concrete.

Internal restraint occurs as a result of a non-uniform distribution of temperature in the hardening concrete. The most common situation is where the concrete surface, directly exposed to ambient temperature, is cooler than the internal core of the concrete element. In this case, during the heating phase of the temperature cycle, the greater tendency for the internal sections of the concrete to expand induces tensile stresses in the near-surface zones of the concrete. This results in surface cracks, usually in the form of map cracking. These cracks do not extend far into the concrete and they usually close or reduce in size significantly when the concrete cools down to ambient temperature.

On the other hand, when the internal zone of the concrete cools down from the peak temperature, the tendency of this zone to contract is restrained by the cooler surface zones. This induces a tensile stress in the internal section of the structure. If these stresses are equal to the tensile strength of the concrete, cracks will form in the internal zone of the concrete. Depending on the extent and distribution of these cracks, they can have serious implications for leakage of water-retaining structures and structural cracking of load-carrying elements. Figure 15.2 shows an example of this phenomenon and illustrates the effect of formwork removal in significantly increasing the tensile stress in the surface zone as a result of the increased temperature gradients between the surface and core of the concrete section.[15.2]

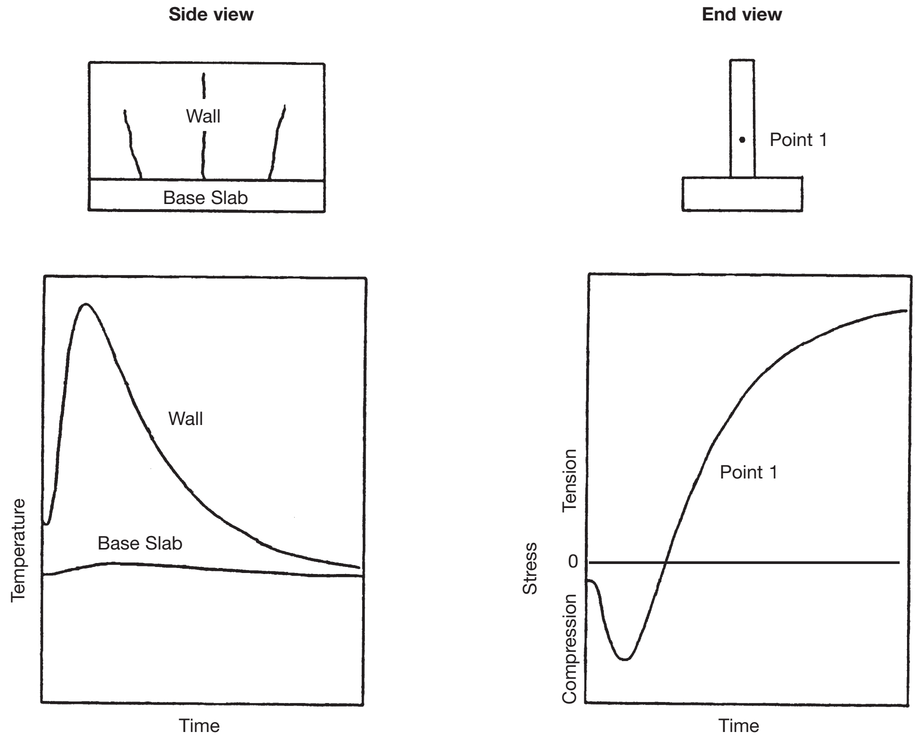


Figure 15.1: An example of the development of cracking stresses in a concrete wall as a result of external restraint presented by the previously cast base slab (from reference 15.2)

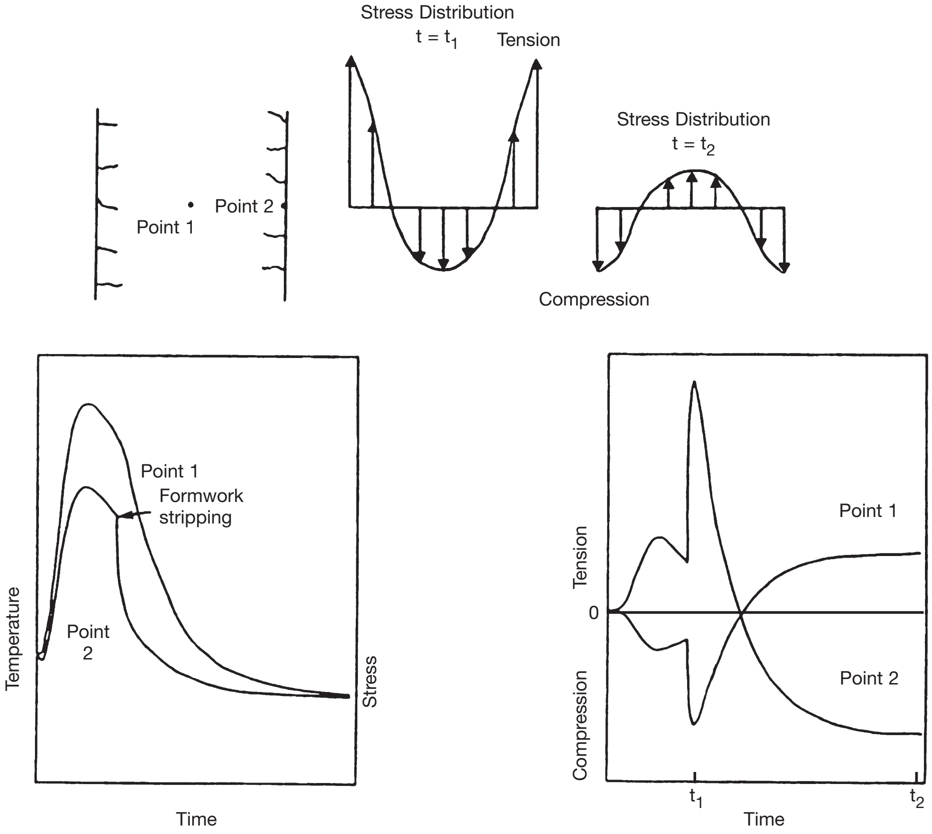


Figure 15.2: Development of stresses in a large concrete element caused by internal restraint as a result of a non-uniform distribution of temperature (from reference 15.2)

15.3 The thermal properties of concrete

15.3.1 Specific heat capacity

Specific heat capacity is defined as the amount of heat energy required to raise the temperature of a unit mass of material by 1°C with SI units of J/kg.K. Specific heat can vary significantly with temperature and, depending on the expected range of temperature variation in the concrete structure, this may have to be accounted for in a prediction model. For concrete, it is common and sufficiently accurate to use mass-weighted average specific heat of the constituent materials.[15.5]

In this context, the specific heat capacity of cement and cement extenders is taken as 880 J/kg.K; a value of 4 187 J/kg.K is used for that of water. Table 15.2 shows values of specific heat capacity, for a range common South African aggregate types at three temperature levels.

15.3.2 Thermal conductivity

Thermal conductivity is a measure of the ability of a material to conduct heat and indicates the heat flow under a given temperature gradient. The coefficient of thermal conductivity of concrete typically varies between 1,2 and 3,5 W/m.K and is dependent on the moisture content of the concrete, type of aggregate, density, and porosity of

Table 15.2: Specific heat capacity of some common aggregates[15.6]

Aggregate type	Specific heat capacity, J/kg.K, at a temperature of:		
	10°C	21°C	32°C
Quartzite	875	910	945
Dolomite	940	965	995
Limestone	925	935	960
Granite	920	920	935
Basalt	945	945	960
Rhyolite	920	945	970

the concrete. Thermal conductivity is also temperature dependent but it is generally accepted that this effect is negligible over the range of temperatures commonly encountered in concrete construction.[15.7] However, the effect of temperature on thermal conductivity should not be neglected when concrete is subjected to very high or very low temperatures as a part of the in-service environmental conditions.

Typical values of conductivity for water-saturated concretes made with a range of common aggregate types are given in Table 15.3. As can be expected, thermal conductivity also varies with the progress of the cementing reactions. However, there is no agreement as to the nature of this variation, with researchers finding both increasing and decreasing thermal conductivity as the reactions proceed. This is shown in Figure 15.3.

Table 15.3: Thermal conductivity of concrete made with a range of common aggregates[15.7]

Type of aggregate	Thermal conductivity of concrete, W/m.K
Quartzite	3,5
Dolomite	3,2
Limestone	2,6 - 3,3
Granite	2,6 - 2,7
Rhyolite	2,2
Basalt	1,9 - 2,2

Campbell-Allen and Thorne[15.9] have developed an extensive mathematical model for determining the thermal conductivity of hardened concrete. Figure 15.4 shows the effects of moisture content, aggregate properties and concrete type on the variation of thermal conductivity of concrete, as determined by this model.

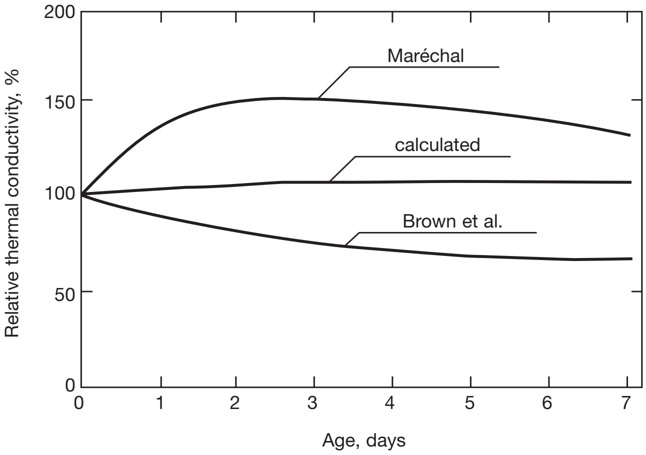


Figure 15.3: Variation of thermal conductivity with time during the early stages of hydration (results from different authors analysed in reference 15.8)

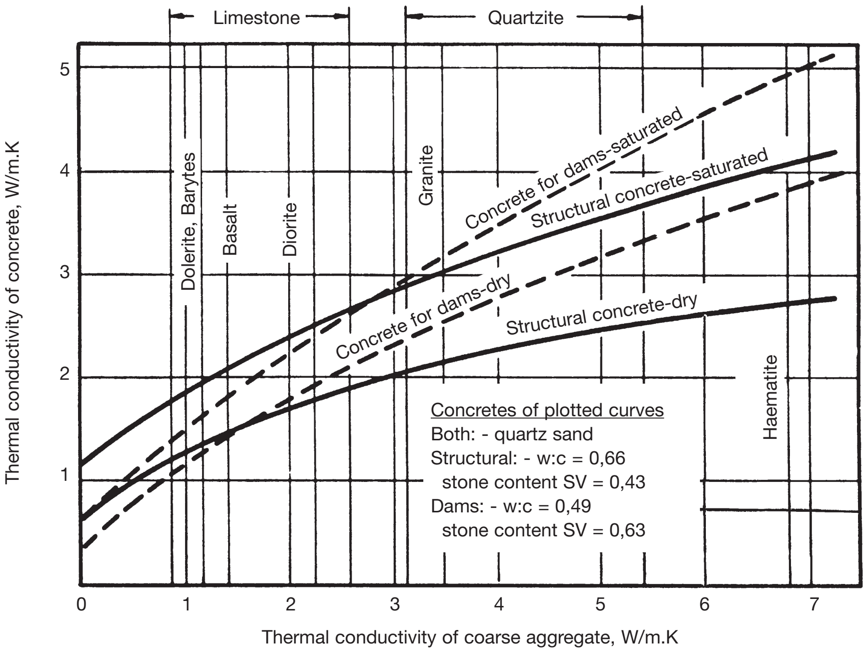


Figure 15.4: Effect of aggregate properties, moisture content and concrete type on the thermal conductivity of concrete - theoretical relationships (determined from reference 15.9)

15.4 Heat evolution from cementing reactions

15.4.1 Rate of heat evolution and the hydration process

The hydration process is described by Roy[15.11] as occurring in three stages as shown in Figure 15.5, which depicts a curve of rate of development of heat of hydration with respect to time. Stage 1 commences immediately after contact with water and results in a very early heat liberation peak due primarily to C_3A hydration and to a lesser extent C_3S[15.10]. Thereafter the heat rate decreases rapidly due to reaction with calcium sulphate to form ettringite which forms a partly protective coating and retards C_3A hydration. During this period the hydration of C_3S proceeds slowly and heat is liberated at a low rate and the hydrate produced consists mainly of CSH (calcium silicate hydrate). The second stage is characterised by rapid heat evolution followed by a period of deceleration, creating a second hydration peak. The hydrates produced during this stage are CSH and $Ca(OH)_2$. Setting occurs near the inflection point preceding the second peak, indicating the formation of sufficient CSH to develop a network structure. A third exothermic peak or shoulder is often encountered due to the conversion of ettringite to the monosulphate hydrate form. C_2S and the ferrite phase also contribute to the formation of CSH and ettringite/ monosulphate respectively.

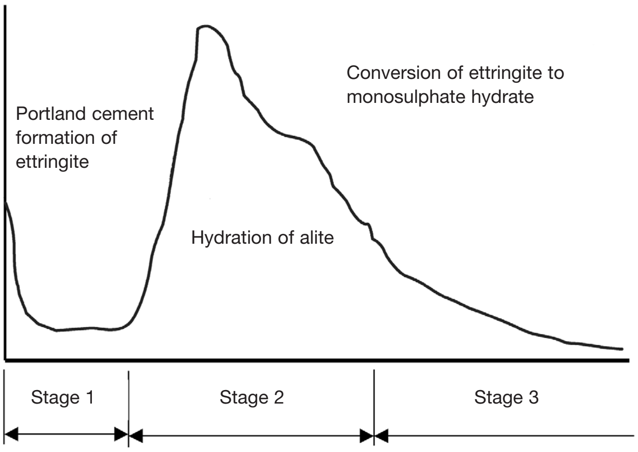


Figure 15.5: Classification of hydration stages by isothermal calorimetry (after Roy)[15.11]

15.4.2 Determination of heat evolution of cementitious binders

A number of test methods have been developed to determine the rate of heat liberated by reacting cement and extenders. These methods can conveniently be divided into three groups: isothermal, adiabatic and semi-adiabatic. Isothermal methods are aimed at measuring the heat generated by samples kept at a constant temperature; semi-adiabatic methods monitor the rate of heat evolution while heat exchange with the surrounding environment is reduced but not entirely prevented. An important problem with these two methods is that the rate of heat evolution is itself temperature dependent. Hence, the heat that is lost to the environment is not available to increase the temperature of the concrete sample and this affects the rate of heat evolution. To address this problem, some researchers have turned to adiabatic calorimetry, in which heat exchange between the sample and the environment is largely eliminated.[15.5, 15.12, 15.13, 15.14]

For the purposes of this discussion, the heat of hydration characteristics of cement are based on adiabatic calorimeter testing, using equipment such as that described by Gibbon and Ballim in reference 15.12 and illustrated schematically in Figure 15.6. Adiabatic testing is convenient, reproducible and practical. It has the added advantage that the test can be conducted on a sample of the actual concrete to be used in the structure.

In principle the test involves placing a one-litre sample of concrete, immediately after casting, in a water bath, such that a stationary pocket of air separates the sample from the water. The signal from a temperature probe placed in the sample is monitored via an analogue-to-digital conversion card by a personal computer, which switches a heater in the water bath on and off to maintain the water at the same temperature as the concrete sample. This ensures that there is no exchange of heat between the concrete sample and the surrounding environment. The pocket of air around the sample is important because it dampens any harmonic responses between the sample and the water temperatures as a result of the measurement sensitivity of the thermal probes. The test is usually run over a period of up to seven days, by which time the rate of heat evolution is too low

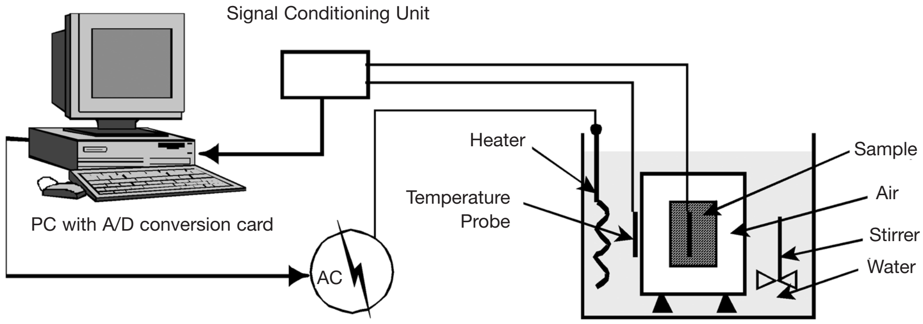


Figure 15.6: Schematic arrangement of an adiabatic calorimeter

to be detected as a temperature difference by the thermal probes.

The test produces a curve of temperature vs. time for a particular binder type and mix composition. The total heat evolved at any time by a unit mass of cementitious binder can be determined from the relationship:

$$q = C_p \cdot \delta T \ \frac{m_s}{m_c} \quad \text{......} \quad 15.1$$

where:

q = heat liberated during hydration per unit cementitious material

C_p = the specific heat capacity of the sample

δT = the change in temperature of the sample over the time period under consideration

m_s = the mass of the sample

m_c = the mass of cementitious material in the sample

The rate of heat evolution can then be determined by means of numerical differentiation with respect to time (t) of the total heat curve ie:

$$\frac{\partial q}{\partial t} \approx \frac{\delta q}{\delta t} \quad \text{......} \quad 15.2$$

An example of the total heat and heat rate curve for a "typical" South African cement, as measured in the WITS University adiabatic calorimeter, is shown in Figure 15.7.

15.4.3 The concrete maturity heat rate

Predicting the temperatures in a concrete element requires a time-based solution of the Fourier equation, which in its two dimensional form is given as:

$$p \cdot C_p \cdot \frac{\partial T}{\partial t} = k \left(\frac{\partial^2 T}{\partial x^2} + \frac{\partial^2 T}{\partial y^2} \right) + \frac{\partial q}{\partial t} \quad \text{......} \quad 15.3$$

where:

p = density of the concrete

Cp = the specific heat capacity of the concrete

T = temperature

t = time

k = thermal conductivity of the concrete

x and y = coordinates at a particular point in a structure

q = heat evolved by the hydrating cement

$\frac{\partial q}{\partial t}$ = the time rate of heat evolution at a particular point (x, y) in the structure. This is the heat rate function as shown in Figure 15.7.

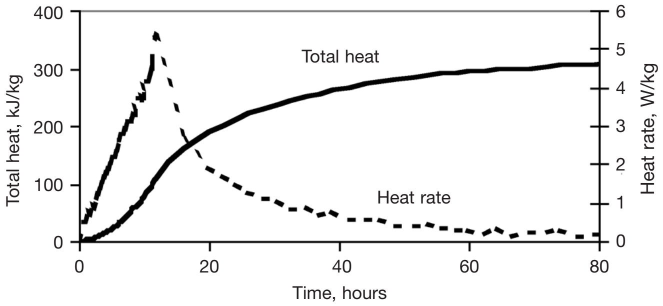


Figure 15.7: Typical heat curves for a local portland cement

However, the heat rate curve shown in Figure 15.7 therefore only applies to the unique temperature conditions under which the adiabatic test was conducted. Since different points in a concrete structure can be expected to experience different time-temperature histories, the rate of heat evolution will vary at different points in the structure. To address this problem, Ballim and Graham[15.15] describe an approach in which the heat rate is expressed in maturity (or extent of hydration) form, rather than in clock-time form. The approach they proposes uses the Arrhenius form of maturity, which can be written as (based on Naik)[15.16]

$$M = \sum_{i=l}^{i=n} \exp \left[\left(\frac{E}{R} \right) \left(\frac{1}{293} - \frac{1}{273 + 0.5 \cdot (T_i - T_{i-1})} \right) \right] \cdot (t_i - t_{i-1})$$

where:

M = the equivalent maturity time as for a concrete cured at 20ºC, expressed as t_{20} hours (see Figure 15.8)

E = the activation energy parameter (33,5 kJ/mole) which is taken as a constant (Bamford and Tipper)[15.17]

R = the universal gas constant (8,314 J/mole)

T_i = the temperature in ºC at the end of the i^{th} time interval

t_i = clock time at the end of the i^{th} interval

The rate of heat evolution curve can therefore be normalised by expressing it as a maturity rate of heat evolution: $\frac{\partial q}{\partial M}$

This form of the heat rate curve can then be used as input for solution of Equation 15.3. The time-based heat rate at each point in the structure at a particular time of analysis is determined from the expression:

$$\frac{\partial q}{\partial t} = \frac{\partial q}{\partial M} \cdot \frac{dM}{dt} \quad \text{......} \quad 15.5$$

Equation 15.5 shows that, in the solution to Equation 15.3 for a concrete structure, it is necessary to monitor the development of maturity as well as the rate of change of maturity at each point under analysis in the structure.

Figure 15.8 shows the heat curves for the portland cement presented in Figure 15.7 above, but expressed in maturity form. In this maturity form, the heat rate curve is suitable for use in a numerical solution of Equation 15.3.

With a knowledge of the specific heat capacity of the concrete, the results of the measured temperature rise in the adiabatic calorimeter can be used to determine the total amount of heat liberated by the hydrating cement over time. In the absence of measured results for a particular binder type, Table 15.4 provides some guidance as to the total heat evolved by South African portland cements (PC) and blends of PC with fly ash (FA) or ground granulated blast-furnace slag (GGBS) under adiabatic conditions. The rate of heat evolution (in units of watts per kg of cementitious material) under adiabatic conditions is then determined by differentiating the total heat evolution curve with respect to time.

A complicating but important issue is that, in an actual structure, the temperature will vary at different points and the rate of reaction at a point at any instant depends on the total amount of heat already released and the temperature at the time under consideration.

Open Temperature Model Excel Spreadsheet

15.5 Finite difference temperature prediction model

Equation 15.3 is the transient form of the Fourier equation in that temperature at any point in the structure varies with time in first order form, as well as with position (x; y) in second order form. Also, the rate of heat evolution is required as input in the solution of Equation 15.3 and this term is time-dependent because the rate of hydration varies with time. An appropriate form of the rate of heat evolution in concrete is developed in Equations 15.4 and 15.5.

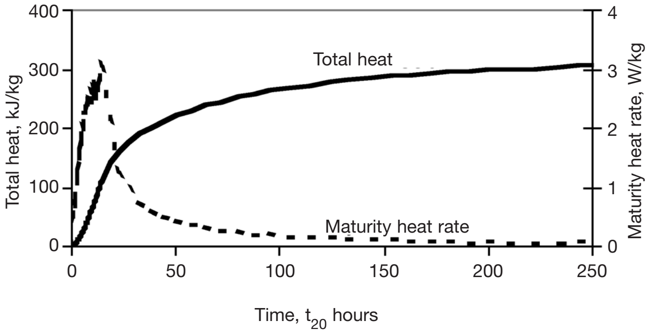


Figure 15.8: Heat curves of Figure 15.7 expressed in maturity form

Table 15.4: Approximate heat of reaction, kJ/kg, after curing at 20°C for different cementitious materials at different ages

Cementitious material	**Approximate heat of reaction, kJ/kg, under adiabatic conditions after:**		
	3 days	5 days	7 days
CEM I	315	350	375
High early strength PC	370	380	390
50% PC + 50% GGBS	240	300	325
30% PC + 70% GGBS	190	240	265
70% PC + 30% FA	250	275	300

The temperature prediction model presented here uses a finite difference numerical technique to solve Equation 15.3 for a rectangular block of concrete, which has a z dimension significantly larger than the x and y dimensions. As shown in Figure 15.9, the block is cast onto a rock foundation (thermal conductivity = k_R) and the ambient air temperature (T_A) varies with time.

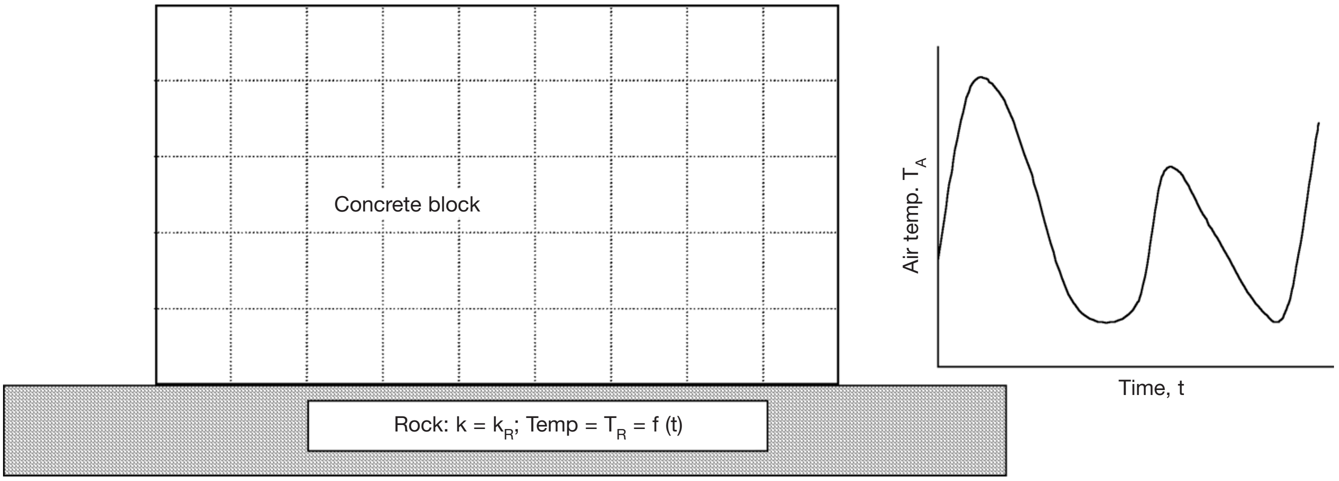


Figure 15.9: Schematic arrangement of the modelled concrete block

15.5.1 Operation of the temperature prediction model

Starting Up

The temperature prediction model which can be found on the CD that accompanies this book, is designed to operate on a desk top computer, using Microsoft Excel 2000® software. The model runs as a macro written in VBA code and the computer should be set up to recognise such macros and alert you to their presence. To the best of our ability, the macro is virus-free and is safe to be enabled. To ensure that the macro can be enabled, proceed as follows:

- Open a blank Excel workbook.
- Select Tools; Macro; Security.
- In the "Security Level" tab, select "Medium" then "OK".

The file containing the model can now be opened from the attached compact disc and is titled: Ballim Temp Model.xls. Each modelling exercise must start with a freshly loaded version of this file. When the file is opened, at the security check prompt, select "Enable".

The file opens on a worksheet titled "Input" which appears as shown in Figure 15.10. Click on one of the selection boxes ("Select Binder Type", say). If this shows box-edit markers around the box, rather than a view of the selection menu, do the following:

- Deselect the box by clicking on another cell.
- Select View; Toolbars; Control Toolbox.
- In the Control Toolbox, click on the "Design Mode" icon (the one with a set square, ruler and pencil). This toggles between the "Edit" and "Operate" modes of the selection boxes.
- Close the Toolbox by clicking on the "x".

Data should only be entered in the blue areas and selections should be made from the three drop-down menus. There are two other worksheets entitled "Analysis" and "Results" which the user should not normally need to access at the model set-up stage. The section below discusses data entry in more detail.

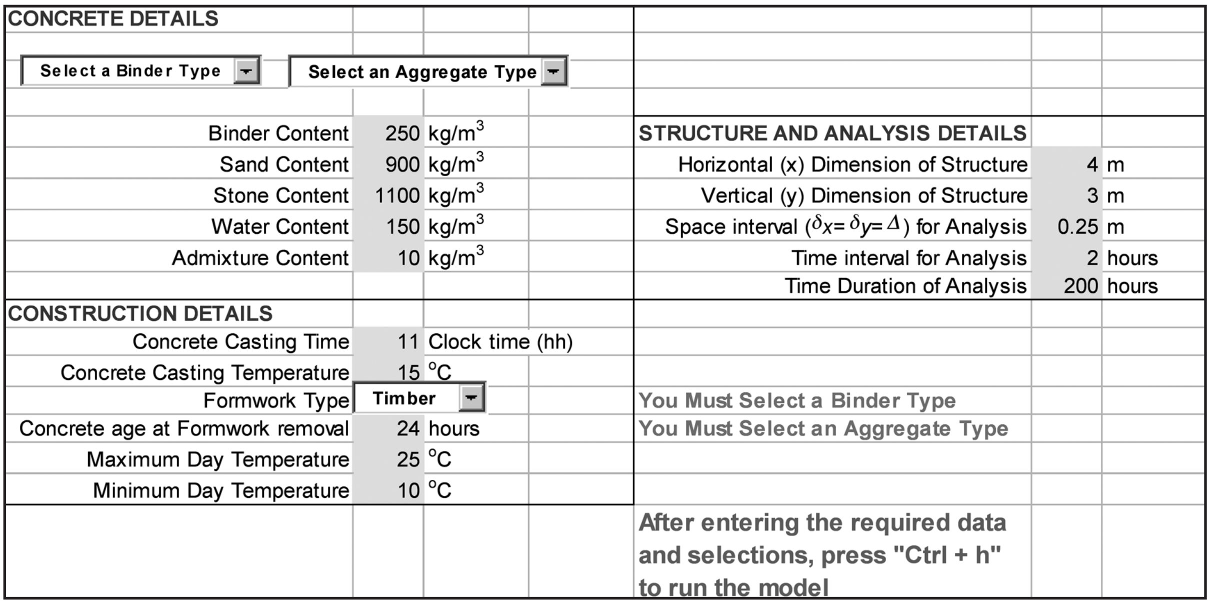
Concrete Details Section

This section calls for details of the actual concrete being considered for the structure. The mass of the different concrete components should be added in the blue areas. In this case, the admixture content refers to liquid admixtures. The mass of powder chemical admixtures should be added to the sand content.

These quantities are used to calculate the density of the concrete (as the sum of the component masses) and the specific heat capacity of the concrete. The specific heat capacity is taken as the weighted average of the specific heats of the solid materials (taken as 900 J/kg.°C) and that of the water and liquid admixture (taken as 4 190 J/kg.°C).

Binder type selection

The software allows a selection of one binder type from a list of seven binders. This sets up the measured maturity heat-rate profile for the binder type selected. The selection includes a typical South African CEM I, three blends of this CEM I with GGBS and three blends of the CEM I



CONCRETE DETAILS

Binder Content 250 kg/m^3
Sand Content 900 kg/m^3
Stone Content 1100 kg/m^3
Water Content 150 kg/m^3
Admixture Content 10 kg/m^3

STRUCTURE AND ANALYSIS DETAILS
Horizontal (x) Dimension of Structure 4 m
Vertical (y) Dimension of Structure 3 m
Space interval ($\delta x = \delta y = \Delta$) for Analysis 0.25 m
Time interval for Analysis 2 hours
Time Duration of Analysis 200 hours

CONSTRUCTION DETAILS
Concrete Casting Time 11 Clock time (hh)
Concrete Casting Temperature 15 °C
Formwork Type Timber
Concrete age at Formwork removal 24 hours
Maximum Day Temperature 25 °C
Minimum Day Temperature 10 °C

You Must Select a Binder Type
You Must Select an Aggregate Type

After entering the required data and selections, press "Ctrl + h" to run the model

Figure 15.10: Appearance of the Input worksheet for data entry before running the model

with FA. Users wishing to run the model with a different heat-rate profile should contact the authors at the contact details given at the end of this chapter. When a binder type selection is made, the red note on the lower right of the screen will turn off.

Aggregate type selection

The main purpose of this selection is to assign a value of the thermal conductivity of the concrete. Concrete thermal conductivity is strongly influenced by the aggregate type and the software allows a selection from the list of aggregates shown in Table 15.3. This table also shows the thermal conductivity values that are assigned to the concrete for use in the model. These values have been adapted from those presented by Van Breugel[15.8].

It should be noted that there can be a range of thermal conductivity values for aggregates which are nominally the same. The values presented in Table 15.3 are "average" values and, as with other parameters, are intended to provide a first estimate of the temperatures in the structure.

Again, when an aggregate type selection is made, the red note on the lower right of the screen will turn off.

Construction Details Section

Concrete casting time

This is the time of day at which the concrete is cast, using a 24-hour clock (eg 4:00 pm should be entered as 16). This value is required to estimate the ambient temperature at each time interval in the analysis.

Concrete casting temperature

This is the initial temperature of the concrete at the time of casting. This value will appear as the temperature at all nodes in the time t = 0 block of the results sheet after the model is run.

The initial temperature of the concrete is an important design and construction variable, which can be used to control the temperature profiles in the concrete. Strategies to cool the aggregates or using chilled water as mix water are effective ways to reduce the initial temperature of the concrete. In the absence of a more direct measurement, the initial temperature of the concrete can be estimated from the mass and specific heat weighted average as shown in Equation 15.6.

$$T_c = \frac{\sum_{i=l}^{n} m_i C_{pi} T_i}{\sum_{i=l}^{n} m_i C_{pi}} \quad \text{.......................} \quad 15.6$$

where:

T_c = the temperature of the concrete mixture

m_i = mass of the i^{th} component of the concrete (cement, sand, stone, water, …)

C_{pi} = specific heat capacity of the i^{th} component

T_i = temperature of the i^{th} component at the time it is introduced into the mixer

The specific heat capacity of the solid materials can be taken as 900 J/kg.ºC and that of the water and liquid admixture can be taken as 4 190 J/kg.ºC.

Formwork type

This is selected from a drop-down menu, which has only two selections: timber or steel. This information is necessary to select the appropriate values for the heat transfer coefficient. The model uses a value of 25 W/m^2.ºC for all exposed concrete surfaces. This value is appropriate for light wind weather conditions. While the side surfaces are covered with formwork, an h value of 5 W/m^2.ºC is used if the formwork is timber, or 25 W/m^2.ºC if steel formwork is used. If no formwork selection is made, the programme defaults to timber formwork. It is assumed that the top surface of the concrete is not covered by any formwork.

Concrete age at formwork removal

As discussed above, this value is used to determine the time at which the value of h should be changed to the exposed value for the side surfaces of the structure.

Maximum and minimum day temperatures

These values are necessary to estimate the ambient temperature at different times during the day. The model takes the time at which the minimum temperature occurs, as 06:00.

Structure and Analysis Details Section

Horizontal and vertical dimensions of the structure

These are the overall dimensions of the structure and set the limits for the node positions in the model analysis. For a rectangular prismatic structure, the smaller of the two horizontal dimensions should be used as this will represent the direction of steepest temperature gradients.

Space interval ($\delta x = \delta y = \Delta$) for analysis

This value determines the spacing of the nodes to be used in the finite-difference analysis. The model temperature predictions will be calculated and reported at each of these nodes. The overall dimension of the structure should be whole number multiples of the space interval selected. If this is not so, the model will extend the overall dimension of the structure to the next nearest node position. There are a few limitations on the value of the space interval selected:

- There should be more than three nodes in both directions
- There should be less than 26 nodes in the x direction and less than 39 nodes in the y direction

- For a particular selection of the space interval, the time interval should be small enough to satisfy the conditions set by Equations 15.7, 15.8, 15.9 and 15.10.

$$\delta t \leq \frac{\rho \cdot C_p \cdot \Delta^2}{4 \cdot k} \qquad \text{15.7}$$

$$\delta t \leq \frac{\rho \cdot C_p \cdot \Delta^2}{k\left(4+2 \cdot \dfrac{k_r}{k}\right)} \qquad \text{15.8}$$

$$\delta t \leq \frac{\rho \cdot C_p \cdot \Delta^2}{4 \cdot k\left[\ 1+\left(\dfrac{h \cdot \Delta}{k}\right)\right]} \qquad \text{15.9}$$

$$\delta t \leq \frac{\rho \cdot C_p \cdot \Delta^2}{4\ (k + h \cdot \Delta)} \qquad \text{15.10}$$

If a value for the space interval is entered and one or more of the conditions mentioned above are not satisfied, an appropriate warning will appear in one of the cells below the "Structure and Analysis Details" box. A different value of space interval should be selected until this warning message is cleared.

Time interval for analysis

This is the δt used in the model and represents the concrete age intervals at which the analysis is undertaken and temperature results reported. The most important limitation on the time interval is that it should satisfy the conditions set by Equations 15.7, 15.8, 15.9 and 15.10. If a time interval is selected which does not satisfy all these conditions, an appropriate warning will appear in one of the cells below the "Structure and Analysis Details" box. A smaller value of time interval (or different value of space interval) should be selected until this warning message is cleared. The model does allow fractions of an hour to be used and it is good practice to use a time interval of four hours or less.

Time duration of analysis

This is the time or concrete age over which the analysis is to be undertaken. The number of time cycles of analysis that will appear in the results sheet is the time duration divided by the time interval selected.

Running the Model

After the data fields have been completed in the "Input" worksheet and the appropriate selections have been made, hold down the "ctrl" button and press "h" to run the model. At the end of the model run, the "Analysis" and "Input" worksheets are deleted and the "Results" worksheet is displayed.

The top of the "Results" worksheet shows the basic data that was used in the analysis. Temperature predictions are shown at each node for successively increasing time intervals. Figure 15.11 shows the first three blocks of results in the analysis of a concrete element measuring 3 m x 2,1 m. The casting temperature of the concrete is 15ºC and the analysis was undertaken in space intervals of 0,3 m with time intervals of two hours.

Saving and Analysing the Results

After the model run, it is important that the results be saved under a new file name using the "Save As" facility. Saving the file without changing the file name may overwrite the master analysis file. Also, the file still contains the macro used for the model analysis but this will no longer operate as a temperature prediction model. It is advisable to delete this macro using the "Tools; Macro …" facility.

The user can now interrogate the data in the "Results" worksheet to consider the time-temperature profiles at different points in the structure. Predicted temperatures at particular points in the structure can also be extracted and compared with temperatures at other points in order to determine the likely temperature gradients in the structure.

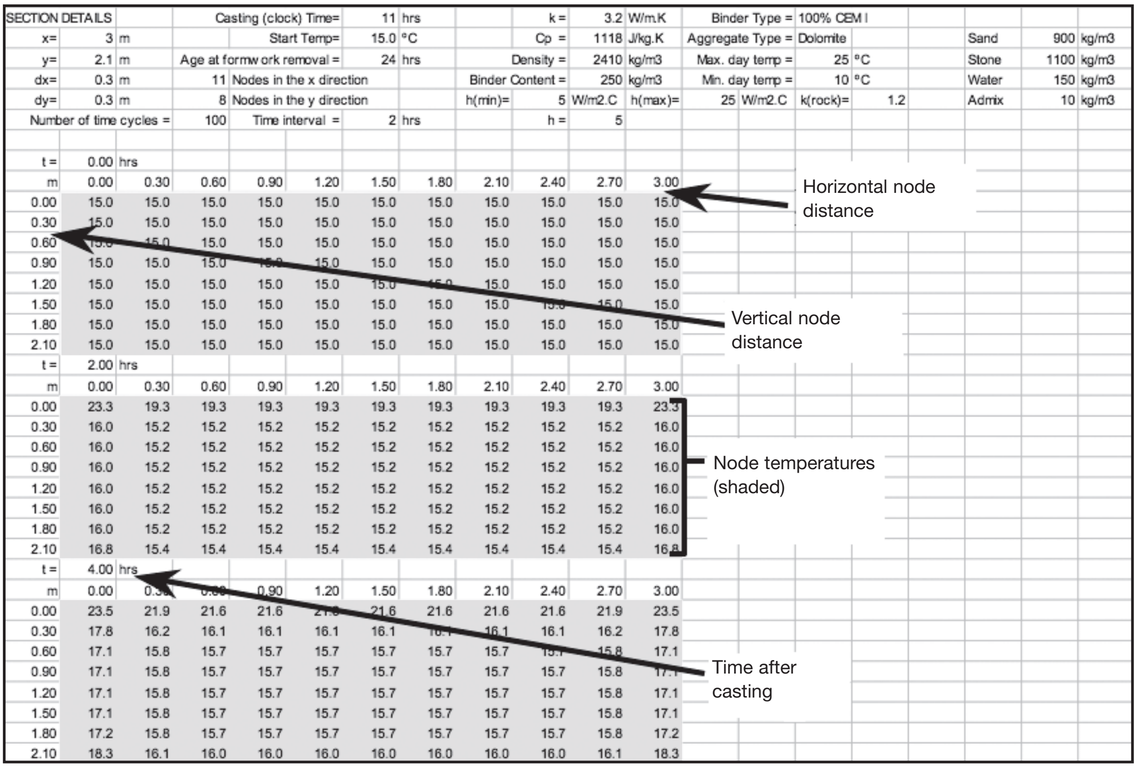
The method described in this chapter provides a simple and reasonably accurate method for predicting temperatures in concrete elements during the early stages of hydration. The model has been structured to be applicable in a wide range of applications. Heat rate data for a typical CEM I and various combinations of FA and GGBS are included in the model.

It must be emphasised that this model was developed as a preliminary design and construction tool, and should be used to provide initial estimates of the temperature profiles that are likely to occur in a large concrete element.

15.6 Practical measures for avoiding thermal cracking at early ages

Many specifications for mass concrete construction attempt to reduce the tendency for thermal cracking of the concrete at early ages by limiting the maximum temperature difference that is allowed to occur in the concrete. This approach has been shown to be inadequate[15.18, 15.19] since it does not account for the cracking potential of the concrete, which, for a particular concrete, varies with time and progress of the cementing reactions.[15.21] In particular, the temperature difference does not take account of the so-called "zero-stress" temperature[15.20] which is the temperature at which the concrete is de-stressed as the internal compressive stresses developed during heating are converted to tensile stresses. Hence, the temperature difference required to cause cracking will vary with time.

In light of this, the strategies to be employed in minimising the risk of thermal cracking of concrete at early ages must address a number of issues.



SECTION DETAILS					Casting (clock) Time=	11	hrs		k =	3.2	W/m.K		Binder Type =	100% CEM I					
x=	3	m			Start Temp=	15.0	°C		Cp =	1118	J/kg.K		Aggregate Type =	Dolomite			Sand	900	kg/m3
y=	2.1	m			Age at formwork removal =	24	hrs		Density =	2410	kg/m3		Max. day temp =	25	°C		Stone	1100	kg/m3
dx=	0.3	m	11	Nodes in the x direction					Binder Content =	250	kg/m3		Min. day temp =	10	°C		Water	150	kg/m3
dy=	0.3	m	8	Nodes in the y direction				h(min)=	5	W/m2.C	h(max)=	25	W/m2.C	k(rock)=	1.2		Admix	10	kg/m3
		Number of time cycles =	100		Time interval =	2	hrs		h =	5									
t =	0.00	hrs																	
m	0.00	0.30	0.60	0.90	1.20	1.50	1.80	2.10	2.40	2.70	3.00								
0.00	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0								
0.30	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0								
0.60	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0								
0.90	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0								
1.20	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0								
1.50	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0								
1.80	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0								
2.10	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0								
t =	2.00	hrs																	
m	0.00	0.30	0.60	0.90	1.20	1.50	1.80	2.10	2.40	2.70	3.00								
0.00	23.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	19.3	23.3								
0.30	16.0	15.2	15.2	15.2	15.2	15.2	15.2	15.2	15.2	15.2	16.0								
0.60	16.0	15.2	15.2	15.2	15.2	15.2	15.2	15.2	15.2	15.2	16.0								
0.90	16.0	15.2	15.2	15.2	15.2	15.2	15.2	15.2	15.2	15.2	16.0								
1.20	16.0	15.2	15.2	15.2	15.2	15.2	15.2	15.2	15.2	15.2	16.0								
1.50	16.0	15.2	15.2	15.2	15.2	15.2	15.2	15.2	15.2	15.2	16.0								
1.80	16.0	15.2	15.2	15.2	15.2	15.2	15.2	15.2	15.2	15.2	16.0								
2.10	16.8	15.4	15.4	15.4	15.4	15.4	15.4	15.4	15.4	15.4	16.8								
t =	4.00	hrs																	
m	0.00	0.30	0.60	0.90	1.20	1.50	1.80	2.10	2.40	2.70	3.00								
0.00	23.5	21.9	21.6	21.6	21.6	21.6	21.6	21.6	21.6	21.9	23.5								
0.30	17.8	16.2	16.1	16.1	16.1	16.1	16.1	16.1	16.1	16.2	17.8								
0.60	17.1	15.8	15.7	15.7	15.7	15.7	15.7	15.7	15.7	15.8	17.1								
0.90	17.1	15.8	15.7	15.7	15.7	15.7	15.7	15.7	15.7	15.8	17.1								
1.20	17.1	15.8	15.7	15.7	15.7	15.7	15.7	15.7	15.7	15.8	17.1								
1.50	17.1	15.8	15.7	15.7	15.7	15.7	15.7	15.7	15.7	15.8	17.1								
1.80	17.2	15.8	15.7	15.7	15.7	15.7	15.7	15.7	15.7	15.8	17.2								
2.10	18.3	16.1	16.0	16.0	16.0	16.0	16.0	16.0	16.0	16.1	18.3								

Horizontal node distance

Vertical node distance

Node temperatures (shaded)

Time after casting

Figure 15.11: Example of results for the first three time intervals as produced by the model

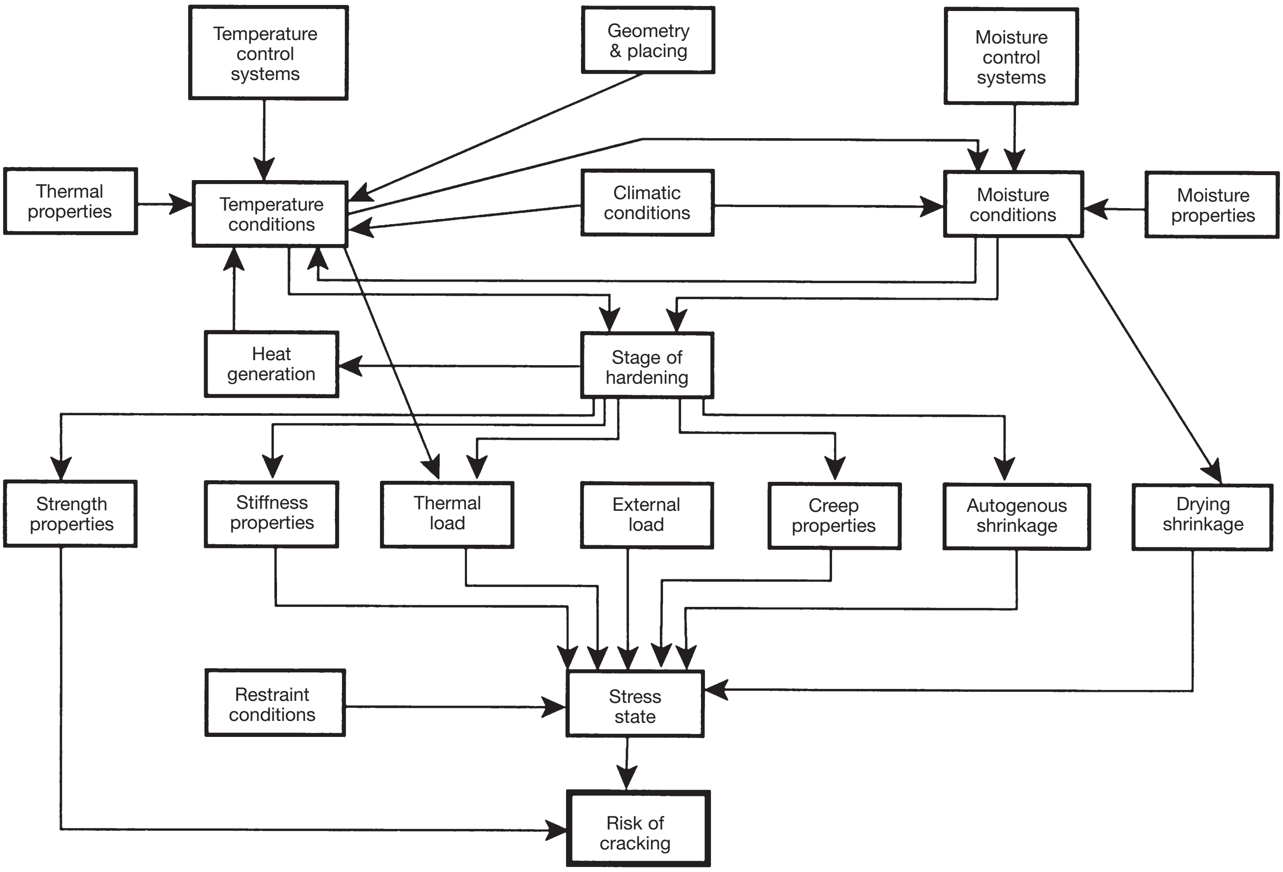


Figure 15.12: The main interacting factors that influence the risk of cracking in concrete (based on reference 15.2)

1. Minimise the temperature differences that occur in the structure;
2. Minimise the degree of restraint that may cause the development of stress;
3. Use concrete in which the transient mechanical properties are such as to maximise the resistance to cracking.

As shown in Figure 15.12, there are a large number and range of factors that interact to influence the risk of cracking and any actions aimed at addressing this problem should therefore be carefully designed and monitored. The discussion that follows will present some broad principles as to the strategies that could be used to minimise cracking. For more detail, interested readers should refer to the specialist literature on the subject (see, for example, references 15.1, 15.2, 15.7 and 15.22).

15.6.1 Practical measures at the design and pre-construction stage

Member size

Section dimensions should be kept as small as possible and consideration should be given to the use of hollow or voided sections. As a general guide, the maximum temperature rise that will occur at the centre of a section that is at least 2 m thick, is approximately equal the maximum temperature rise under adiabatic conditions.

Project specifications

- Consider the use of a longer term concrete strength specification, eg 56 or 90 days, instead of a 28-day strength specification. This will allow the use of a more slowly reacting cemetitious material and a lower cementitious content.
- In light of the above discussion, consider carefully the value of a specification based on the maximum temperature difference in the concrete. If such a specification is used, consideration should be given to the significance of temperature differences that occur at different times during the early stage after concrete has been cast.
- Consider the use of high-energy compaction equipment which will allow concrete with a lower water content and lower cementitious content to be used. Of course, this concept can be extended to the use of roller-compacted concrete.
- Reduce restraint by suitable design and location of construction joints as well as careful planning of construction and concrete placing sequences.[15.22]

Concrete mix design

- Use a so-called "low heat" cementitious material in the concrete and maximise the proportion of cement extenders like FA and GGBS. While these materials do not significantly reduce the total amount of heat liberated by the concrete, they do reduce the rate of the heat evolution. Hence, depending on section geometry and environmental conditions, the maximum temperature is reduced and the time of occurrence of this maximum is lengthened. This is likely to result in tensile stresses being brought to bear on the concrete at a time when the physical properties have developed sufficiently to resist such stresses.

- Employ strategies and materials that are aimed at reducing the water content of the concrete mix. This reduces the cementitious content and, hence, the amount of heat liberated; it also reduces the drying and autogenous shrinkage, which in turn reduces the total restraint stress on the concrete. As far as possible, such strategies will include the use of:
  - Water reducing admixtures
  - As large a maximum aggregate size as possible,
  - A well graded, total aggregate profile

In considering the aggregate particle shape, the benefits of more rounded particles in reducing the water requirement of the mix should be weighed against the improved tensile strength that concrete derives from angular aggregate particles.

- Using a cracking frame which they developed, Springenschmid and Brietenbucher[15.23] recommend that, in order to improve the cracking resistance of concrete, the mix should be designed so as to:
  - Have a low early strength development
  - Include a partial cement replacement with FA
  - Use aggregates with low coefficients of thermal expansion
  - Include approximately 4% of entrained air
  - Avoid the use of silica fume

15.6.2 Practical measures during the construction stage

Reducing the concrete placing temperature

Cooling the concrete to a level below the ambient temperature at the time of placing is an effective way to reduce the maximum temperature rise in the concrete. Bernander[15.24] recommends that in most circumstances, the fresh concrete temperature should be between 8 and 12ºC. Furthermore, where possible, the concrete should be placed during the late afternoon so that the ambient temperature is low during the critical early stages of reaction. Concrete cooling can be achieved by:

- Shading the aggregate stockpiles from direct sunlight
- Controlled water sprinkling of the aggregate stockpiles

- Burying water pipes and/or painting all exposed pipes and tanks white
- Using crushed ice as the mixing water
- Injecting liquid nitrogen into the concrete during mixing

Formwork and insulation

Insulating the concrete with timber formwork and top surface covers after casting will reduce the temperature gradients within the concrete but will also increase the maximum temperature rise in the concrete. Depending on the rate and conditions of cooling, this may result in more significant internal cracking than would have occurred had no insulation been used. This aspect requires careful consideration. Harrison[15.22] recommends that, for sections that are less that 500 mm thick, steel forms should be used in order to maximise the rate of heat loss from the surface. On the other hand, for large isolated sections, insulated formwork and slow cooling should be used in order to minimise the temperature gradients. The same considerations are relevant to formwork striking times. It should also be noted that, in cases where the grouting and finishing of joints can be done only when the concrete has returned to approximately ambient temperature, insulation will delay the process and may extend the construction period.

Internal water cooling

Cooling the concrete by pumping chilled water through a network of thin-walled pipes cast into the concrete is probably the most effective way of reducing the internal concrete temperature. This method can significantly decrease the maximum temperatures reached and the post-peak heat removal period. This means that the concrete volume changes occur over a shorter period during the earlier age of the concrete, when the creep capacity is high and the elastic modulus is low, thereby reducing the stress development in the concrete. A word of caution is necessary regarding the termination of cooling – an abrupt termination of the cooling system can cause sufficiently large tensile stresses to crack the concrete internally when a new temperature equilibrium is established. Also, if the cementing reactions have been significantly suppressed in the vicinity of the cooling pipes, they will be re-initiated when the cooling system is terminated. Again, if the cooling system is terminated abruptly, this could cause tensile stresses to develop around locally heating areas which may result in cracking. Furthermore, this has implications for the timing of joint grouting in mass concrete structures, which must be completed soon after the termination of the cooling procedure. Consideration should also be given to the compressive stresses that may be brought to bear on the grouting material when the cementing reactions are re-initiated and the internal temperature of the concrete increases.

The design and operation of cooling systems is a complex subject and is outside the scope of this chapter. Interested readers should refer to Chapter 4 of reference 15.7 as well as reference 15.25.

Construction sequence

The sequence of construction influences the degree of restraint that previously cast concrete offers to newly cast concrete, particularly in mass concrete construction such as large dams. The sequence of casting bays or blocks in both the vertical and horizontal directions requires careful planning in order to minimise restraint. As an example, Harrison[15.22] shows that "sequential" construction (freshly cast concrete is in contact with previously cast concrete on the bottom and one of the vertical faces only) results in less restraint than "alternate bay" construction (contact is on the bottom and both vertical faces).

As regards vertical construction lifts, it is recommended that the time interval between casting successive lifts should be kept at a minimum. If a concrete lift is placed onto a previous lift that is still warm, because the lower lift is still contracting, the degree of restraint which it offers to the contraction of the upper lift is reduced. Hence, the cracking potential is reduced. In the case of slipform construction, this principle can be used to almost eliminate thermally induced restraint stresses.

Finally, as an example of minimising restraint stresses by considering the rigidity of the structure, when a large wall and base construction is undertaken, a wall kicker, approximately 1,5 m high, should be cast as soon as possible after the base has been cast. The kicker, being less rigid than the base, will result in the development lower restraint stresses when the rest of the wall is cast onto it.

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Chapter 16

Control of concrete quality

Mike McDonald

16.1 Introduction

Monitoring and controlling the quality of ready-mixed concrete is similar to any process control system. However, the nature of the raw materials used in the production of concrete and the large number of factors affecting both initial and final strength make concrete a highly variable product, with a coefficient of variation typically between 10 and 20%.[16.1]

Controlling the quality of the different concrete mixes used on any project is further complicated by the use of different cement types and contents, aggregate availability, workability requirements entailing the use of chemical admixtures, and rate of strength gain and strength required at different ages.

ISO 2394[16.2] distinguishes between production control and compliance control:

- Production control entails ensuring and monitoring "an acceptable quality of design, materials, products and work on the construction site," and detecting "gross errors and other hazards." The purpose of production control is to steer a production process and thereby guarantee an acceptable result.
- Compliance control consists of monitoring data relating to specific properties of the concrete. The purpose of compliance control is to ensure that the result of the production process complies with given specifications.

Although the properties of materials, components or structures that are the object of control may not necessarily be the same in these two control procedures, the same process is followed:[16.2]

- Collection of information
- Judgement based on this information
- Decision based upon the judgement

16.2 Factors affecting quality of concrete

Many factors influence the strength and quality of concrete. Probably the most comprehensive list was published by Boyd Mercer[16.3] who named 60 factors. Sources of difference may be grouped in five categories:

- Different raw materials, which may differ greatly in their concrete-making properties.
- Mix proportions of given materials may differ, producing concrete with very different properties.
- Random variations of constituents combined with random variation in mix proportions and mixing, which may result in significant variation in concrete properties.
- The effect of non-representative sampling and inaccurate testing, which may be substantial.
- Process errors, which may be due to either to equipment or operators or a combination of both.

16.3 Codes, specifications and control systems

With this inherent variability in concrete, the question must be asked how can one ensure that concrete of suitable quality has been produced and used in the structure?

Early solutions to this problem were prescriptive by nature. The properties of the constituent materials and proportions in which these were to be combined were defined in detail. This was an inconvenient approach especially where local materials differed from those prescribed. This in turn entailed providing materials from distant sources with attendant high transportation costs or extensive and possibly unnecessary modification of the local materials, also with adverse cost implications.

Later, compressive strength came to be recognised as the most critical property of structural concrete and was specified in construction codes in terms of the minimum strengths acceptable for different grades.

Compressive strength testing is destructive and must be conducted on specially prepared samples, not on actual structures. It is therefore not possible to apply total control in the fullest sense to concrete. In this situation, statistical control is required. This was recognised in early papers.[16.4-16.7] Concrete quality assessment in compliance with durability requirements is as important as the fulfilment of structural requirements. The convenience of compressive and tensile strength testing has ensured that durability may be indexed to appropriate compressive strength standards. A recent development in South Africa is the use of indices derived from oxygen permeability, chloride conductivity and water sorptivity test results.[16.8]

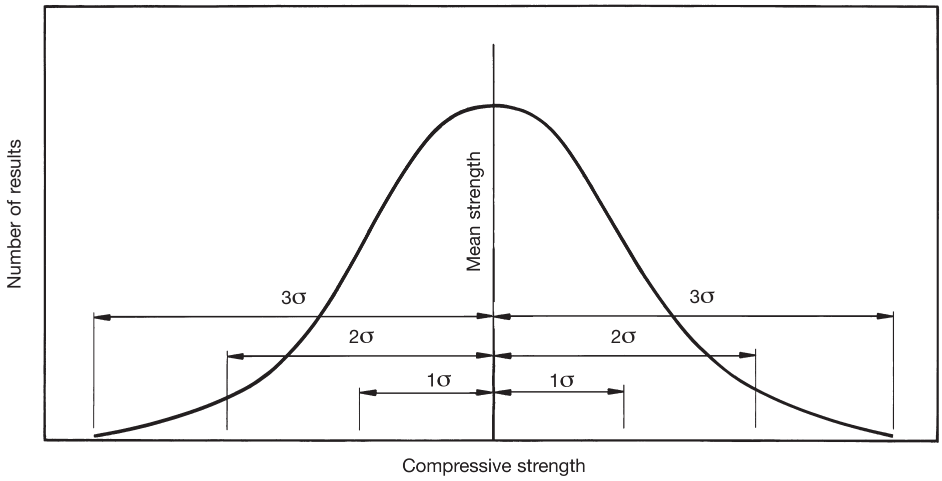

Figure 16.1: Normal distribution curve; σ = standard deviation
(Curve is truncated to have an overall width of six times standard deviation.)

In concrete quality control, the first assumption in the application of statistics is that the distribution of strengths for a given grade of concrete follows the *normal* or *gaussian distribution* (see Figure 16.1). This provides a convenient mathematical model defined by two parameters: mean and standard deviation. If these are known, the likelihood of different proportions of the material defined deviating by more than known values from a given value can be estimated. In practice[16.9] concrete strengths are not strictly random normal variables but are subject to changing bias. Bias is a tendency for results obtained over a limited period to exhibit a significant likelihood of departure from the intended statistical distribution.

Control entails identification of and correction for this bias by assigning appropriate target values for mean and standard deviation which define the intended normal distribution. Mixes are then adjusted on an ongoing basis when trends away from the target values occur.

This second principle lies in the application of *stochastic methods,* which entails defining and maintaining targeted values.[16.9] Stochastic is defined as "subject to chance" but also as "trying to achieve a target". In concrete quality control, mix proportions are adjusted if test data indicate that actual standards being achieved differ from those used or assumed in setting the mix parameters. The adjustments may either be to restore performance to intended levels or to attain different levels if actual control parameters are found to differ significantly from those assumed in setting target strengths.

Partial strength and load factors implied in structural design codes assume a normal distribution for both load on and material strength in structures. Where stresses due to loads exceed strength on a probabilistic basis, structural failure can occur. Specified material strength requirements are therefore set to ensure that the probability of concrete strengths being lower than the critical value is small. While the theoretical normal curves extend to infinity, in practice curves are modified to have extreme values three standard deviations from the mean.

In earlier codes, minimum strength values were specified, but the current trend is to specify a higher value and to permit not more than a certain percentage of results to fall below this value.

This specified strength is termed the characteristic strength (CS) in South African structural codes.[16.10] This value must be exceeded by at least 95% of test results where tests are carried out using standard test methods.[16.11-16.16] CS is still linked to a lower "minimum" value and criteria are laid down with respect to both acceptance limits and the action required if these are not attained.

It is now also appreciated that control procedures cover two aspects of concrete manufacture and construction, and that the property of the material under review, principally compressive strength, will be different in respect of these two criteria. In early codes, emphasis was entirely on compliance control in respect of structural elements already placed. While this process remains in current codes and specifications, it is now complemented by production control, the purpose of which is to increase the assurance that the concrete produced will be acceptable. This change in approach is essential as the strength of concrete is established only 28 days after it is mixed and placed.

The following section deals with the elements of these control processes as called for in South African construction codes and specifications.

16.4 Practical quality control

The principal elements of a concrete quality-control programme should cover the following processes:

1. Determination of appropriate target strengths for the various grades of concrete.
2. Designing mixes for the target strengths required.
3. Sampling and testing of an adequate proportion of field production.
4. Statistical evaluation of test results in relation to defined performance standards.
5. Decision regarding acceptance relating to production and compliance control.
6. Remedial action if concrete is deemed non-compliant with the defined requirements.

16.4.1 Target design strength

South African structural design practice conforms with British and European codes with regard to characteristic strength (CS). For a normally distributed property this condition requires the average strength, which equates to the target design strength, to exceed the CS by not less than 1,64 times the standard deviation (SD) of the population.

Where test data are available from previous production with given materials and production facilities, a suitable estimate of SD may be calculated. New production facilities with no record of performance capability pose a problem. Certain international structural codes provide specific guidelines as to the initial strength margins between target average and specified (deemed to be CS) values to be used in this situation. Table 16.1 gives examples of SDs for different levels of control and hence the minimum margins between target and specified strengths.

Table 16.1: Examples of degrees of control, standard deviation and margins

Control	SD, MPa	Minimum margin, MPa
Poor	7	11,5
Average	6	9,8
Good	5	8,2

In practice, SDs of between 3 and 4 MPa are common for well managed batching plants, and values of less than 3 MPa are maintained consistently by some facilities.

Unless prior results are available, it is advisable to assume poor control in setting margins. In the early stages of a project, sampling and testing frequencies should be increased to substantiate assumed performance standards.

Although by definition CS entails not less than 95% of test results exceeding the concrete grade strength, certain code provisions make it expedient to set target margins greater than 1,64 x SD for the difference between average strengths and specified CS. This is particularly significant in respect of section 14.3.3.1 of SANS 10100-2:1992,[16.17] which reduced the required excess strength margin to more realistic levels. The average of any three consecutive test results should not be less than 2 MPa in excess of the CS to provide a safeguard that production standards are being maintained.

16.4.2 Mix design

Strength is primarily a function of water:cement ratio w:c and for given requirements, w:c is determined from recent past performance for the materials and the concrete production facility. (See Chapter 11 for guidance on the determination of suitable mix proportions.)

In the absence of recent performance data for the materials to be used on a project, conduct preliminary trial mixes in a suitably equipped laboratory to provide guidelines for the balance between w:c, strength and water requirements of aggregates and cement. A conservative approach should be adopted until sufficient field test results are available to provide a basis for adjusting mixes.

16.4.3 Sampling and testing

The testing rate for structural concrete varies depending on the criticality and complexity of the project. The rate should be increased in the early stages of a project.

It is important that testing is executed competently and accurately; the expense is generally insignificant in the total cost of the project.

For a truly statistical approach, representative samples of concrete[16.11] should be taken from random batches once the batcher has adjusted initial batches to achieve the consistence sought.

The statistical approach does not address gross operator error or equipment malfunction. Experienced operators and supervisors continually assess the quality of concrete visually and should be authorised to discard batches deemed incorrect, establish the cause of the error and, where possible, rectify the problem.

If concrete varies visibly throughout the discharge, it may be inferred that mixing is inadequate. Testing to evaluate mix uniformity within individual batches is not generally included in routine testing, but both BS [16.18] and ASTM [16.19] standards give acceptance criteria and tolerances.

The evaluation of concrete mix quality is based upon test results for cubes cured under water at controlled temperatures.[16.12] Results obtained from specimens exposed on site to evaluate the influence of ambient conditions do not form part of the control data for the mixes. Low strength test results of exposed test specimens should be addressed by protection and curing of concrete on site, not by adjustments to mix proportions to compensate for poor site practice. Great care should be exercised in using this approach as cube specimens with a lack of adequate mass will suffer far more significant adverse effects of exposure to cold weather than most structures.

South African practice defines a valid concrete test result as the average compressive strength test results of three specimens from a single batch of concrete tested at the same age under standard conditions. The 28-day strength is the normal control standard but earlier tests are also made for production control, and predictions of 28-day strengths are based on the results of early tests.

A test is deemed not valid if the range in the set of cubes exceeds 15% of the average strength. If this occurs, an investigation of possible inadequate sample preparation, substandard cube moulds, compression machine malfunction, or operator error should be established (and rectified). Interpretation of these test results to obtain an indication of structural adequacy may be necessary, entailing a higher level of engineering judgement.

16.4.4 Acceptance criteria

The principal South African specification in which acceptance criteria for structural concrete are defined is SANS 2001-CC1:2007, *Construction works Part CC1: Concrete works (structural).*[16.20] Action to be taken in the event of non-compliance may be as laid down in SANS 10100-2:1992, amended 1994.[16.17]

Section 5 of SANS 2001-CC1 covers compliance with the requirements as follows:

5.1.2 Acceptance of strength concrete

5.1.2.1 Should any valid test result obtained on concrete of a specific grade show that the strength is more than 3 MPa below the specified strength, the concrete yielding such result shall be deemed not to comply with the requirements of this part of SANS 2001.

5.1.2.2. The average of any three consecutive valid test results obtained on concrete of a specific grade shall exceed the specified strength by at least 2 MPa.

SANS 2001-CCI has additional requirements for larger groups of consecutive results. However, C&CI believes that 5.1.2.1 and 5.1.2.2 provide sufficiently onerous requirements and that the additional section is unnecessary.

5.1.2.3 *Where a concreting operation is of such magnitude or the sampling of such frequency that 30 or more valid test results have become available within three months, and if permitted or required in terms of the specification data, the results shall be assessed statistically instead of in terms of 5.1.2.2, as relevant. In such case, the average of all the test results of a specific grade at any stage shall exceed the specified strength by at least 1,64 standard deviations. If the average of such test results fails to meet this requirement, the mix design shall be adjusted to ensure compliance with this criterion.*

16.4.5 Presentation of control data

Control charts

The need to use control charts stems from the variability that is found in compressive strength test results. Different forms of control chart have been devised to provide a visual guide on performance.

Single-result chart

The simplest chart to monitor on site is one where each result for a given grade of concrete is plotted sequentially as shown in Figure 16.2. Lines on this chart to indicate when action is necessary would be:

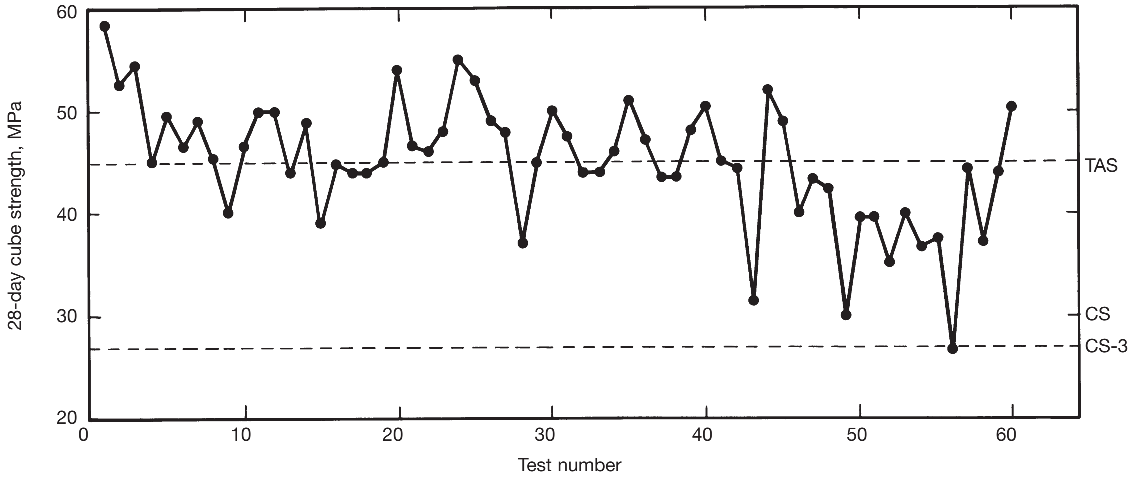

Figure 16.2: Chart of individual strength results for 30 MPa concrete, target average strength 45 MPa

1. CS
2. CS minus 3 MPa
3. Target average strength (TAS)

The scatter on this chart is great. While the minimum strength line, ie CS minus 3 MPa, is simple to interpret, the same cannot be said for the CS and TAS levels. Not more than one result in 20 should fall below CS level and half the results should exceed TAS. These requirements are not readily interpreted on the graphical presentation and the question of bias in the results is not addressed.

Moving-average chart

The moving average of a selected size group of results is a form of control chart which provides a better picture of distribution, but which entails more arithmetical calculation. In the context of clause 5.1.2.2 of SANS 2001-CC1, the logical group size would be three test results. Action levels would be CS and CS plus 2 MPa for compliance and production control respectively. The same data as depicted in Figure 16.2 are presented in this revised format in Figure 16.3. The tendency to bias is slightly clearer.

Individual values provide negligible assistance in terms of production control assessment and even a moving average of three results represents a very limited population in this respect.

CUSUM technique [16.21, 16.22]

In the CUSUM method, individual results are plotted not as absolute values but successively as the cumulative

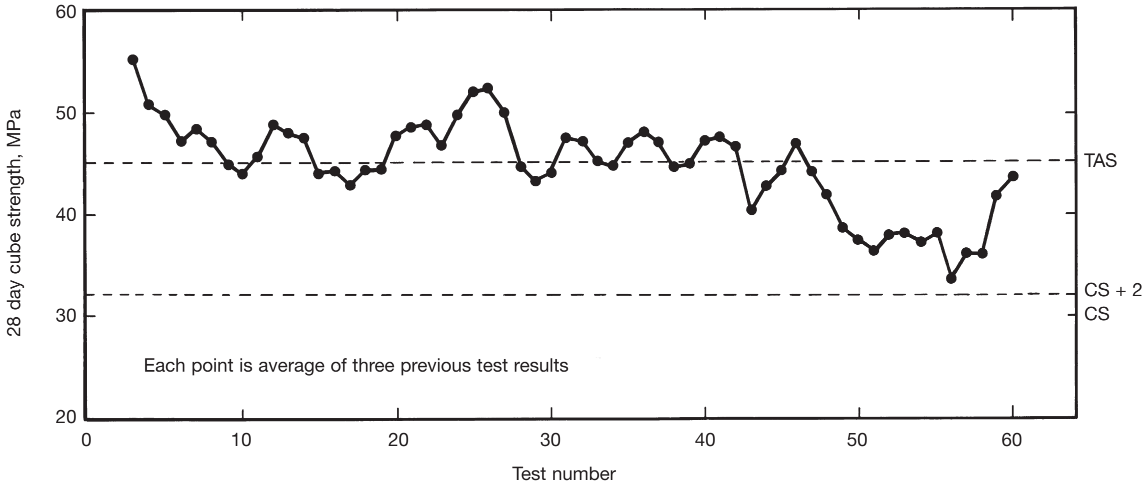

Figure 16.3: Chart of moving average strength of consecutive groups of three tests

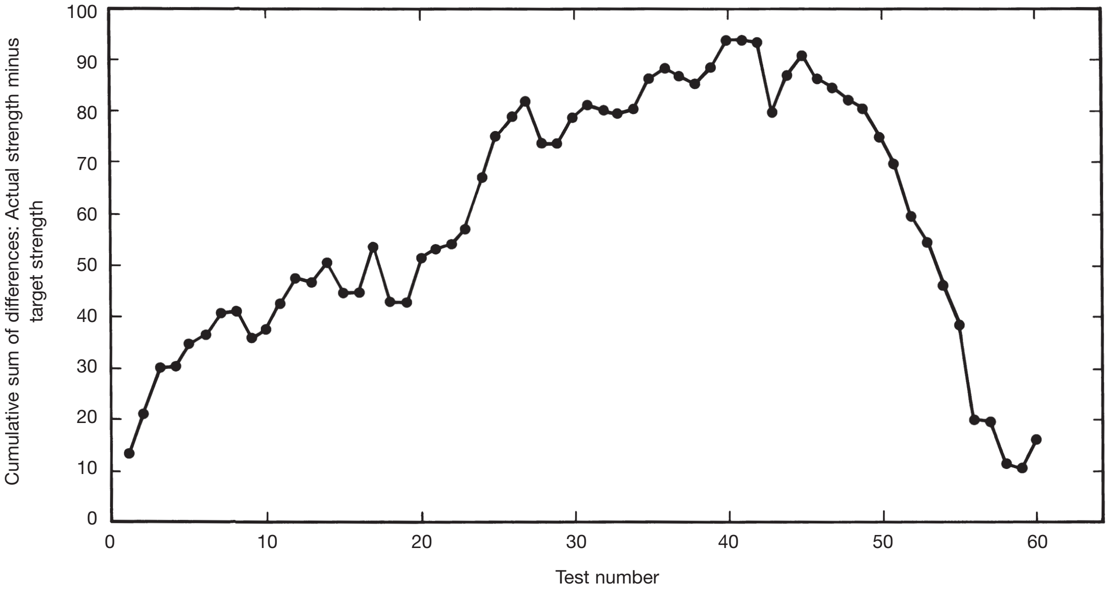

Figure 16.4: Chart for CUSUM of actual vs target strength

sum of differences between the actual and target values. If the mean of actual values is equal to the target over a number of results, the average slope for this range will be horizontal. This trend is clear even if the actual plot is very irregular with a property as variable as concrete strength. However, if the property being measured is tending away from the target value, the slope of the CUSUM plot will reveal the difference more readily than a standard control chart. An upward slope will indicate that the target value is being exceeded on average. Conversely, a downward slope will reveal a shortfall relative to the target value. Presented in this manner, trends, and more particularly changes in trend, are more clearly evident than in other forms of data presentation. As a result, the need for corrective action is recognised sooner, very often before a statistical result with a high level of confidence would emerge from the same data using conventional analysis methods. Figure 16.4 depicts the same data as Figures 16.2 and 16.3 but in a CUSUM format.

The CUSUM method, offered as one quality control approach in the British Ready Mixed Concrete Association's Authorisation Scheme manual,[16.23] covers all of the four aspects listed above in the stochastic approach. In addition, it increases the data available for analysis by reducing all results to a common strength standard.

Estimated 28-day strength

This is the prime control in the CUSUM system. Early strengths are generally derived from seven-day tests on cubes cured under standard conditions, but 24-hour accelerated tests are preferable where appropriate correlations have been developed, as these can reduce the delay in effecting corrective action. If a single grade of concrete is being analysed, then a factor can suffice to estimate the corresponding 28-day strength. If, however, the results from a range of mixes are being combined, then at least a linear relationship between the early test value and the estimated 28-day strength should be used. A polynomial relationship may provide a more accurate result.

The technique used in massing results is to adopt a single strength grade as the standard. Actual test results for other strength grades are adjusted to the standard by the difference between the target design strengths corresponding to the two different grades, not the difference between the characteristic strengths.

It is also important to assign an appropriate strength to any mixes that have either a specified minimum cement content or maximum w:c which produces different target mean strengths from those normally associated with the nominal grades. If these strength adjustments are not made, these mixes should not be included in the CUSUM analysis on the basis of their nominal grades.

The CUSUM plot of the estimated 28-day strength is used to evaluate whether adjustments to mix proportions are required. As the plot is based upon mean strengths and the deviation from the target mean can be derived easily from the average slope over the group of results exhibiting the trend requiring adjustment, this adjustment is clearly defined. Appropriate mix changes either in terms of w:c or cement content are readily quantified.

Estimated 28-day strength-actual strength correlation

The age-strength pattern of cement fluctuates as much as the 28-day strength does. Consequently, it is necessary to compare actual strengths with predicted values and the difference between these is the second control in the BRMCA[16.23] CUSUM analysis. If this CUSUM line reveals a trend away from the horizontal, two adjustments to the control process are made. Firstly, a new correlation relationship is determined. This is applied to further tests until the need for revision occurs again. Secondly, the revised correlation relationship is applied retrospectively to the data covering the span from which the new relationship was derived. This analysis may indicate that adjustments made to mixes during this period require modification.

When massing results, it is important to differentiate between values from mixes of greater and lesser strength than the control value. It can happen that the inclusion of the assumed w:c or cement content-to-strength relationship differs from the actual relationship. If the two curves intersect at the control mix level, the overall CUSUM will appear satisfactory, but the mixes will deviate increasingly from target as they move away from the control mix strength.

To check whether the strength vs w:c relationship used is appropriate and reliable, results of tests representing concrete of higher than the control strength should be grouped in the CUSUM analysis. Similarly, results from the lower strength mixes are grouped. Differences in the trends of these two sets of concurrent results over any period will indicate when the relationship in use should be reassessed. In effect mini-CUSUM analyses are made within the overall analysis to carry out this appraisal, and these should be repeated regularly.

Standard deviation

Due to the ease of calculating SDs, there is a tendency to assign them undue importance without appreciating the breadth of the confidence limits.[16.24] In practice, confidence limits, testing accuracy and control all dictate that a simplified approach to SD values be adopted and that values be rounded off to 0,5 MPa to be in line with the reporting accuracy in strength tests.

The CUSUM method adopts a stochastic approach in monitoring SD.

Estimates of the actual value are derived from the range of successive pairs of results. Each individual estimate is subject to very wide confidence limits but the cumulative sum of differences between actual and target values illustrates trends and reveals whether the value(s) assumed in setting target design strengths must be revised. In this regard, it must be noted that the deviations from target in respect of SDs are generally smaller than those occurring in target mean strength and may therefore be overlooked.

While the use of CUSUM charts does give a good visual representation of changes in concrete quality, some care must be taken.[16.1] Using the V-masks simplifies decision-making, ie when action is required. Changes in mean strength are generally straightforward, but changes in SD are more difficult to detect and interpret as the standard V-masks tend to overestimate the SD by 1,5 MPa on average.

Three factors influencing SDs need to be discussed, bearing in mind that the values used primarily cover fluctuations from the combined influence of material, batching and testing variations.

Firstly, if an abrupt change in material quality occurs, the delay before corrective action takes place results in an increased SD over the period of transition. However, control standards may not have deteriorated, and this increased SD only reflects the need for mix adjustment.

The other two situations both concern massing of data. The CUSUM approach to SD assumes a constant target value across the full range of strength. This may not be strictly accurate and high-strength mixes in particular may have significantly higher SDs. This should be taken into consideration in setting target design strengths for these mixes and in massing results.

The other point requiring examination occurs if the possible deviation between target and actual w:c or cement content vs strength relationships referred to previously arises. The massing of paired strengths will then result in an over-estimate of SD.

16.4.6 Non-compliance

In the event of non-compliance with specification requirements for concrete strength occurring, detailed instructions for the necessary investigation and further testing of the structural elements in question are listed, principally in section 15 of SANS 10100-2:1992.[16.17] There is however one important omission: establish that the tests in question were not influenced by operator error or testing equipment malfunction. It is not generally appreciated that test results can be seriously affected, with losses of up to 50% in compressive strength arising from poorly executed testing. The most likely problem is eccentricity of loading: the failure cracking pattern of the test specimen should reveal that this occurred. A well-administered quality control programme should require that all test specimens representing failures are set aside for further examination.

In South Africa, laboratories accredited to ISO 17025: 2005[16.25] are required to participate in interlaboratory proficiency testing schemes. Such evaluations should provide evidence of a history of satisfactory ability to perform the compressive strength testing of concrete. Analysis of these data may also permit identification of assignable causes that might lead to unusual results.

If test results are reliable and a problem does exist, proceed to remedial action. This is firstly in respect of further production of concrete and secondly assessment of whether corrective measures are necessary for the affected structural members. It is important to note that the code does not automatically require the removal of any concrete of strength less than CS minus 3 MPa, but calls for analysis to determine whether any critical sections are suspect in respect of load-bearing capacity.

Adjustment of mixes

Section 5.1.2 of SANS 2001-CC1[16.20] lists three separate criteria for judging whether corrective action is necessary. Not only the mix but the whole production process must be reviewed. Corrective action must address the problem, not compensate for an error by other means. If on investigation, mix adjustments are deemed necessary, the problem is to assess suitable adjustments to mix proportions; such adjustments are not quantified in the single or triple test assessment. The need to modify a concrete mix is clearly evident if the average of all the test results of a specific grade at any stage exceeds the specified strength by more than 1,64 x SD.[16.20] The adjustments to the mixes are either in terms of changes in w:c or in kilograms of cement. Typically, a change of one MPa in average strength entails an adjustment of 0,03 to 0,04 in w:c or of 6 to 8 kg/m^3 in the cementitious content of a mix.

The problem is not clearly defined where a limited number of tests indicates that an adjustment to mixes is necessary to correct a sudden change in strength trends, when past performance of the mix has been satisfactory and an investigation has shown that batching is being correctly executed. If an increase in strength is indicated, a change to the next higher grade of concrete, normally 5 MPa more in CS, is a practical solution. With a decrease in strength, a more cautious approach is called for. There is a delay in obtaining results and if change in the quality of one or more of the constituents of the concrete has occurred, this may well have been recognised and corrected at source by the time that the concrete mix is being adjusted. Identification of the reason why the change was necessary and careful communication with suppliers are indicated.

Structural investigations

Two criteria are used to judge whether the concrete placed in a portion of a structure is acceptable. If any individual result is more than 3 MPa less than CS, or if the average of any three consecutive test results is less than CS, an investigation into those parts of the structure in which the concrete represented by the test results was used must be carried out.

It is really only as a last resort that demolition and replacement of structural concrete should be required. The various stages in sections 14.4 and 15 of SANS 10100-2 [16.17] are intended to ensure that every effort is made to avoid such counterproductive action.

The various stages of the investigation start with non-destructive testing, typically using the Schmidt hammer to establish, within the limitations of the equipment, whether the problem is general or occasional within the lot under investigation. If only a portion of the lot is suspected as being under strength from this investigation, the course of action, ie core testing, will probably depart from random testing as envisaged for the general control testing. Areas selected for coring will specifically be those where concrete is considered potentially deficient (see clause 14.4.2 of SANS 10100-2). In this case, interpretation of those core results would not fit in with the overall statistical analysis of concrete quality since the samples were selected and not random.

Another aspect of field samples, specifically cores, which renders correlation of strengths from different testing programmes difficult, is the question of curing. While the lower strengths accepted for field cores are intended to take field curing into consideration, lack of curing and protection in the more extreme situations may affect concrete to an even greater degree than allowed. The problem is then to resolve how much of the fault lies with the concrete mix and how much is a result of poor site practice. The former should not be adjusted to excessively conservative standards to compensate for the latter.

Further reading

It would appear that the logical starting point on a more comprehensive study of the application of statistical methods for the control of concrete quality would be ACI 214-77(1997) *Recommended practice for evaluation of strength test results of concrete.*

However this report still essentially reflects the early approach to this subject, ie that the performance of mixes is determined retrospectively for compliance assessment without adequate emphasis on ongoing production control. It is interesting to note that ACI 318-1999 *Building code requirements for reinforced concrete* and ACI 318R-99 *Commentary* offer a different approach with greater emphasis on mix adjustment as dictated by varying performance.

Nevertheless ACI 214 offers both interesting techniques for analysing data and a lengthy bibliography. One omission is that it does not consider the CUSUM method of analysis of data.

The following publications are recommended to the reader:

BS 2846**, Guide to statistical interpretation of data. Part 1: 1991, Routine analysis of quantitative data. Part 2: 1981 (1985), Estimation of the mean: confidence interval. Part 4: 1976 (1985), Techniques of estimation and tests relating to means and variances,** London: British Standards Institution.

BS 5700:1984 (1992), **Guide to process control using quality control chart methods and CUSUM techniques,** London: British Standards Institution, 1984 (1992).

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Chapter 17
High-performance concrete

Hans Beushausen and Frank Dehn

17.1 Introduction

High-performance concrete (HPC) is the result of performance improvements of conventional concretes, mainly based on empirical experience and research over the last two to three decades. A broad definition of "high-performance" concrete is that HPC achieves certain performance and uniformity criteria that cannot always be achieved routinely by using conventional materials and normal mixing, placing and curing practices.[17.1] These criteria may include improvements of fresh and hardened concrete properties such as self compactability, early-age and long-term mechanical properties, toughness, volume stability, durability, etc.

It could be argued that "high-performance" needs to be defined according to the specific requirements encountered for a specific application. However, as is the case with conventional, or "normal", concrete, compressive strength is commonly adopted for its classification. The terminology of high-strength concrete (HSC) is sometimes used instead of HPC. Strictly speaking, this is incorrect, as high-strength concrete, which is defined as concrete having a compressive strength higher than 60 MPa (Europe), or 40 MPa (USA),[17.2] can for certain applications be considered high-performance concrete, but high-performance concrete does not necessarily need to be of high strength. In this chapter HPC, which at the same time can be described as HSC, is discussed as this covers the most common applications.

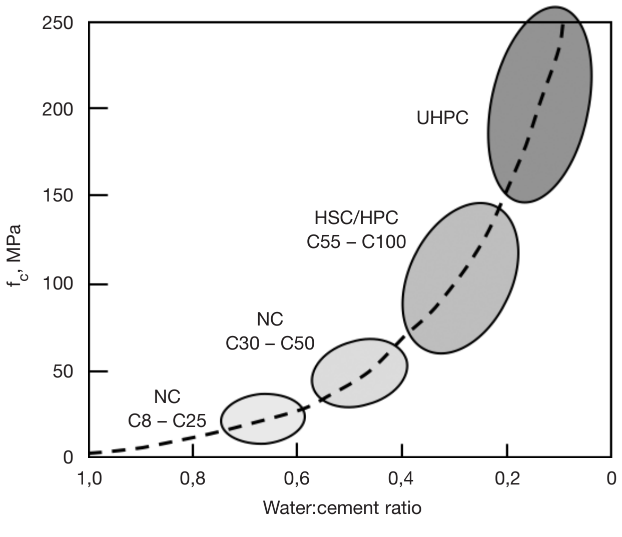

Figure 17.1: Rough classification of different concrete strength / performance classes in relation to w:c and compressive strength f_c[17.4]

NC = normal concrete
HSC/HPC = high-strength concrete / high-performance concrete
UHPC = ultra-high-performance concrete
C 5, etc = cylinder strength classes according to EN 206-1

Typical compressive strength of HPC ranges between 60 and 130 MPa (Figure 17.1). Through reduction of the maximum aggregate size to ≤ 1 000 µm and water:binder ratio to approximately 0,20, strengths significantly higher than 200 MPa may be achieved. Concrete of such high strength is commonly called ultra-high-performance concrete (UHPC), and is discussed in section 17.5.

The required workability properties of HPC can be controlled through the application of chemical admixtures and cement extenders, and through consideration of the cement paste content. Admixtures used for HPC include superplasticisers, accelerators, retarders, and air-entraining agents. Cement extenders improve concrete strength and durability through latent hydraulic and/or pozzolanic reactions and/or the fine filler effect. Common cement extenders used in producing HPC include silica fume (CSF), fly ash (FA), ground granulated blastfurnace slag (GGBS) and Metakaolin, their selection and proportioning depending on the fresh and hardened concrete properties required.

In order to achieve consistently high quality, manufacture, handling and casting procedures of HPC require special attention. Most important in this respect are careful selection of mix ingredients and the creation of optimum manufacturing conditions. Changes in material properties may have a significant influence on fresh and hardened concrete properties. Prior to selection of the mix ingredients, it must therefore be confirmed that material performance criteria are met.

Based on comparisons with conventional concrete, this chapter presents a brief overview of material selection, mix proportioning, typical fresh and hardened concrete properties, and aspects to consider during manufacture, handling and placing of HPC.

17.2 Constituent materials and mix design

17.2.1 Cementitious materials

In principle, all cements complying with SANS 50197-1 can be used to manufacture HPC. However, the low water:binder ratios required result in the interaction between cement clinker and superplasticiser being critical.[17.3] It should therefore be established that the chosen cement is compatible with the admixtures to be used and field or laboratory tests should be carried out to confirm this.

Depending on concrete performance requirements and environmental conditions, strength development characteristics of the cement as well temperature influences on the hydration reaction may also need to be considered. In view of quality assurance it is of major importance to establish limiting values for water demand, fineness and sulphate content for the cement to be used in producing HPC.

CEM I 42,5 R cement is especially suitable for manufacture of HPC, as it has desirable characteristics in terms of workability and strength development. The maximum achievable long-term strength for this type of cement is however limited and for strength classes above 105 MPa, CEM I 52,5 R has proven to give more satisfactory results. Here it must be considered that, because of its higher fineness, the use of CEM I 52,5 R results in a higher water demand and higher heat of hydration and may require a retarding agent to prevent rapid stiffening.

Most South African cements have a relatively low C_3A content. Experience in practice has shown that cements with low C_3A contents commonly yield good results in HPC production as they have a relatively low water demand and result in moderate temperature development during hydration. In addition, autogenous shrinkage can be decreased.

The binder content of HPC commonly ranges between 380 and 500 kg/m^3 (Figure 17.2). As a consequence, high temperatures and autogenous shrinkage during hydration may result in early-age shrinkage cracking if volume changes are restrained. For concrete production at high ambient temperatures or for concrete elements with large cross-sections, it may therefore be beneficial to use cement extenders such as FA or GGBS. Cement extenders are often included in HPC, utilising their positive influences on fresh and hardened concrete properties. For example, CSF is often used as a 1:1 substitution of about 8% of the CEM I to increase strength. Substitution of CEM I with other cement extenders, such as FA or GGBS, may result in loss of early strength but could improve fresh concrete properties such as workability and slump retention, and produce better long-term durability.

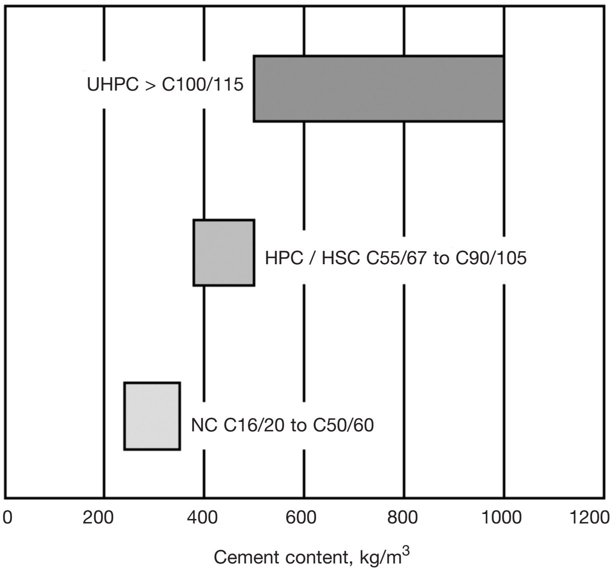

Figure 17.2: Typical ranges of cement content for different concrete strength classes (strength classes according to EN 206-1: C $f_{ck}/f_{ck,cube}$, f_{ck} = characteristic cylinder strength, $f_{ck,cube}$ = characteristic cube strength)[17.4]

17.2.2 Aggregates

With increasing concrete strength, mechanical properties of aggregates are of increasing importance. Further characteristics to consider include shape, grading and chemical reactions between aggregate and cement paste. Compared to aggregates used in conventional concrete, water demand needs to be checked and controlled more stringently for aggregates used in HPC. Owing to high concrete strength requirements, aggregates should have a water demand that is as low as possible, something that can be achieved through selection of rounded aggregates and optimised grading of different sizes (Figure 17.3).

The fines content in the sand (<0,150 mm) should be low, since the mix is rich in fines as a result of the high cement content. Crusher sand is commonly preferred for manufacture of HPC, both overseas[17.4] and in South Africa. For South African materials, preference should be given to crusher sand with a fineness modulus of 2,7 to 3,0.[17.3]

In order to achieve high packing density and avoid stress concentrations due to lack of homogeneity of the concrete matrix, the maximum size of coarse aggregate should generally be limited to 8 to 16 mm. In South Africa, however, good results have also been achieved with coarse aggregates between 19 and 26,5 mm.[17.5] Concrete members made from HPC are often densely reinforced, which supports the need for a limited maximum coarse aggregate size in such applications. Crushed coarse aggregates are generally preferred for the manufacture of HPC with design strengths of over 100 MPa. In South Africa, andesite, dolerite and

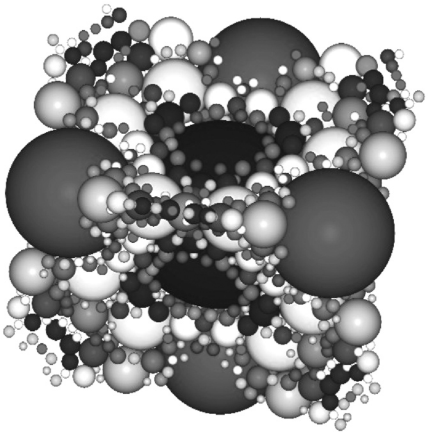

Figure 17.3: Optimisation of aggregate distribution (schematic)

dolomite have been identified as the most suitable types for both coarse and fine aggregate.[17.3]

17.2.3 Admixtures

Due to the required low water contents and low water:binder ratios, manufacture of HPC would not be possible without a high quality superplasticiser. Depending on specific project requirements, other common admixtures such as retarders or air-entraining agents may also need to be incorporated. Because of the importance of superplasticisers, these are discussed in some detail here, while information on other admixture types can be found in Chapter 5.

The plasticising effects of superplasticisers can mainly be ascribed to a combination of the following mechanisms: [17.4]

- Creation of electrostatic repulsion between particles, hence deflocculation and dispersion of fine particles, resulting in higher mobility of these constituents
- Reduction in the surface tension of the mixing water and hence better lubrication of fine and coarse mix ingredients
- Retardation of the hydration reaction on the surface of cement particles, resulting in an increase in available water for lubrication
- Change in the morphology of hydration products

Suitable superplasticisers for the manufacture of HPC are commonly based on the older sulphonated melamine formaldehyde (SMF) and sulphonated napthtalene formaldehyde (SNF), or on the newer polycarboxylate acid (AP) and polycarboxylate ester (PCE). In an experimental comparison, the new generation superplasticisers AP and PCE were found to have higher efficiency and longer slump retention time, compared to SMF and SNF, especially for low w:c ratio mortars or concretes.[17.6]

The compatibility of cement and admixtures cannot be evaluated based on chemical compositions of the materials alone, as other factors such as the water:binder ratio also influence the results. The optimum dosage of superplasticiser for a given workability therefore needs to be obtained from field or laboratory trials. Factors to consider are ambient temperature and required slump retention time. The workability of concretes with low water:binder ratios can only be improved to a certain limiting value. This limiting value is termed "saturation point", beyond which an increase in superplasticiser dosage will not increase workability any further. Addition of superplasticiser beyond the saturation point may cause segregation of the mix and should be avoided.

Typical superplasticiser dosages for the manufacture HPC are about 2 to 7% by mass of cementitious materials using older types of admixture and about 2 to 4% by mass of cementitious materials using PCE.

17.2.4 Mix design

Conventional concrete mix design procedures, such as the one developed by the C&CI (see Chapter 11), cannot account for low workability at low water:binder ratios and can therefore not simply be extrapolated for the design of HPC mixes. In the design of HPC with low water:binder ratios, workability is controlled by admixtures.

Mix design of HPC is commonly based on previously applied mixes and practical experience. In most cases, trial batches need to be made to find optimum mix proportions and water:binder ratios for the required performance. Typical mix proportions using South African materials were summarised by Addis, [17.5] who stated that for high-strength concrete (HSC), water:binder ratios are usually between 0,25 and 0,45. Stone content, for 19-mm stone with a relative density of 2,9, is usually between 1 050 and 1 200 kg/m^3. Water content in mixes containing superplasticiser normally ranges between 130 and 160 ℓ/m^3 for a slump of 75 mm.

17.3 Manufacture, casting and curing

17.3.1 Concrete manufacture

The most important issue during manufacture and handling of HPC is to achieve and retain sufficient workability until concrete placement. Batching accuracy and mixing intensity of the utilised manufacturing equipment are therefore important aspects to consider. Using modern equipment, a batching accuracy of ± 1% by mass can usually be achieved and should be aimed for when producing HPC.

Due to the viscous nature of fresh HPC, higher mixing intensities are commonly required, compared with manufacture of conventional concrete. It should however be considered that excessive mixing may result in faster stiffening of the fresh concrete and that HPC may develop higher temperatures during mixing, compared with concrete of normal strength.[17.7] Typical mixing sequences for the addition of mix constituents and the respective mixing durations, based on practical experience, are summarised in Table 17.1. Admixtures should preferably be added last.

If HPC is produced as readymix, the expected duration of transport and placement needs to be considered when designing for a particular setting time. In most cases, retarding admixtures are not required, since high dosages of superplasticisers also have a retarding effect. To achieve the required consistency it is often necessary to add further superplasticiser to the ready-mixed concrete just before casting. The quantity of extra admixture should however be kept to a minimum and it must be ensured that dispersion of admixtures is achieved through sufficient mixing time. A minimum additional mixing time of eight minutes in the readymix truck is, for example, recommended in German guidelines for high-strength concrete.[17.8]

Table 17.1: Typical mixing sequences for the addition of mix constituents and mixing durations[17.4]

	Method 1		Method 2	
	Material	**Duration, seconds**	**Material**	**Duration, seconds**
1	Aggregates and silica fume	30	Aggregates and cement	30
2	Cement and fly ash	30	Water	30
3	Water	60	Silica fume and fly ash	60
4	Admixtures	60	Admixtures	60

17.3.2 Placing of HPC

HPC can be placed using concrete pumps or by other conventional means. Due to the high viscosity of the mix, which is a result mainly of the use of CSF and high binder contents, greater dropping heights are commonly possible without segregation, and about 30 to 50% more compaction energy is required compared to conventional concrete. The high viscosity of HPC makes it more difficult to de-aerate the fresh concrete. The distance between poker vibrator insertion points should therefore be reduced to approximately half of the normal distance. If concrete is cast in layers, single layer thickness should not exceed 300 mm.[17.4]

Another unique feature of HPC is that surface finishing of freshly placed concrete is difficult due to the high viscosity of the mix. Once finished, it is therefore difficult to alter the surface any further. This may result in the need for trial installations of the particular mix when the concrete surface is designed to be directly exposed to traffic.

17.3.3 Protection and curing of HPC

Protection before finishing and curing after finishing are even more important for HPC than for conventional concrete, particularly during the early hours after placement. The high viscosity of the fresh concrete largely prevents bleeding in HPC. A thin and tough mortar layer can often be seen on the concrete surface, which needs to be prevented from drying out. Proper curing, preferably by keeping the concrete surface wet, reduces stresses resulting from restrained autogenous shrinkage and high hydration temperature development, both of which, due to the commonly high binder contents of HPC, are important aspects to consider.

Guse et al[17.9] investigated carbonation characteristics of HPC and found that the duration of curing has a significant influence on carbonation depths. For practical applications of HPC, König et al[17.4] concluded that the curing duration should be increased, compared to conventional concrete, and be no less than 72 hours.

HPC commonly contains insufficient mixing water to maintain the water-filled capillaries needed to sustain cement hydration and pozzolanic reactions. For this reason, Kovler[17.10, 17.11] concluded that methods based on internal water supply are more effective for concretes with very low water:binder ratios. A relatively new curing method that was developed for this reason, and which has successfully been implemented overseas, is internal curing using water-saturated porous aggregates or super-absorbent polymers. The principle of internal curing is to incorporate an internal water source in order to provide water necessary to replace that consumed by chemical shrinkage during hydration. According to Kovler,[17.10, 17.11] the amount of internal curing water needed to mitigate self-desiccation in modern HPC may be up to 25 kg/m^3 of concrete. Practical applications and research results on this relatively new curing technique appear promising for application in HPC production.

17.4 Properties of HPC

17.4.1 Microstructure

The differences between the mechanical and physical properties of HPC and conventional concrete are best understood by considering the differences in microstructure. The characteristics of conventional concrete are largely influenced by the relatively small stiffness of the cement paste and the interfacial transition zone (ITZ) between cement paste and aggregates. The latter is commonly of higher porosity and therefore of lower strength and stiffness and higher permeability compared to the remainder of the matrix.

The reduction of water:binder ratios and the use of cement extenders such as silica fume in HPC improve the properties of both cement paste matrix and the ITZ.

The use of cement extenders of high fineness together with superplasticisers, which help to evenly disperse fine particles in the mix, increases packing density and hence reduces pore volume and pore size in the cement paste. Of even greater importance for the properties of HPC is the improvement of the ITZ between coarse aggregate and cement paste, which also results mainly from reduction in the water:binder ratio and the use of cement extenders. In concretes without extenders, calcium hydroxide forms on the aggregate surface, resulting in a zone of weakness and high permeability. In the hydration reactions of cement extenders, the calcium hydroxide is largely consumed to create additional silicate phases. In HPC, the properties of the ITZ therefore hardly differ from those of the bulk cement paste, which improves the properties of the overall matrix significantly. As a result, fracture planes in HPC commonly do not occur along the ITZ, as is the case with conventional concrete, but rather through the aggregates, leaving a smoother fracture plane.

The improvement in microstructure influences all relevant characteristics of hardened HPC, including compressive strength, tensile strength, abrasion resistance, elastic modulus and durability.

17.4.2 Hardening process

Hardening cement needs about 40% of its own mass of water for complete hydration. HPC commonly has water: binder ratios of less than 0,4, which results in non-hydrated cement particles in the matrix. Such particles act as a fine filler and can improve strength and impermeability of the concrete. In conventional concrete, the hydration reaction is slowed down due mainly to the growing layer of hydration products on the cement particle surface, which makes it difficult for the water to diffuse into the cement particles for further chemical reactions. This is a gradually occurring process. In contrast, the low water content of HPC results in a rapid completion of the hydration reaction, which occurs, depending on the water:binder ratio, after approximately 20 to 30 hours. This process results in self-desiccation and causes rapid development of strength and elastic modulus.

17.4.3 Hydration heat development

The heat development resulting from cementing reactions in HPC cannot be related to the cement content in the same way as is commonly done with conventional concrete. The reason for this is the incomplete hydration of cement at water:binder ratios lower than 0,4. Self-desiccation stops the hydration reaction and thus the associated heat development. The total heat of hydration, ie the quantity of heat evolved during hydration (expressed in kJ/kg of binder), is therefore smaller in HPC, compared to concrete with conventional water:binder ratios. The rate of heat evolution in HPC is, however, significantly greater than conventional concrete, ie the initial temperature rise during hydration is higher, resulting in higher temperature gradients within a member. Therefore, the danger of crack development resulting from thermal stresses and associated durability problems, needs to be taken into account.

17.4.4 Compressive strength

In conventional concretes, the interfacial transition zone (ITZ) between aggregate and cement matrix is often the weak link in terms of compressive strength. Through reduction of the water:binder ratio to below 0,4 and addition of reactive cement extenders such as FA and CSF in HPC, the ITZ is strengthened to an extent where the aggregate strength may be the deciding factor for compressive strength. The selection of suitable aggregates with appropriate strength is therefore of great importance for the design of HPC.

As in conventional concrete, compressive strength of HPC depends primarily on the water:binder ratio. Since the cement particles have a very high fineness, incomplete hydration resulting from low water contents does not have a negative effect on compressive strength. The addition of CSF to the mix has a significant influence on the strength of HPC and practical experience in Europe indicates that compressive strengths consistently higher than approximately 90 MPa are difficult to achieve without addition of CSF.[17.4] The influence of CSF content on compressive strength was investigated by Bentur,[17.12] as shown in Figure 17.4.

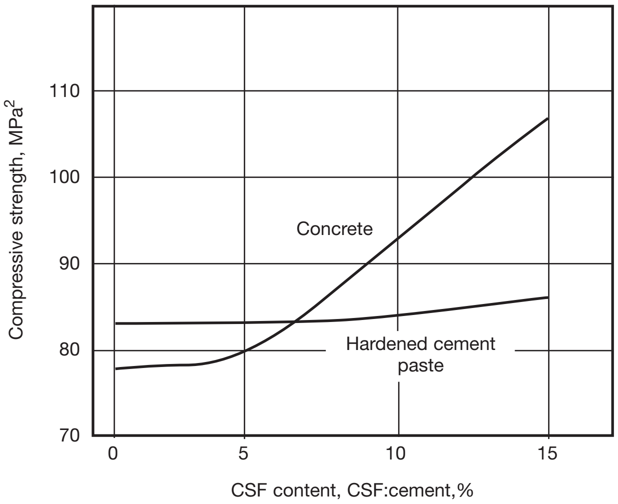

Figure 17.4: Influence of CSF content on compressive strength[17.12]

17.4.5 Strength development

The low water:binder ratios typical for HPC have a large influence on strength development at early ages. The lower the water:binder ratio, the higher the strength development in the first few days after casting. A rough guide for strength development of HPC in relation to cement type, for a curing temperature of 20°C is presented in Table 17.2.[17.4]

Table 17.2: Typical strength development $f_ct/f_{c,28d}$ of HPC in relation to cement type[17.4]

Cement type	Age, days				
	1	2	3	7	28
CEM I 52,5 R	0,8	0,9	0,9	1,0	1,0
CEM I 42,5 R	0,6	0,7	0,7	0,8	1,0
CEM I 32,5 R	0,5	0,6	0,7	0,8	1,0

17.4.6 Deformation characteristics

Shrinkage

Shrinkage characteristics of HPC differ from those of conventional concrete due to the different mechanisms involved. For conventional concrete with water:binder ratios > 0,4, drying shrinkage and carbonation shrinkage are the prevailing types of shrinkage. In HPC, drying shrinkage plays a less important role as it decreases with a decrease in water:binder ratio, which may be attributed to lower permeability. Similarly, capillary shrinkage (vs chemical shrinkage) and carbonation shrinkage are almost negligible in HPC. In contrast, autogenous shrinkage, which is small in normal-strength concrete, plays an important role in HPC and is often the predominant factor causing cracking at early ages.

Autogenous shrinkage develops immediately after setting due to internal consumption of water in hydration reactions, ie self-desiccation. In high-strength concretes, autogenous shrinkage may account for approximately 60% of total shrinkage. As opposed to drying shrinkage, autogenous shrinkage increases with a decrease in water:binder ratio. This is related to internal desiccation resulting from declining relative humidity in the concrete matrix. This is of particular concern in view of high hydration temperatures developing in the early hours of hardening. Both autogenous shrinkage and temperature gradients due to hydration heat development must be considered when estimating the risk of cracking at early ages. HPC mixes designed for members with high serviceability or durability requirements should therefore be tested for autogenous shrinkage characteristics prior to application. When testing HPC for shrinkage, it must be remembered that measurements that exclude the first 24 hours will underestimate the true shrinkage value. Additional information on shrinkage characteristics of HPC is presented in Chapter 8.

Creep

Changes in the pore structure, the increase in density, and the homogenisation of cement matrix and aggregate stiffness result in the somewhat different creep characteristics of HPC, compared to conventional concrete. In HPC, creep strains decrease with an increase in concrete strength and, compared to normal-strength concrete, the final creep strain is reached at earlier ages. The influence of member dimensions (cross-sectional area) on time-dependent deformations of HPC is typically smaller than experienced in conventional concrete, which may need to be considered in the estimation of strain values. For additional information on creep characteristics of HPC, see Chapter 8.

Instantaneous deformation under load

Generally, the elastic modulus of concrete shows an increase as strength increases. This increase is, however, not proportional and strength alone is not a good indicator of elastic properties of HPC. The amount and type of coarse aggregate used in the mix play a more important role in determining the elastic properties of HPC. Conventional methods for the estimation of the elastic modulus are often based on compressive strength and do therefore not necessarily apply to HPC. In line with rapid early-age strength development (compare Table 17.2), the elastic modulus develops rapidly within the first 24 hours, and thereafter only increases moderately.[17.13]

Typical stress-strain curves for concrete in relation to compressive strength are shown in Figure 17.5. The higher the compressive strength of the concrete, the smaller the non-linear portion in the stress-strain diagram. The reason for the more linear stress-strain curve of HPC is the higher homogeneity of the concrete matrix, compared to concrete of lower strength, resulting from similar stiffness of cement matrix and aggregates and the improved interfacial transition zone. The non-linear stress-strain behaviour of normal-strength concrete is a result mainly of micro-cracking at stresses above approximately 40% of ultimate strength. Cracking in concrete of normal strength usually starts at the interface between cement matrix and aggregates. In contrast, due to similar stiffness of cement matrix and aggregates, as well as a denser ITZ, the onset of micro-cracking in HPC is difficult to predict.[17.14]

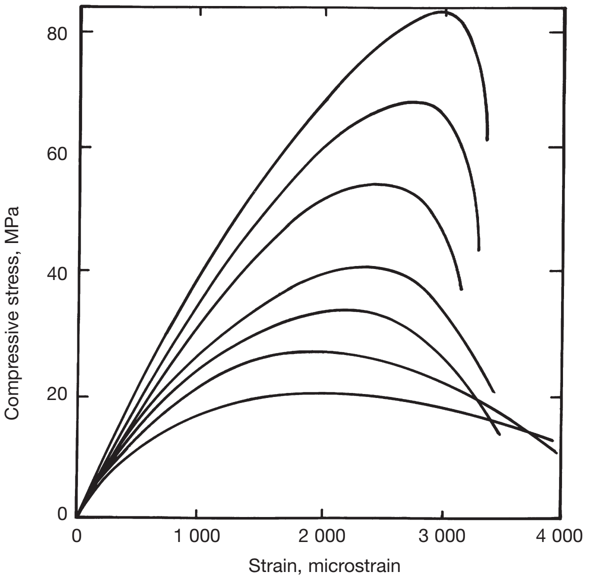

Figure 17.5: Typical compressive stress-strain curves for concrete[17.15]

HPC is more brittle than conventional concrete, ie there is a rapid drop of load after peak stress is reached, followed by sudden, often explosive failure, which needs to be considered in structural design. In some European codes (eg the German DIN 1045-1), this behaviour is considered by reducing the theoretical compressive failure strain to 2,2‰, compared to 3,0‰ commonly applied to conventional concrete.

17.4.7 Durability

Due to the optimised matrix, HPC has a significantly lower porosity than conventional concrete, and hence a lower permeability. The transport of aggressive agents, for example carbon dioxide and chlorides, into the concrete is therefore reduced, resulting in potentially higher durability of reinforced concrete. Owing to the denser microstructure, resistance to chemical attack, freeze-thaw cycles and abrasion is increased.[17.4]

The mechanisms controlling the durability of HPC are, however, somewhat different from those of conventional concrete, as the development of microcracks resulting from autogenous shrinkage can increase permeability and hence reduce durability of the concrete. Measures to minimise autogenous volume changes, for example proper curing procedures, are therefore essential in ensuring the durability of HPC.

Another issue that may need to be considered is that the fire resistance of HPC is generally lower than conventional concrete, as the dense microstructure hinders the release of vapour pressure that develops at high temperatures.

17.4.8 Costs

The primary reasons for choosing HPC over conventional concrete are to produce a more economical product, provide a feasible technical solution, or a combination of both. In comparison to normal-strength concrete, HPC commonly requires additional quantities of materials such as cement and admixtures, which results in higher costs per cubic metre. However, the cost of the finished product is the sum of various individual components, all of which need to be considered when assessing the economic viability of HPC. The use of HPC generally results in structural members such as columns or bridge beams of smaller cross-section, which results in lower concrete quantities needed and the consequent reduction in costs of formwork, transport and handling. In addition, reduced self-weight resulting from smaller cross-sections reduces the amount of reinforcement required in structural members. Smaller cross-sections of individual members can further aid in reducing the overall dimensions of the structure, which may result in additional financial benefits due to less space lost to the structure.

Of possibly even greater importance are long-term cost savings associated with the potentially higher durability of the product and the resultant reduction in maintenance costs. Due to the many influencing factors, the economic feasibility of HPC needs to be assessed for individual projects, taking all the above factors into account.

17.5 Ultra-high-performance concrete

Developments in concrete technology in recent years have resulted in practical manufacture and application of so-called ultra-high-performance concrete (UHPC), which has compressive strengths between 150 and 250 MPa. Typical applications are thin-walled elements such as shells[17.16] or concrete overlays on pavements and bridges for repair and strengthening purposes.[17.17] In addition to the rules that apply to the design and manufacture of HPC, design and manufacture of UHPC is based on the following measures:

- Reduction of the water:binder ratio to below 0,2
- Mandatory use of silica fume and superplasticisers
- Optimisation of packing density and grading of all solid particles (including binders)
- Reduction of maximum aggregate size to 2 mm (in most cases, 1 mm)
- Selection of high-strength aggregates

UHPC is a very brittle material and results in explosive failure when the applied stress exceeds the ultimate compressive strength. For this reason, and for an increase in tensile and flexural strength, metallic and/or synthetic fibres are commonly added to the mix.

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Chapter 18

High-density concrete

John Goodman

18.1 Introduction

Concrete with a density higher than the normal 2 300 to 2 550 kg/m^3 is used for special purposes such as radiation shielding, counterweights and ballast.

This chapter deals with choice of materials, manufacture of high-density concrete (HDC) and its physical properties.

18.2 Choice of materials

18.2.1 Aggregates

Higher concrete density is achieved by using high-density aggregates. Selection of an aggregate depends on its intended use, physical and chemical properties, availability and cost.

To produce concrete densities of 3 700 to 3 800 kg/m^3, the aggregate should have a particle relative density of at least 4,5. To produce concrete densities of around 4 800 kg/m^3, the aggregate should have a particle relative density of at least 6,0.[18.1]

Using natural aggregates, concrete densities of up to approximately 4 000 kg/m^3 are possible. Man-made materials, such as ferrosilicon slag or steel or lead shot, will probably be required to achieve higher densities.

For good workability, maximum density and economy, aggregates should be roughly cubical or rounded in shape, and free of flat or elongated particles.

In the case of radiation shielding, determination should be made of trace elements within the aggregate which may become reactive when subjected to radiation.[18.2] High-density aggregates with high fixed-water contents are often used if both gamma rays and neutron radiation are to be attenuated.[18.3]* ASTM C 637-98a[18.4] covers special aggregates for use in radiation-shielding concretes, in which composition or high particle relative density, or both, are of prime consideration.

It is desirable that the particle relative density of the fine aggregate be similar to that of the coarse aggregate. Use of a fine aggregate with a much lower density promotes settlement of the coarse aggregate through the mortar.

Aggregates should be supplied to consistent gradings.

Hydrogen evolution in HDC containing ferrophosphorous and ferrosilicon slags has been known to result in a reaction that produces large volumes of hydrogen. These slags should be used only after laboratory tests have shown the suitability of the materials.[18.1]

High-density aggregates, of which some have been used in South Africa for HDC, are discussed below. Physical properties of some aggregates, and densities of concretes made with them by the author, are summarised in Table 18.1.

Barytes

Barytes aggregate has been used in large quantities for HDC. With barytes, concrete densities of up to 3 500 kg/m^3 become possible but the material can have two disadvantages or potential sources of trouble. The ore is often associated with minerals containing zinc and lead, many of which can retard the setting of concrete, particularly if the lead is present as galena. The other drawback is that barytes is somewhat friable and particles can break down in the mixer. Overmixing should therefore be avoided and a careful check kept of the water content and workability. Concrete made using barytes was used for sinking a submarine pipeline near Durban.

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[*] For radiation shielding, the type and intensity of the radiation usually determine the requirements for density and water content of the concrete. The effectiveness of the shield against gamma rays is approximately proportional to the mass of the concrete – the higher the mass, the more effective the shield. An effective shield against neutron radiation requires both heavy and light elements. The hydrogen in water provides an effective light element in concrete shields.[18.3] Where high fixed-water content is desirable in an aggregate, suitable hydrous iron ore, serpentine or bauxite may be used.[18.1]

Ilmenite

This is available in quantity in South Africa in the form of fine sand near Richards Bay.

Haematite

Haematite has a strong red colour, which can make it unsuitable for some applications. Concrete with a density of 4 130 kg/m^3 was made in the laboratory by the author, using haematite for both coarse and fine aggregate.

Manganese slag

This aggregate has a particle relative density ranging between 2,7 and 3,2 and produces concrete with strengths and E values somewhat higher than normal.[18.5]

Chromite

Particle relative density ranges between 4,0 and 4,9. Available as a hard, lumpy ore which can be crushed to coarse or fine aggregate, chromite has been successfully used for making ballast blocks for domestic washing machines and in refractory bricks when used with calcium aluminate cement.

Magnetite

Magnetite is becoming more widely used as an aggregate. As with other iron ores, use of magnetite can result in complications with contaminants and with particle size since most producers are reluctant to supply the small volumes needed for concrete, preferring to concentrate on the huge quantities needed for steel production. Magnetite is used for HDC for radiation shielding, counterweights, hospital x-ray shielding, for coating pipes and for protecting bank vaults.

Galena

Galena is an ore of lead. It is available from specific mines near Aggeneys and Kuruman. Although galena is reported to retard setting, this was not experienced during testing at the Cement and Concrete Institute.

Steel punchings and shot

These materials are used when concrete densities higher than about 4 500 kg/m^3 are required.

Lead shot

Lead shot has been used as a high-density aggregate in the UK and is useful when concrete densities of circa 7000 kg / m^3 are needed. However, lead is soluble in alkalis and can form a gel. A full technical assessment for its use is recommended. The assessment should consider safety regulations for the use of lead, and the selection of a suitable cement. To avoid the highly alkaline conditions present in portland cement paste, lead shot has been used with high alumina cement (HAC) for the production of high-density concrete.[18.6]

18.2.2 Cement

Cements conforming to SANS 50197-1[18.7], which would be suitable for conventional concrete and produce the required physical properties, are generally suitable. Cement with a low alkali content or a suitable blend of CEM I cement and cement extender should be used when alkali-reactive constituents are present in the aggregates. A blended cement may be used to reduce temperature differentials and the possibility of associated cracking in the concrete. Ground granulated blastfurnace slag, fly ash and condensed silica fume have lower relative densities than portland cement and are used only if their inclusion does not reduce the density of the concrete below specified limits. With fly ash blends however, reductions in water content and, hence, small increases in concrete density have been obtained. HAC can be used if lead shot is used as aggregate.[18.6]

18.2.3 Admixtures

Conventionally placed HDC may contain some chemical admixtures.[18.8] It is advisable to establish by test, preferably under field conditions, that the admixture will achieve the required results without undesirable side effects.

Table 18.1: Physical properties of high-density aggregates and concrete

Aggregate	Size of aggregate particles	Aggregate particle relative density	Concrete density, kg /m^3
Magnetite	minus 1,18 mm	4,78	*
Haematite	minus 9,5 mm plus 9,5 mm, minus 37,5 mm	4,89 5,02	4 130
Ilmenite	minus 0,6 mm	4,44	*
Zircon	minus 0,3 mm	4,79	*
Barytes	plus 9,5 mm, minus 26,5 mm minus 4,75 mm	4,45 4,35	3 580
Galena	minus 19 mm	4,00	3 220
Chromite	minus 26,5 mm minus 1,18 mm	4,08 4,68	3 530

* Since coarse aggregate was not available in this material, concrete was not made.

Air-entraining agents help to control bleeding and settlement, improve workability and assist in obtaining more homogeneous concrete. However, entrained air will reduce the density of the concrete. Water-reducing admixtures will help to increase concrete density by reducing the amount of water, which is the ingredient with the lowest density. Shrinkage-reducing admixtures have been used for radiation-shielding concrete where dense, crack-free concrete is required. [18.9]

18.3 Mix proportions

Concrete of normal workability can be proportioned for densities as high as 5 600 kg/m^3 by using high-density aggregates such as iron ore, iron or steel shot, barytes, and iron or steel punchings.[18.2] Because of the unusual nature of the concrete, actual mix proportions should be determined by proper concrete mix design (see Chapter 11 and reference 18.2).

The concrete mix should be proportioned to provide the desired compressive strength and density, and have adequate workability. The chemical constituents and fixed water content of the resulting mix must, if relevant, provide satisfactory shielding properties.[18.8]

If the concrete in service is to be exposed to a dry environment, it should be proportioned so that the required density is achieved in a dried state.

Settlement in the plastic mix can generally be minimised by proper proportioning and incorporation of suitable chemical admixtures.[18.10]

18.4 Manufacture of concrete

18.4.1 Storage and handling of materials

The particles of some coarse aggregates, iron ore in particular, may break down during handling. Should this happen, they should be rescreened just before batching or an allowance made for the increased fines. A soft, powdery aggregate surface has been known to affect workability. Aggregates with such surfaces should be washed before use.

Aggregate should be shipped, handled and stored in a manner that will prevent loss of fines, contamination by foreign material and significant aggregate breakage or segregation.[18.8]

Aggregates may be shipped in heavy wooden boxes, watertight bags, steel containers or in bulk by railroad cars or trucks. Stored aggregates should be protected from moisture. The mass of aggregate in each container should be clearly marked on the container. [18.8]

Batching and mixing equipment

Batching equipment should be sufficiently accurate to provide consistent batches of concrete.

Standard mixing and handling equipment is used for HDC but care should be taken to avoid overloading.[18.8]

Equipment should be cleaned before use to minimise contamination.

18.4.2 Transporting and placing

Transporting HDC without agitation tends to cause excessive consolidation or packing.[18.8] If HDC mixtures are proportioned properly, they are pumpable and will pump better at a lower slump than normal-density concrete.

Conventional placement methods may be used for HDC, provided that the mixture is workable and the forms are relatively free of embedded items. Such concrete, however, presents special problems due to the tendency of the high-density aggregate particles to segregate. Segregation is greatest where the aggregates are not uniform in grading or density, the mixture contains excessive water, or the slump is excessive. Concrete slump should generally be between 40 and 75 mm for high-density aggregate mixtures.[18.10]

Placement of conventionally mixed HDC is subject to the same considerations of quality control as normal concrete, except that it is far more susceptible to variations in quality due to improper handling. HDC is particularly subject to segregation which results in variations of strength and density.

HDC usually will not "flow" in a form and must be placed in each discrete area and compacted in place with a minimum of vibration. Under no circumstances should an attempt be made to move HDC with vibration equipment.[18.8] Concrete should be placed in layers not more than 300 mm thick. If an excessive amount of grout builds up on the surface, it should be removed while the concrete is still plastic. [18.8]

When segregation cannot be avoided or when embedded items or restrictions prohibit conventional placement, the preplaced aggregate method may be used.[18.10]

Preplaced aggregate construction consists of erecting formwork, preplacing coarse aggregate in the forms, and mixing and placing high-density grout using the same procedures as those employed with normal grout.[18.8]

Postplaced aggregate is a rarely used technique in which high-density grout, up to 300 mm deep, is placed in the form; high-density coarse aggregate is then placed in the grout and worked into place by rodding. Internal vibration should be avoided, especially where the grout contains high density fine aggregates.[18.10]

Compaction

Because of the purposes for which HDC is required, it is essential to obtain maximum density and freedom from segregation and voids. Compaction therefore requires extra care, particularly as it is more difficult than with normal concrete. Internal vibration is often supplemented with external vibration, but extra care should be taken when the high-density aggregates are friable and easily broken down.[18.10] Poker vibrators must be inserted at closer intervals and greater care must be exercised to ensure that all the concrete is properly compacted.

Vibrators operating at normal frequencies are usually satisfactory. However, by using higher frequencies (180 Hz) for shorter periods, the danger of segregation is reduced. In all cases over-vibration must be avoided as this results in segregation of the high-density aggregate by settlement.[18.10]

Formwork

Pressures exerted on formwork by HDC are greater than those exerted by normal concrete and formwork must be designed accordingly. Pressure on vertical forms can be reduced by placing concrete in slowly rising lifts.

18.4.3 Testing and control

Because of the need to satisfy special requirements, testing and quality control are extremely important. In many cases, testing of the structure, including the removal of cores, is not permissible. Care must be taken to ensure that good concreting techniques are used.

Quality control tests on freshly mixed concrete could include tests of unit mass, temperature, slump and air content. Acceptance criteria should be specified for important properties. Where necessary, the quality of the concrete produced should be controlled by an established program of sampling and testing.

18.5 Physical properties

Except for unit mass, the physical properties of HDC are generally similar to those of normal concrete. The relationship between strength and water:cement ratio is generally similar, although strengths may be higher than normal.

If any specific properties (eg modulus of elasticity, strength, shrinkage, coefficient of thermal expansion, thermal conductivity, etc) are particularly important, it is advisable to conduct tests with the proposed materials and mix proportions.

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Chapter 19

Low-density concrete

Elsabé Kearsley

19.1 Introduction

Low-density concrete (LDC) is usually defined as concrete having an air-dry density of below 2 000 kg/m^3 as opposed to normal concrete with a density in the region of 2 350 kg/m^3. LDC has practical and economical advantages as the reduced mass of the superstructure can lead to savings in the cost of super- and substructures and components can be handled and erected relatively easily. LDC is not however cheaper than conventional concrete and as own weight results in passive resistance, special care has to be taken during the design of these structures. Fire resistance and thermal insulation of LDC are superior to those of normal density concrete. The thermal efficiency of LDC has made it a material used extensively in countries with cold winters. Although LDC has not been widely used in South Africa, the current energy shortage has emphasised the need for energy efficient building materials, which could and should result in an increase in the use of LDC.

LDC can be manufactured with densities between 300 and 1 850 kg/m^3. The strength of LDC is a function of density and therefore density can be used to classify LDC according to the purpose for which it is to be used. LDC can be classified as one of the following:

- Structural LDC has a density lying between 1 350 and 1 850 kg/m^3 and a minimum compressive strength of 20 MPa.
- Insulating LDC has a density of between 300 and 800 kg/m^3 and is used for non-structural applications, mainly for thermal insulation purposes.
- A third category, falling between the previous two, is used for masonry units.

Replacing some of the solid material in the mixture with air voids can also reduce the density of concrete. These voids can be located in one of the following three places:

- In the aggregate particles; known as low-density-aggregate concrete.
- In the cement paste; known as cellular or aerated concrete.
- Between coarse aggregate particles (by omitting fine aggregate); known as no-fines concrete.

In this chapter the first two types of LDC will be discussed, while no-fines concrete is covered in Chapter 25.

19.2 Low-density aggregates

When low-density aggregate is used in concrete it is important to not only take the local availability of the aggregate types into account but also to consider the decreased density and the increased water absorption of the lightweight aggregates. Table 19.1 gives a list of low-density aggregates commonly used in low-density-aggregate concrete (LDAC) with their respective air-dry loose bulk densities. Each aggregate is discussed below. (Aggregates dealt with in sections 19.2.2 to 19.2.4 are not currently manufactured in South Africa.)

Table 19.1: Air-dry loose bulk densities of low-density aggregates

Aggregate type	Loose bulk density, kg/m^3
Furnace clinker and ash	720 - 1 040
Expanded slag	700 - 970
Expanded clays and shales	320 - 960
Sintered fly ash	770 - 1 040
Exfoliated vermiculite	60 - 160
Expanded perlite	50 - 240
Wood particles	320 - 480
Plastic particles	10 - 20
Pumice	480 - 880
Diatomite	450 - 800

Figure 19.1 gives an indication of the density ranges where different types of low-density aggregates are generally used. All natural or manufactured aggregates have the potential to absorb water at a rate which diminishes with time and this absorption affects the workability and other properties of the concrete. If dry, typical natural aggregates absorb about 0,5 to 2% of the dry mass within 24 hours in water. Low-density aggregates typically absorb between 5 and 15%. This high water absorption results in a loss in workability, and the water and cement contents of low-density-aggregate concretes are usually higher than those of concretes of normal weight. The water content should be limited by using chemical admixtures.

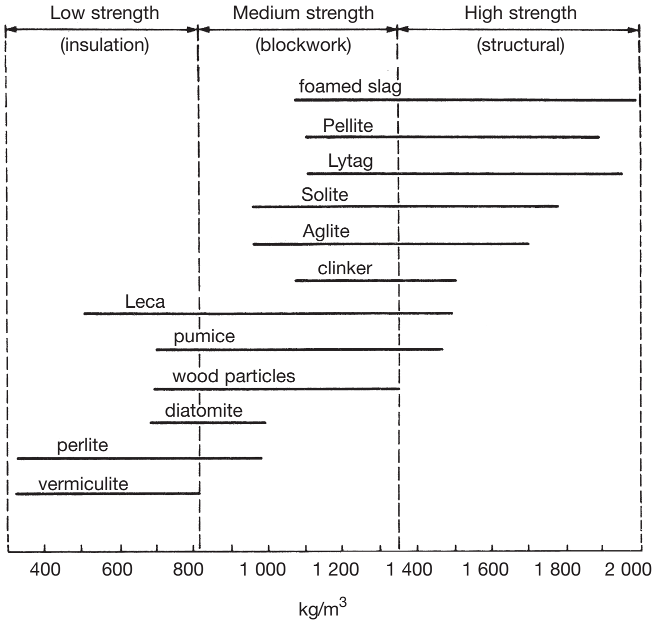

Figure 19.1: Typical range of nominal air-dry densities for various LDACs[19.1]

19.2.1 Furnace clinker and ash

Furnace clinker

This type of aggregate gives good results if the clinker is obtained from high-temperature furnaces where the amount of combustible material in the residual clinker is reduced to a minimum. Clinker that has a high residue of unburnt or partly burnt coal (more than about 5%) will produce an unsound aggregate which, in turn, will lead to the production of low-strength concrete with high shrinkage. Because clinker may contain oxides or sulphur and other compounds that can cause corrosion of steel, clinker aggregate should not be used in reinforced concrete or in concrete used as a cover to steelwork.

Clinker aggregate should comply with the requirements of SANS 794.[19.2]

Furnace bottom ash

Modern electric power stations use pulverised coal in furnaces. The resultant ash is collected as fly ash from the flue gases (see Chapter 1) and as furnace bottom ash.

Furnace bottom ash is variable in chemical composition and quality and does not fall within the SANS 794[19.2] standard definition of clinker, which is "well-burnt furnace residues which have been fused or sintered into lumps." However, it is possible, after crushing and screening, to use the better grades as low-density aggregate for masonry units.

19.2.2 Expanded slag

Foamed slag

Molten slag from blastfurnaces is treated in the molten state (at approximately 1 500°C) with a controlled amount of water (or, in some industrial processes, by steam and compressed air) so that steam is trapped in the molten mass to give the slag a porous structure similar to that of natural pumice. This is called foamed slag. Foamed slag is relatively inert, and has a good reputation for reliability.

Pelletised slag

Pelletised expanded slag, in which molten slag particles are bloated with water in a rotating drum and then rounded by being thrown through air, is lighter than foamed slag and the particles have smoother surfaces.

19.2.3 Expanded clay and shale

When certain types of clay and shale are heated to about 1 200°C they reach a point of fusion, and gases generated in the mass rapidly expand to form a honeycomb of small cells separated by walls of vitrified material. Three types of aggregate have been produced in the UK from these raw materials:

Aglite is produced by mixing a blend of clay and shale with ground coke and passing the mixture over a sinter-strand hearth and then through crushers to give the particles an angular shape.

Leca is produced from a special grade of clay, suitable for bloating, which is ground and pre-treated before passing through a rotary kiln; this patented process forms a range of smooth spherical pellets with a glazed but porous skin.

Sintag is produced by crushing raw shale with a controlled combustible content and sintering it at about 1 200°C.

19.2.4 Sintered fly ash

Ash collected from the flue gases discharged from modern power stations burning pulverised fuel is known as pulverised fuel ash or fly ash (FA) and consists of minute spherical glass particles of which the fineness is similar to that of cement. Low-density aggregate is produced by dampening the FA with water and mixing it with coal slurry. The material is then fed onto rotating pans (known as pelletisers) to form spherical pellets which are then sintered at a temperature of 1 200°C. This causes the ash particles to coalesce, without fully melting, to form an aggregate known as Lytag.

19.2.5 Exfoliated vermiculite

Vermiculite is a mineral of laminar formation similar in appearance to mica but, unlike mica, it expands (exfoliates) rapidly when heated, thereby radically reducing its density. The raw ore is dried, crushed and graded for size. The grading process is carried out by winnowing in a stream of air. The individual grades of material are then rapidly passed through hot burners (about 1 000ºC); this causes the vermiculite to exfoliate.

19.2.6 Expanded perlite

Perlite is glassy volcanic rock containing water. The industrial process of expanding the rock to form a low-density aggregate consists of crushing the material to graded sizes and rapidly heating it to the point of incipient fusion (about 1 800ºC). At this temperature, the water dissociates itself and expands the glass into a balloon-like formation of small bubbles to produce a cellular material with a correspondingly low density.

19.2.7 Wood particles

Graded wood particles have been used as an aggregate but some method of pre-treatment of the material is usually necessary, otherwise the tannins, soluble carbohydrates, waxes and resins which wood contains may affect the hydration of the cement. Various patent processes are available, but with most softwoods it is necessary merely to mix in calcium chloride or lime. Another treatment is to boil the particles in water to which ferrous sulphate has been added.

19.2.8 Plastic particles

Low-density plastic particles, such as expanded polystyrene, are made of beads that contain an expanding agent; the beads are softened in steam heat and the agent forces them to expand to about four times their original diameter. The expanded beads are then, in some processes, coated with resin and cement.

The maximum size of expanded polystyrene beads is about 4 mm.

19.2.9 Pumice and diatomite

Some aggregates, usually of volcanic origin, are both light and strong enough to be used as found. Pumice has a cellular structure owing to gases being present in the molten lava at the time of volcanic eruption.

Diatomite is a calcined diatomaceous rock formed from minute fossil skeletons of marine life.

19.3 Low-density-aggregate concrete

19.3.1 Materials

LDAC is a hardened mixture of cementitious material, coarse and fine aggregate, and water. Cementitious materials must comply with SANS 50197-1[19.3] and SANS 1491.[19.4] The coarse aggregate may be any of the low-density aggregates already described and nominal particle size will normally be between 19 and 5 mm. The fine aggregate may consist of natural sand or any of the low-density aggregates crushed to a grading of 5 mm down.

Graded wood particles, when used as an aggregate in concrete blocks, are mixed with sand, cement and water. The sand may be omitted from the mix, in which case the resulting material has a much higher shrinkage value.

Methods of mix design for LDAC are in most cases similar to those for concretes using dense aggregates, but care must be taken in allowing for the individual characteristics of each type of aggregate.

19.3.2 Production and manufacture

Production of LDAC is carried out in standard batching plants and concrete mixers, as for normal-density concrete. Mixing instructions should be adhered to with special care; manufacturers of low-density aggregates will generally give specific advice. In some cases, to obtain a more consistent mix, it may be advisable to pre-wet the coarse aggregate to reduce the amount of water absorbed by the aggregate during mixing.

In view of the harsher characteristics of some of the low-density aggregates, and their low bulk densities, there is a tendency for the coarse aggregate to separate from the mortar, especially in the higher workability range. This is due to buoyancy of the coarse aggregate, and it is therefore essential to ensure that the mix has been designed to accommodate higher workabilities. Air entrainment (4 to 6%) will improve the workability of the mix and, by reducing the density of the mortar, will reduce the likelihood of flotation of the coarse aggregate. In addition, air entrainment will enable the water content of the mix to be reduced, thus increasing the potential durability of the concrete as well as reducing shrinkage and creep. Air entrainment is not, however, necessary with all mixes and the advice of the aggregate manufacturer should be sought. Also, pumping admixtures may be used for reducing segregation or flotation.

Internal vibration, with a poker vibrator of high frequency and low amplitude, should be used, but the concrete should be vibrated only until it is compacted. Because of the low density of the fresh LDAC, it should be placed continuously onto the vibrator: large volumes of concrete should not be put into position and then vibrated; this causes blowholes because entrapped air is not expelled by the weight of the concrete.

19.3.3 Properties

Compressive strength

The strength of LDAC is not only a function of the water:cement ratio but also a function of the void content and the strength of the aggregate used. Typical strengths for concrete made with different types of light weight aggregates can be seen in Figure 19.2.

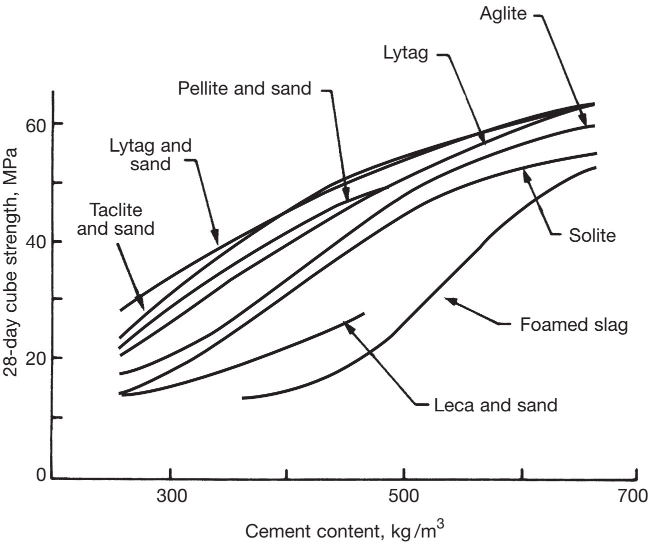

Figure 19.2: Typical relation between 28-day cube strength and cement content for LDAC[19.1]

Deformation

The moduli of elasticity of LDAs are lower than most natural rocks. This fact, together with the need for higher cement contents for a given strength, results in greater overall deformation of LDAC, compared with conventional concrete. Reference 19.1 discusses, in some detail, modulus of elasticity, creep and shrinkage.

Durability

The durability of LDAC is characterised by the following:[19.1]

- Because inorganic LDAs are sound and stable having been subjected to temperatures of the order of 1 100 to 1 200ºC, they do not react detrimentally with water nor with water in combination with alkalis from the cement.
- The relatively high cement contents and low water: cement ratios used should be advantageous in resisting inflow of aggressive agents.
- Abrasion resistance tends to be good initially because of the rich mortar in LDAC, but once the surface is lost the concrete will tend to abrade rapidly.
- LDAC is not necessarily more permeable than conventional concrete.
- Provided coarse aggregate particles are properly dispersed in the concrete, the resistance to carbonation of LDAC is not inferior to that of conventional concrete.
- Corrosion protection of steel reinforcement is good, provided that cementitious content is at least 300 kg/m^3.

Typical proportions and properties of insulating and structural LDACs are shown in Tables 19.3 and 19.4 respectively.

Table 19.3: Typical properties of insulating low-density-aggregate concrete

Aggregate type	Cement: aggregate ratio by volume	Air-dry density, kg/m^3	28-day cube strength, MPa	Thermal conductivity,* W/m.K	Drying shrinkage, Microstrain
Vermiculite	1:8 1:6 1:4	400 480 560	0,7 0,9 1,2	0,1 - 0,2	3 500 - 4 500
Perlite	1:7 1:6 1:5 1:4	400 480 560 640	1,4 2,1 3,4 4,7	0,1 - 0,2	1 400 - 2 000
Graded wood particles	1:4 1:3 1:2	640 880 1 200	1,7 4,8 12,1	0,2 - 0,3	2 500 - 5 000
* For guidance only					

Table 19.4: Typical properties of structural low-density-aggregate concrete

Aggregate type	Quantities per cubic metre of concrete				Air-dry density, kg / m^3	28-day cube strength, MPa	Modulus of elasticity E, GPa	Thermal conductivity,* W/m.K
	Cement, kg	Aggregate in loose state						
		Fine, m^3	Medium, m^3	Coarse, m^3				
Leca	310	0,65	-	0,65	1 050	8,5	3,5 - 10,5	0,3 - 0,6
	350	0,60	-	0,60	1 200	10,5		
	440	0,60	0,60	-	1 300	14,0		
	350	0,40 **	-	0,80	1 350	15,5		
	330	0,45 **	0,80	-	1 500	17,0		
Aglite	390	0,75	0,65	-	1 500	21,0	14 - 18	0,6
	440	0,75	0,70	-	1 550	26,0		
	480	0,70	0,70	-	1 600	31,5		
Lytag	320	0,55	-	0,70	1 550	22,5	12 - 18	0,5
	400	0,50	-	0,80	1 600	30,0		
	500	0,40	-	0,80	1 600	37,5		
	550	0,35	-	0,80	1 650	45,0		
Foamed slag	480	0,85	-	1,10	1 900	21,0	18 - 24	0,9
	540	0,85	-	1,10	2 000	27,5		
	570	0,90	-	0,85	2 000	34,5		

* For guidance only

** Natural sand fine aggregate

It is advisable, however, to contact the individual producers of low-density aggregates to obtain their advice on mix designs for specific purposes.

Although Perlite is not listed in Table 19.4, it is possible to manufacture structural LDAC with a compressive strength in excess of 20 MPa at a dry density in the region of 1 100 kg/m^3 (wet density 1 400 kg/m^3) by using a cement: Perlite volume ratio of 1:3 and a low water content.

Table 19.5: Typical properties of AAC[19.5]

Dry density, kg/m^3	Compressive strength, MPa	Elastic modulus, GPa	Thermal conductivity, W/m.K
450	3,2	1,6	0,12
525	4,0	2,0	0,14
600	4,5	2,4	0,16
675	6,3	2,5	0,18
750	7,5	2,7	0,20

19.4 Cellular or aerated concrete

Cellular concrete is produced by introducing voids into the plastic mortar mixture, resulting in a material with a cellular structure, containing voids with a diameter of between 0,1 and 1 mm. As no coarse aggregate is used in cellular concrete, the term concrete is, strictly speaking, inappropriate.

The voids in cellular or aerated concrete can be produced by gas generated during a chemical reaction, normally caused by the addition of aluminium powder (gas concrete) or by the introduction of air. Cellular concrete produced by the introduction of a foaming agent is called foamed concrete.

19.4.1 Gas concrete

Gas concrete should be produced in a well-controlled factory environment. When aluminium powder (approximately 0,2% of the cement content) is added to cement paste or mortar a chemical reaction takes place, resulting in the release of hydrogen gas. The gas forms bubbles in the mixture thus expanding the mixture to fill the moulds at the required density. The aerated concrete is then cured in steam at atmospheric pressure. Units can be cut to exact sizes directly after curing. Autoclaved aerated concrete (AAC) is produced by curing the aerated concrete in an autoclave under high pressure in steam at about 180ºC. Autoclaved aerated concrete has better strength, volume stability and durability than non-autoclaved aerated concrete. Typical properties of AAC are listed in Table 19.5.

19.4.2 Foamed concrete

The Dutch research centre, Civieltechnisch Centrum Uitvoering Research en Regelgeving (C.U.R.), Recommendation 14,[19.6] defines foamed concrete as a cementitious material, with a minimum of 20% (per volume) foam entrained into the plastic mortar.

Foamed concrete is produced by entrapping numerous small bubbles of air in cement paste or mortar. Mechanical foaming can take place in two principal ways:

- By pre-foaming a suitable foaming agent with water and then combining the foam with the paste or mortar.
- By adding a quantity of foaming agent to the slurry and whisking the mixture into a stable mass with the required density.

The most commonly used foam concentrates are based on protein hydrolyzates or synthetic surfactants. They are formulated to produce air bubbles that are stable and able to resist the physical and chemical forces imposed during mixing, placing and hardening.

Foamed concrete is the most popular of all LDCs in developing countries. The reasons for this are the low capital expenditure on equipment and the ready availability of the principal materials. The material can be produced on a small scale, even at site level, and it is relatively easy to place and finish without heavy or expensive equipment. Foamed concrete is self-compacting, free flowing and pumpable and therefore easy to place in inaccessible places. It has good thermal and acoustic properties and is also frost resistant. It is however too weak for direct exposure to traffic and hail and should be protected by a wearing layer (asphalt can be used for roads).

Since the introduction of cellular concrete systems to the construction industry over 50 years ago, the use of foamed concrete has been almost exclusively limited to non- structural void filling, thermal insulation, acoustic damping, trench filling for reinstatement of roads and for building blocks. In the Netherlands foamed concrete has been used not only for level corrections in housing developments, but also as a fill material where ground subsidence has taken place and as a founding layer for road works on very weak soils.

Historically foamed concrete has been perceived to be weak, non-durable with high shrinkage characteristics. Unstable foams have in the past resulted in foamed concrete having properties unsuitable for reinforced, structural applications. Unprotected reinforcement in aerated concrete where the voids are interconnected, would be vulnerable to corrosion even when the external attack is not very severe. It is therefore important to ensure that the air entrained into the foamed concrete is contained in stable bubbles that remain intact and isolated, and do not thus increase the permeability of the cement paste between the voids.

The development of protein-hydrolisation based foaming agents and specialised foam generating equipment has improved the stability of the foam, making it possible to manufacture foamed concrete for structural applications. In recent years foamed concrete has been used as a structural material in schools, apartments and housing developments in countries such as Brazil, Singapore, Kuwait, Nigeria, Botswana, Mexico, Indonesia, Libya, Saudi Arabia, Algeria, Iraq and Egypt.

Typical properties of foamed concrete are listed in Table 19.6. It is standard practice to classify foamed concrete according to the dry density thereof as indicated in Table 19.6. This density is determined from oven-dried specimens and the actual density of foamed concrete would usually be higher than this density as there would generally be evaporable water present in foamed concrete. The presence of water in foamed concrete elements also results in an increase in thermal conductivity.

Table 19.6: Typical properties of foamed concrete

Dry density, kg/m^3	Compressive strength, MPa	Elastic modulus, GPa	Thermal conductivity, W/m.K
400	1	1	0,1
600	1,5	1,5	0,11
800	2	2,5	0,2
1000	5	3	0,3
1200	10	4	0,4
1400	15	6	0,55
1600	25	12	0,65

19.5 Durability of low-density concrete

Although the properties of LDC as measured in laboratories indicate that well-manufactured LDC is as least as durable as normal-density concrete, the performance of the material can only be proven with full-scale structures under service conditions in the field. Over the last 50 years various low-density structures have been constructed throughout the world and current indications are that appropriate limits on water and cement contents (similar to those used for normal concrete) will ensure that LDC is no less durable than normal concrete.[19.5]

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Chapter 20

Self-compacting concrete

Hans Beushausen

20.1 Introduction

Self-compacting concrete (SCC) is defined as concrete that can fill formwork and encapsulate reinforcing bars through the action of gravity only, while maintaining homogeneity.[20.1] SCC therefore achieves full compaction without externally or internally applied vibration energy and de-aerates by itself.

SCC is characterised by its excellent flow properties combined with a high resistance to segregation. Mix constituents are generally the same as those used for conventional concrete, but using different proportions of the individual components. Compared to conventional concrete, the quantity of coarse aggregate is reduced and a higher dosage of superplasticiser is added to the mix. Sufficient segregation resistance and stability is achieved by using a high fines content, sometimes in combination with a viscosity-modifying admixture. Mix design procedures for SCC differ from those applied to conventional concrete and rely to a large degree on experimental testing of trial mixes to identify optimum mix ratios for the specific materials used.

Applications of SCC include any type of construction and building member commonly cast from conventional concrete. In general, SCC has similar properties to conventional concrete, the main difference lying in the workability properties in the fresh state. Special experimental methods are applied to test and verify the workability properties of SCC.

The main construction objectives of SCC can be summarised as improved productivity, improved working environment, and improved homogeneity.[20.1] Improved productivity results from complete elimination of compacting work, which reduces the cost of on-site labour and increases the speed of concrete placement, which in turn may shorten the total time of construction. Further improvements of productivity are related to the flowing consistency of SCC, which allows time-efficient concrete handling and placement using modern formwork filling and pumping procedures. The improved working environment is associated with the elimination of hand-held vibrators, the reduction of noise, and the cutback on equipment lying around, which otherwise may present tripping hazards and hinder construction.[20.2] Improved homogeneity results from the elimination of poor workmanship during compaction as SCC achieves full compaction even in members with congested reinforcement, without relying on a skilled workforce for proper vibration. Improved homogeneity in turn enhances quality properties, which was the original reason for developing SCC.[20.3]

SCC allows concrete to be cast in confined spaces where the compaction of conventional mixes may be difficult due to access limitations for vibrators. Further advantages of SCC often noted in the literature include improved concrete aesthetics for architectural applications and cost savings.[20.2] Economic factors need to be evaluated taking all relevant aspects into account. The cost per unit volume of delivered SCC is generally higher than conventional concrete, and due to near-hydraulic formwork pressures resulting from the high fluidity, stronger formwork may be required for certain applications. These factors need to be weighed against savings in time, labour and equipment to assess the economic viability of SCC for a particular project.

SCC was developed in Japan in the 1980’s and, over the past decade, has increasingly been utilised by the international concrete industry, mainly in North America, Europe and Japan. Owing to its increased popularity, SCC has been the subject of numerous research projects, which has led to extensive documentation of its properties in the fresh and hardened states, as well as production-related issues. Summaries and state-of-the-art reports concerning design, manufacture, properties and application of SCC have been published to promote the use of SCC and are generally useful sources of information on this relatively new technology.[20.1, 20.2, 20.4, 20.5] A list of references for further reading is provided at the end of this chapter.

20.2 Fresh concrete properties

20.2.1 Rheology

Rheological parameters used to characterise SCC are flow behaviour and plastic viscosity. The flow behaviour of fresh concrete is mainly influenced by the properties of its cement paste and mortar. For the development and optimisation of SCC, an understanding of the rheology of the different fluid phases of fresh concrete, ie paste, mortar (paste and sand) and concrete (mortar and coarse aggregate) is essential.

The rheological behaviour of fresh concrete is described by means of the flow curve, which defines yield stress and plastic viscosity.[20.1] The models commonly used to describe the rheological behaviour of suspensions are the Bingham and the Newtonian models. These two models are illustrated in Figure 20.1. In the Bingham model, a certain shear stress, termed yield stress $\mathbf{T}_0$, is required to initiate flow of the suspension. Thereafter, an increase in shear stress results from an increase in strain rate. The plastic viscosity μ_{pl} describes the gradient of the curve and is a measure of the resistance to flow. In the Newtonian model, the yield stress is zero. An example of a fluid that follows the Newtonian model relatively closely is water, which does not need external energy to flow under its own weight. Conventional fresh concrete follows the Bingham model, ie in order to flow, the yield stress $\mathbf{T}_0$ must be overcome, which is commonly achieved through external vibration energy. SCC, not requiring external vibration, has a low yield stress and approaches the behaviour of a Newtonian fluid.

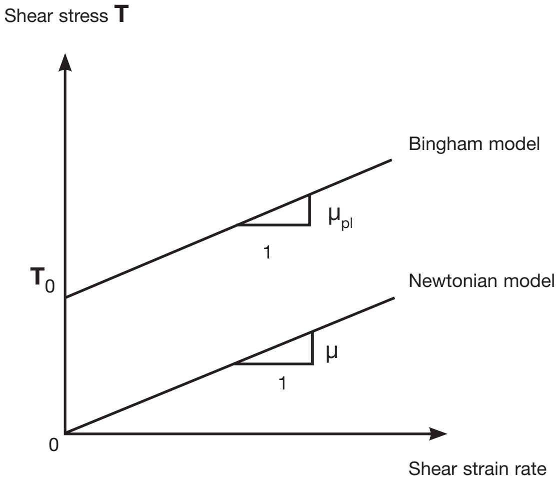

Figure 20.1: Principles of the Bingham and Newtonian models for the rheological behaviour of suspensions

In the cement paste, agglomerating fine particles need to be dispersed to decrease the yield stress and enable the mix to flow under its own weight. This is achieved by adding superplasticisers. Using a sufficiently high dosage of superplasticisers can fully eliminate the yield stress of the cement paste.[20.6] However, in order to prevent segregation of coarse aggregate, sufficient stability has to be provided through an increase in plastic viscosity. This is achieved through a reduction in the water:binder ratio and the increase in fine particle content, such as cement, cement extenders and inert fine particles. In addition, a viscosity-modifying admixture can be used to increase stability and optimise the plastic viscosity.

In achieving optimal rheological properties of fresh SCC, the balance between flow properties and segregation resistance is essential. The target rheological properties of SCC are a low yield stress and adequate plastic viscosity.[20.1] The latter depends on inter-particle friction of solid particles in the mix and therefore on the materials used, as well as on the casting technique and the size and shape of the structure to be built.

If coarse aggregates are in contact with each other, they produce shear stresses in the mix and hence hinder free flow, which is often referred to as the blocking effect. SCC therefore commonly contains a lower coarse aggregate content, compared to conventional concrete (Figure 20.2).

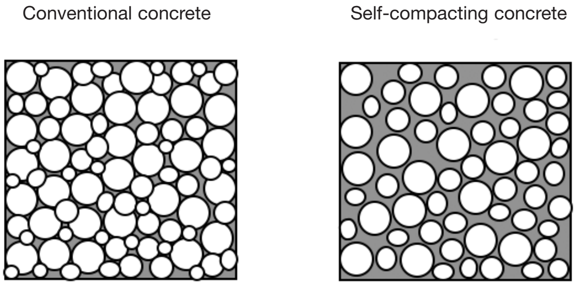

Figure 20.2: Schematic of coarse aggregate distribution in SCC versus conventional concrete

20.2.2 Workability

Workability refers to flow and deformation properties of concrete in practice. As such it reflects the rheological properties of the material, at the same time taking into account site conditions such as reinforcement layout, member shapes and placement methods. Workability of SCC is characterised by its filling ability, segregation resistance, and passing ability, as discussed in the following sections.

20.2.3 Filling ability

The filling ability describes the concrete's ability to flow under its own weight to completely fill the formwork and encapsulate the reinforcement. It is characterised by the deformation capacity (how far the concrete can flow under its own weight) and the velocity of deformation (how fast it can flow). The deformation capacity can be measured using the slump spread (flow diameter), while the velocity is given by the time taken to reach a certain flow diameter (see section 20.3.1 for information on relevant test methods). In order to obtain adequate filling ability, deformation capacity and velocity of deformation need to be balanced. Compared with conventional concrete, adequate filling ability can be achieved by the following actions:[20.1]

- Increasing the deformability of the paste by:
  - Adding superplasticisers (increasing flowability without significantly decreasing viscosity)
  - Balancing the water:powder ratio (a water:powder ratio that is too large may result in segregation, while a water:powder ratio that is too low will decrease the flowability of the mix)

- Reduce inter-particle friction by:
    - Using a low coarse aggregate content (high paste content)
    - Optimising the grading of the powder materials

20.2.4 Segregation resistance

Concrete can be subjected to different types of segregation. As it may result in blocking, segregation of coarse aggregate is considered to be the decisive factor for self-compactibility when casting heavily reinforced sections. Other types of segregation that need to be avoided in the manufacture of SCC include bleeding, segregation of fine aggregates in the mortar, and non-uniformity in air-pore distribution.[20.1] In principle, sufficient resistance to segregation in SCC can be achieved by:

- Limitation of coarse aggregate content
- Limitation of maximum aggregate size
- Application of low water:powder ratios
- Use of viscosity-modifying admixtures

20.2.5 Passing ability

Passing ability refers to the ability of the fresh concrete to flow through narrow formwork sections and between reinforcing bars without blocking through bridging. Bridging occurs when coarse aggregates get stuck in narrow sections and hinder the free flow of the concrete, which is often accompanied by coarse aggregate segregation. The reduction in coarse aggregate content, compared with conventional concrete, helps to avoid blocking. In certain applications, for example in densely reinforced sections, the coarse aggregate size may have to be minimised to ensure free flow through the reinforcement. Measures taken to increase the cohesiveness of the mix, such as the provision of low water:powder ratios and the application of viscosity-modifying admixtures, also help to provide adequate passing ability.

20.3 Testing of fresh concrete properties

Workability testing of fresh SCC includes determination of the three above-mentioned characteristics, ie filling ability, segregation resistance and passing ability. In many ways these are mutually linked as they depend on similar mix parameters and most test methods can be interpreted in terms of different workability characteristics. Several test methods have been developed to characterise the workability of SCC. Principles, detailed procedures and interpretation of these methods are discussed in the literature.[20.1, 20.4] Since no globally accepted standard exists for these methods, dimensions of test equipment and interpretation criteria often differ between countries, or even between different users in the same country. As an overview of basic test principles, the most commonly used methods for the characterisation of fresh properties of SCC are briefly described in the following sections. An example for specifying SCC based on fresh concrete properties is given in section 20.4.

20.3.1 Slump flow test

The slump flow test is a relatively simple test to perform and can be carried out with a normal slump cone. The slump cone is placed on a flat plate and filled with concrete, which is not compacted with a rod as it is with conventional concrete. The cone can be placed with the smaller opening to the bottom to prevent upward hydraulic pressure and therefore ease the testing. After lifting the cone, the spread of concrete is observed (Figure 20.3). The measurements include the slump flow value, ie the ultimate spread diameter, and the flow time. The slump flow value is an indicator of the deformation capability. The flow time, commonly measured as the time taken to reach a spread diameter of 500 mm (T_{500}), is used to evaluate the viscosity of the concrete.

The concrete spread is evaluated visually to detect any segregation. The latter is indicated through an accumulation of coarse aggregates in the centre of the spread (Figure 20.4). The distribution of coarse aggregate should be even across the flow diameter, ie the same quantity of coarse aggregate should be present at the boundary, compared to the inside of the flow spread (Figure 20.5).

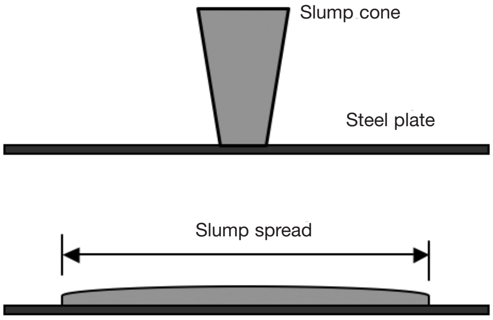

Figure 20.3: Schematic of the slump flow test

Figure 20.4: Example of coarse aggregate segregation, observed during the slump flow test (spread diameter approx. 550 mm)

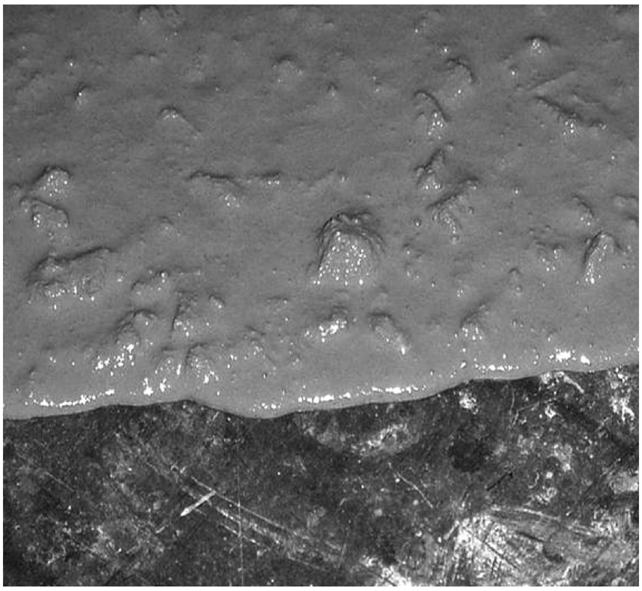

Figure 20.5: Example of good aggregate distribution in the slump flow test: the aggregate distribution at the boundary of the flow spread is the same as the distribution inside the flow spread (scale: width of photograph equals approx. 100 mm)

20.3.2 Slump flow test and blocking resistance

Blocking of coarse aggregates at reinforcing bars can occur even if the concrete has adequate fluidity and viscosity. The concrete's tendency to blocking therefore needs to be evaluated if is intended to be used in the construction of heavily reinforced members. This is done using the slump flow test (as described in section 20.3.1) in conjunction with a blocking ring, in which the slump cone is placed prior to being filled with the concrete (Figure 20.6). The blocking ring is a metal ring with a diameter of 300 mm, to which reinforcement bars 16 mm in diameter are attached with a clear spacing of 2-3 times the bar diameter. The dimensions of the blocking ring can be adjusted according to the specific site conditions. If the concrete flows freely through the blocking ring, the blocking resistance is considered adequate. If part or all of the concrete is trapped inside the ring, blocking resistance is inadequate and the mix design needs to be revised.

20.3.3 V-funnel test

The V-funnel test is used to evaluate the viscosity of the concrete. Typical dimensions of the test equipment are shown in Figure 20.7. After being filled with concrete, the bottom cover of the V-funnel is opened and the flow time measured. The latter is defined as the time taken until light can be seen through the bottom opening when observing from above. The faster the concrete flows, ie the shorter the flow time, the lower the viscosity.

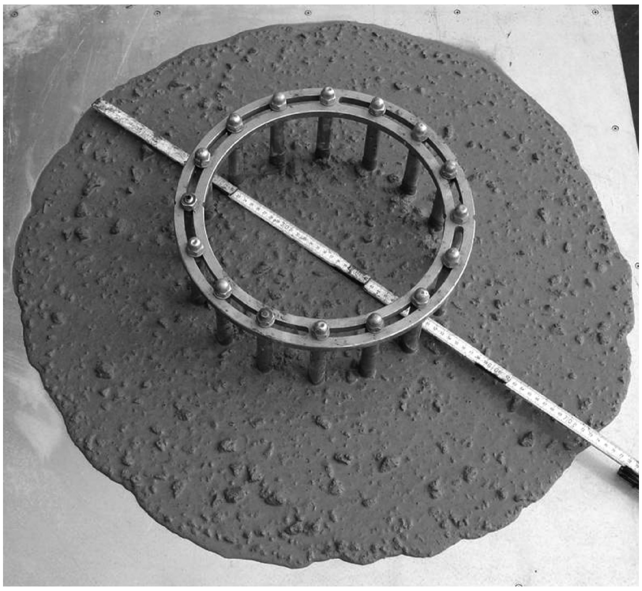

Figure 20.6: Example of the slump flow test together with the blocking ring

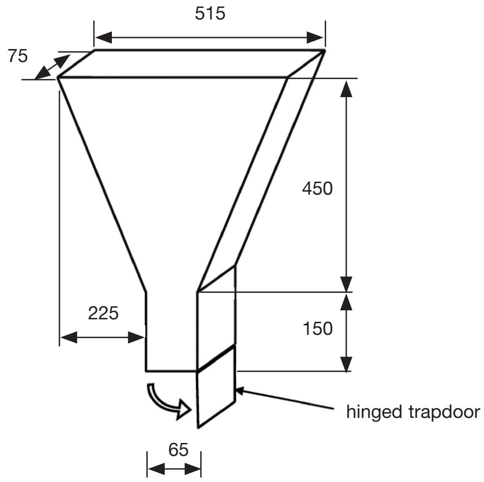

Figure 20.7: Schematic of the V-funnel test (dimensions in mm)[20.4]

The result of the V-funnel test depends on the deformability of the concrete and needs to be interpreted in relation to the test values obtained in the slump flow test.[20.1] A concrete with a larger slump flow results in a shorter V-funnel flow time, even if the viscosity is the same. The V-funnel test can therefore best be interpreted to optimise the viscosity of a concrete mix with constant slump flow value, a longer V-funnel flow time representing higher viscosity and hence better resistance to segregation.

20.3.4 U-box test

Self-levelling properties and blocking resistance of SCC can be investigated using the U-box test. The principles of the test are shown in Figure 20.8. The two compartments of the U-box are separated by a partition, which is removed once one side of the box has been filled with concrete. The concrete flows into the empty compartment and has to pass through vertical reinforcing bars situated at the border between the two compartments. The spacing of the rebars

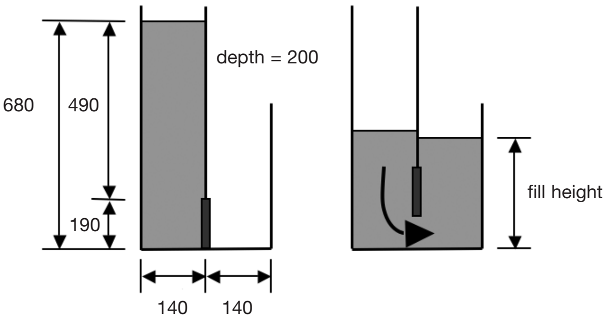

Figure 20.8: Schematic of the U-box test (dimensions in mm)

can be adjusted to the specific performance requirements. The height of the rising concrete in the previously empty compartment, the so-called "fill height" is measured. The fill height must reach a minimum value in order for the concrete to be assumed to have adequate self-levelling properties, commonly set at 300 mm.[20.1] In addition, the time taken for the concrete to rise to its final height can be measured and linked to the viscosity of the mix.

20.3.5 L-box test

The L-box test follows similar principles as the U-box test and is also used to evaluate self-levelling properties and blocking resistance (Figure 20.9). After filling the vertical compartment of the box, the partition is removed and the flow behaviour of the concrete observed. The measurement parameters include flow time to the 200 mm mark, flow time to the 40 mm mark, and final filling heights H_1 and H_2. For good self-levelling characteristics, the ratio between H_2 and H_1 should not exceed 0,80.[20.1] If the box contains vertical reinforcing bars at the border between the vertical and horizontal compartment, the blocking resistance can be visually assessed. The L-box test is similar to the slump flow test in that it evaluates the flowability of concrete, however with the difference that the flow is uni-directional while the slump flow test allows multi-directional flow.

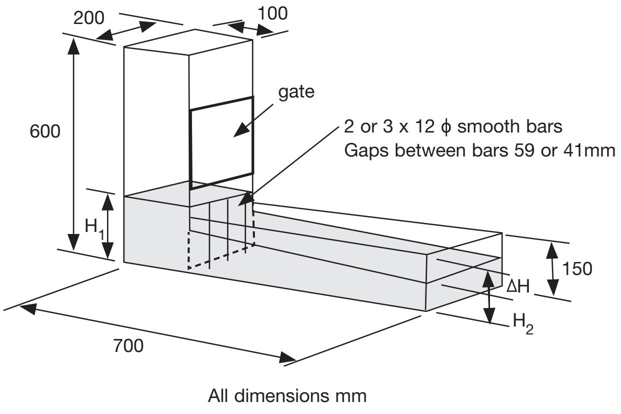

Figure 20.9: Schematic of the L-box test[20.4]

20.3.6 Segregation resistance (sieve) test

The sieve test[20.4] aims at determining how likely an SCC mix is to segregate. A 10-litre concrete sample is placed into a container and allowed to stand at rest for about 15 minutes. At the end of this period, the top 4,8 ± 0,2 kg of concrete is poured from the container into a sieve having 5-mm square apertures. The height from which the concrete is poured into the sieve is 500 ± 50 mm. During this process, a part of the mortar passes through the sieve. The ratio between the mass of mortar passed through the sieve and the total mass of concrete poured onto the sieve is used to indicate the likelihood of segregation. A higher mass of mortar passing through the sieve corresponds to a greater tendency to segregate.

20.4 Specifying SCC based on fresh concrete properties

The European guidelines for self compacting concrete[20.4] present recommendations for the specification of ready-mixed and site-mixed SCC, based on test results for fresh concrete properties. The preferred test methods for the characterisation of SCC are presented in Table 20.1. The classification of SCC based on the numerical test values for fresh concrete properties are shown in Table 20.2.

Table 20.1: Preferred test methods for characterisation of SCC according to ***European guidelines for self compacting concrete*****[20.4]**

Workability characteristic	Preferred test method(s)
Flowability	Slump flow test
Viscosity	T_{500} Slump flow test or V-funnel test
Passing ability	L-box test
Segregation	Segregation resistance (sieve) test

The guidelines present typical classes of slump-flow, viscosity, passing ability and segregation resistance for various applications. A specification example, based on Walraven[20.7] is presented in Figure 20.10.

20.5 Mix design principles

20.5.1 Introduction

In order to be self-compacting, concrete needs a combination of the following fresh properties:[20.1]

- Filling ability, ie sufficiently high fluidity and deformability to ensure adequate flow under self weight
- Segregation resistance during flow and at rest after placing
- Passing ability, ie the ability to pass through and around reinforcing bars without blocking

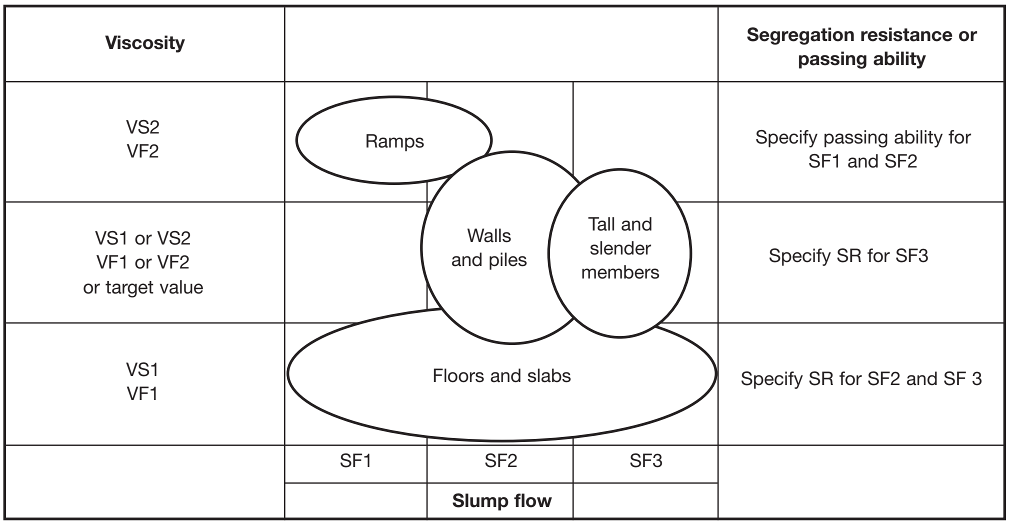

Figure 20.10: Specification example:[20.4] Properties of SCC for various applications based on Walraven [20.7]

Table 20.2: Classification of SCC according to *European guidelines for self compacting concrete*[20.4]

Slump flow classes	
Class	**Slump flow in mm**
SF1	550 - 650
SF2	660 - 750
SF3	760 - 850

Viscosity classes		
Class	**T_{500}, s**	**V-funnel time, s**
VS1 / VF1	≤ 2	≤ 8
VS2 / VF2	> 2	9 - 25

Passing ability classes (L-box)	
Class	**Passing ability, H_2/H_1**
PA1	≥ 0.80 with 2 rebars
PA2	≥ 0.80 with 3 rebars

Segregation resistance classes, sieve	
Class	**Passing ability, %**
SR1	≤ 20
SR2	≤ 15

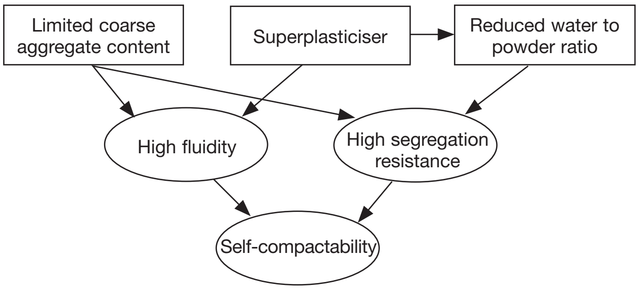

Figure 20.11: General approach to achieving self-compacting properties of fresh concrete [20.1, 20.8]

In principle, passing ability of fresh SCC is achieved through limitation of the coarse aggregate content. The resulting increase in paste content and the addition of superplasticisers provide the necessary fluidity and deformability. Segregation resistance is provided through a reduction in water:powder content. The general approach to achieving self-compacting properties of fresh concrete, based on work done by Ouchi et al[20.1, 20.8] is illustrated in Figure 20.11. A common modification to this approach is the addition of viscosity-modifying agents to improve segregation resistance. Generally, SCC is distinguished by the "powder type", the "viscosity-agent type" and the "combination type", depending on the prevailing method for provision of adequate viscosity.

The primary focus of mix design for SCC is its fresh concrete properties, with less initial attention being given to the subsequent hardened properties.[20.1] Important mix design parameters influencing the workability of the fresh mix are paste content, water:powder ratio, and admixture dosage. Strength of hardened concrete is then commonly controlled through selection of an appropriate combination of powder materials, such as cement, cement extenders and inert fine particles.

A considerable number of variables influence workability properties of SCC. The initial mix design thus commonly includes extensive testing of potential combinations of mix constituents, through which rheological properties are optimised. This is generally done in the laboratory by determination of cement paste and mortar flow properties, from which water:binder ratio, fine particle content and admixture dosage are determined. The rheology of the actual SCC is then confirmed, and adjusted if necessary, using various experimental techniques.

The first published mix design method for SCC was developed in Japan in the mid 1980's[20.9, 20.10] and has become known as the "General Method". This method results in adequate self-compacting properties for many applications, but has been shown to be inefficient in terms of paste content.[20.1] Another problem with the General Method is that it was developed for Japanese materials and may not completely suit materials available in other countries. Mix design methods developed by various researchers are often based on the General Method and are aimed at producing efficient and economical mixes with available materials.

In the following section, the General Method is described as it provides an insight into basic mix design procedures for SCC. A comprehensive list of references for alternative mix design methods, as well as the basic principles of selected alternative methods, can be found in the State-of-the-art report of RILEM TC 174-SCC.[20.1]

20.5.2 Mix design according to the General Method

The method is based on the assumption that SCC can be produced by adding coarse aggregate to a mortar with high fluidity and adequate viscosity. Initially, the required air content for the relevant exposure condition is determined in case air content is a design parameter. The coarse aggregate content, measured as dry rodded content, is set to 50% of the concrete volume minus air content. The fine aggregate content is set to 40% of the resulting mortar volume. The water: powder ratio and superplasticiser dosage are determined experimentally through a series of rheological experiments on cement paste, mortar and concrete. Figure 20.12 shows the mix proportions according to the General Method.

A mortar flow cone is used to measure the spread diameter (slump flow) of paste and mortar, which relates to the deformability of the mix. The viscosity of the mortar is determined using a V-funnel, which measures the flow time. The test principles correspond to those of the slump flow and V-funnel tests described for the testing of concrete in sections 20.3.1 and 20.3.3 respectively. Test procedure, dimensions of the test equipment and interpretation of the paste and mortar workability test methods are discussed in the literature.[20.1]

In the first experimental step, the amount of water not contributing to the flowability of the paste, the so-called optimum water:powder ratio β_P, is determined. This is done by measuring the slump flow of pastes at different water: powder ratios and plotting the water:powder ratio against the relative slump flow Γ_{paste} (final flow area – initial flow area)/initial flow area) (Figure 20.13). The water demand of the powder mixture is obtained through extrapolation of the graph, the interception with the y-axis ($\Gamma_{paste} = 0$) indicating the optimum water: powder ratio β_{paste}.

Following the above mentioned design principles, the fine aggregate content is set to about 40% of the mortar volume. In the second experimental step, superplasticiser dosage and water:powder ratio of the mortar mixture are optimised to achieve a relative flow of $\Gamma_{mortar} = 5$ and a flow time of about 9 – 11 seconds. The resulting mortar mix is then used to start trial mixes on the concrete, using the mix proportions given above. The rheological properties of the concrete are optimised by adjusting the dosage of superplasticiser. A slump flow of 65 cm (slump cone) and a flow time, measured with the V-funnel, of 10 – 20 seconds are considered adequate.[20.1]

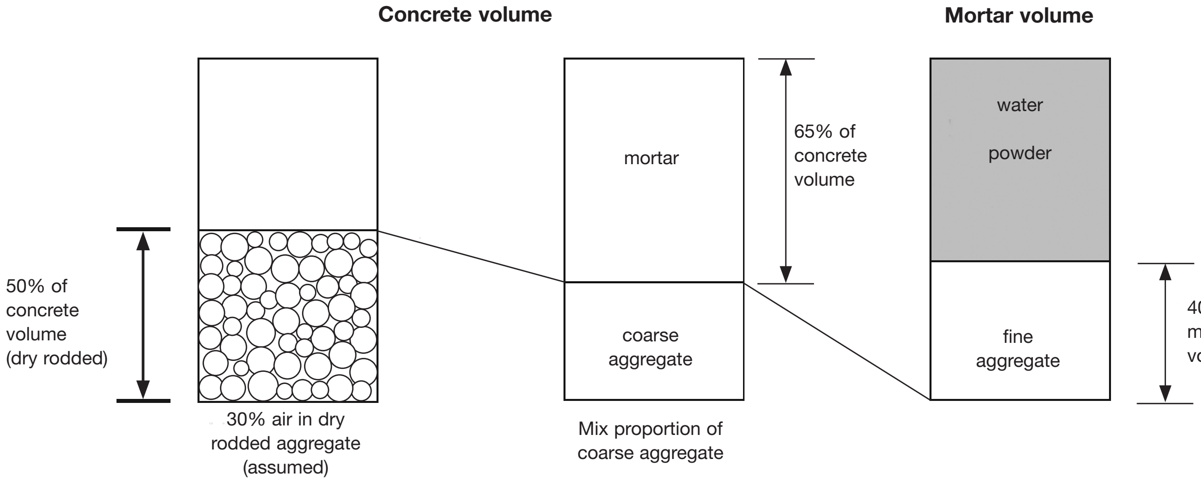

Figure 20.12: Mix proportions according to the General Method[20.9]

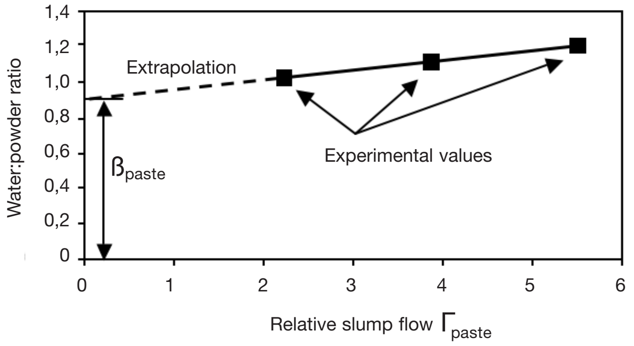

Figure 20.13: Determination of the optimum water:powder ratio of cement paste

20.5.3 Limiting values for mix proportions

Various mix design methods have been developed for SCC, usually applicable to certain types of materials and specific performance requirements. Mix proportions therefore depend on available resources and the design method employed. As an overview, the State-of-the-art report of RILEM TC 174-SCC[20.1] presents useful limiting values for mix proportions for SCC, based on practical experience, as follows:

- Absolute coarse aggregate content of 30 – 34% by volume of the concrete, as opposed to roughly 40 – 45% in conventional concrete. (This corresponds to the dry rodded coarse aggregate content of 50% suggested in the General Method, considering the dry-rodded density of common coarse aggregates to be approximately 60% of the absolute density.)
- A water:powder ratio of 0,8 – 1,2. Mixes approaching a water:powder ratio of 1,2 commonly require a viscosity-modifying agent.
- A water content of 155 – 175 ℓ/m³ if no viscosity-modifying agent is used.
- A water content of up to about 200 ℓ/m³ with a viscosity-modifying agent.
- A paste volume of 34 – 40% of the concrete volume.
- A fine aggregate content of 40 – 50% of the mortar volume

The above quantities are approximately equivalent to the material contents listed in Table 20.3.

Table 20.3: Approximate material contents for SCC, based on practical experience[20.1]

Constituent	Mix proportion, kg/m^3
Coarse aggregate	750 - 920
Fine aggregate	710 - 900
Powder	450 - 600
Water	150 - 200

20.6 Constituent materials

20.6.1 General

In principle, the same constituent materials are used to make SCC as those used for conventional concrete. One main difference is the need for more stringent quality control of materials used in the production of SCC, as minor changes in material properties, eg grading of aggregates and powder materials, or moisture content, may have a significant influence on workability.

20.6.2 Cement and additional powder materials

As a result of the higher paste content and the fresh concrete requirements for cohesion and segregation resistance, SCC contains a higher powder content than conventional concrete. To control hydration heat development and thermal shrinkage, the cement content needs to be kept reasonably low, which is one reason why SCC commonly contains inert, pozzolanic and/or latent hydraulic additions in its powder fraction.[20.4] How effectively and in what quantities different cement extenders or powder materials can be used to manufacture SCC should be tested for the specific materials available. The range of commonly used powder materials in SCC includes FA, CSF, GGBS, mineral fillers, and Metakaolin. Advantages and disadvantages of these materials in the production of SCC are discussed in the literature.[20.4, 20.5, 20.11, 20.12]

20.6.3 Aggregates

Aggregate shape and grading is important as it affects internal friction and hence the flowability of the fresh SCC. Rounded aggregate particles are generally preferred as they result in enhanced flow properties. The maximum coarse aggregate size needs to be chosen according to rebar diameter and spacing, so that blocking is avoided (see section 20.3.2). In general, the maximum coarse aggregate size should be limited to 12 – 20 mm.[20.4]

The properties of fine aggregates have a significantly greater influence on fresh properties of SCC than those of coarse aggregates. Depending on material properties such as grain size and grading, different types of available fine aggregate may need to be mixed for optimum performance. The aggregate fraction finer than 0,125 mm is assumed to contribute to the powder content of the mix and needs to be taken into account when calculating the water:powder ratio.[20.4]

20.6.4 Admixtures

Superplasticisers are an essential component in limiting the water content and at the same time achieving the required fluidity of SCC. A broad range of superplasticisers is available, suitable for different applications and performance requirements of SCC. For ready-mixed concrete it is important to choose superplasticisers with sufficient workability retention, as it is very difficult to successfully improve workability of the concrete on site, owing to the many factors affecting the flow behaviour and viscosity of SCC.

Depending on the mix design, viscosity-modifying admixtures (VMA) may be used to provide stability and segregation resistance. VMA can further help to minimise the effects of variations in moisture content and fines in the sand, making the concrete less sensitive to production-related influences.[20.4]

Other admixtures such as air-entraining agents, retarders or accelerators may be used in the same way as in conventional concrete.

Because the fresh properties of SCC are sensitive to variations in mix quantities and constituents, changing admixture sources, or admixture types from the same source, may result in significant changes in workability properties. For the same reason, a particular admixture may be successful in providing the required performance in a particular mix, but not if there are changes in the mix design or constituent materials. Application of various admixtures should therefore be preceded by thorough experimental investigations for the particular mix design.

20.7 Mixing, handling and curing

Mixing of SCC can be done using the same equipment as conventional concrete. The principal difference is that SCC requires a prolonged mixing time due to reduced frictional forces and to fully activate the superplasticiser.[20.4] Prolonged mixing further improves dispersion of the high amount of fines in the mix.[20.2] If SCC is delivered from a manufacturing plant, workability retention and segregation need to be carefully checked once the concrete arrives on site. The time taken for the transport is required to be as short as possible and is often specified not to exceed 60 minutes. However, using modern superplasticisers commonly allows for longer workability retention.[20.1] New generations of admixtures offer the possibility of setting a specific target on workability retention time depending on prevailing production conditions.[20.2]

As fresh SCC generally requires more stringent quality control than conventional concrete, site acceptance criteria may also be different; these are discussed in the literature.[20.4] The slump flow test is commonly recommended for characterising SCC on site, as it represents a good indication of the uniformity of concrete supply, gives information on the filling ability, and allows a visual assessment of segregation resistance.

Pumping is the most common method of placing SCC, but other discharging equipment can be used. Conventional pumping, as well as pumping bottom up, where the concrete is discharged at the bottom and subsequently rises to the top of the formwork, have been applied successfully.

One of the main advantages of SCC is the increased speed of placing, resulting from the lack of need for vibrating and levelling and the increased flow distances. The latter commonly allows the concrete to be discharged at a single location into the formwork without the pump outlet or discharger having to be moved around. Flow distances should however not exceed 10 m in order to minimise the risk of dynamic segregation.[20.4] The casting procedure should be continuous without unnecessary interruption to help maintain the flow and allow problem-free filling of the formwork.

Due to its high fluidity, fresh SCC exerts higher pressure on formwork than conventional concrete. This needs to be taken into account when designing and assembling the formwork.

Because of its high viscosity, SCC tends to be sticky and does not develop bleed water, which may make the finishing of large horizontal surfaces, eg slab surfaces, difficult. The absence of bleed water together with low water:powder ratios and high paste contents emphasises the need for proper curing of SCC, which should commence as soon as possible after placing and finishing, otherwise there is a very high risk of plastic shrinkage cracking.[20.4]

20.8 Hardened concrete properties

20.8.1 General

SCC is considered to have a denser and more homogeneous microstructure than vibrated concrete, resulting from improved properties of the contact zone between cement paste and aggregates.[20.1,20.4,20.5] Compacting normal fresh concrete causes water to accumulate on coarse aggregate particles, resulting in a porous and weak interfacial transition zone (ITZ). SCC, not being subjected to compaction shows better ITZ properties. Other reasons for an improved microstructure of SCC, compared to conventional concrete, are increased fines content and better water retention properties.[20.1] The practical significance of the improved microstructure of SCC is related mainly to somewhat higher strength at equal water-binder ratio and enhanced durability properties. Furthermore, as a result of the lack of bleeding, the bond between concrete and reinforcing bars is assumed to be improved when using SCC.[20.4]

The variation of in-situ properties of SCC was found to be similar to that of vibrated concrete. For SCC the most important factor for good performance is the mix design, whereas for conventional concrete site practice is more critical.[20.12]

In general, SCC is chosen and designed for its fresh concrete properties, while most hardened properties are assumed to be similar to those of vibrated concrete, at the same or similar mix proportions.

20.8.2 Deformation and volume change

Concrete properties such as shrinkage, creep and elastic modulus are influenced by the coarse aggregate content. Lower aggregate contents, ie higher paste contents, generally result in higher shrinkage and creep strains and lower values for elastic modulus. The reason for this is that shrinkage and creep deformations take place in the paste, while the coarse aggregates provide restraint and dilution to these deformations. SCC has a considerably higher paste content than conventional concrete which, for the above reasons, may need to be considered when deformation properties and volume changes are to be considered in the design.

According to the European guidelines for SCC[20.4] the differences in shrinkage, creep and elastic deformations between SCC and conventional concrete are relatively small, so that the design methods provided in the Eurocodes[20.13] for conventional concrete still apply for SCC. A literature review done by Domone[20.12] revealed that at compressive strength of about 20 MPa, SCC can have a significantly lower elastic modulus (up to 40% lower), than conventional concrete. This difference decreases to about 5% at high strengths of about 90 – 100 MPa. As a result of different elastic moduli, structural elements made from SCC can, at equal load-bearing capacity, be expected to deform more than elements made with conventional concrete.

20.8.3 Durability

Practical experience and research results on durability characteristics of SCC are still limited. A state-of-the-art report prepared by RILEM TC 205-DSC[20.5] summarises current knowledge on durability of SCC, and gives some important conclusions, which are discussed below.

The factors influencing transport properties of SCC appear to be the same as those for conventional concrete, ie water: binder ratio, degree of hydration and mineral additions. At equal water:binder ratio and compressive strength grade, SCC has significantly lower permeability, when tested on oven-dried samples. The main reasons for the enhanced resistance to fluid transport are believed to be the refined pore structure and more uniform and denser ITZ of SCC.
Although research on carbonation resistance of SCC still yields contradictory results, it appears that carbonation of SCC is not significantly different from that of conventional concrete.

The high powder content and admixture dosage in SCC may result in different pore volumes, different ionic compositions in the pore solution and perhaps different chloride binding characteristics, compared with conventional concrete. When electrochemical test methods, which were developed and calibrated for conventional concrete, are applied to characterise SCC for chloride penetration, experimental results need to be carefully analysed and interpreted. In principle, current experience and knowledge suggests that the chloride resistance of SCC is to a large degree dependent on the type of cement and additional powder material used. This is similar to conventional concrete.

The physical resistance to sulphate attack appears to be significantly enhanced in SCC. The improved ability to withstand sulphate attack is a result mainly of the denser microstructure of SCC. However, when a large amount of limestone powder is added to the mix, the risk of delayed ettringite formation may be increased.

Experience and knowledge of alkali-silica reaction (ASR) in SCC is scarce. At this stage it is reasonable to assume that the same factors that influence ASR in conventional concrete also apply to SCC.

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Chapter 21

Fibre-reinforced concrete

Bryan Perrie

21.1 Introduction

Fibrous materials have been used since ancient times to reduce cracking and improve toughness and strength of brittle building materials. Examples are straw in clay bricks and hair in plaster.

Many different types of fibre can be used in concrete. The effects of fibre-reinforcement depend on the type and content of fibre and the properties of the matrix.

The aim of this chapter is to provide information on the properties of the more commonly available fibres and their uses together with some theoretical considerations regarding the use of fibres to achieve certain properties.

A substantial amount of the material in this chapter is sourced from the very comprehensive ACI 544.1R-96 *Report on fiber reinforced concrete.*

21.2 Types of fibre

In this section each of the most commonly used fibre types is discussed, giving information on the manufacture of the fibre, its properties, fibre content in applications and the effects of the fibre type on concretes and mortars.

21.2.1 Glass

Glass fibre, in the form first used as reinforcement in concrete in the early 1960s, consisted of conventional borosilicate glass and soda-lime silica glass. These fibres were found to be alkali reactive and products in which they were used deteriorated rapidly. Alkali-resistant glass containing 16% zirconia was successfully formulated in the 1960s and by 1971 was in commercial production in the UK. Other sources of alkali-resistant glass were developed during the 1970s and 1980s in other parts of the world with higher zirconia contents. Alkali-resistant glass fibre is used in the manufacture of glass-reinforced cement (GRC) products, which have a wide range of applications.

Glass fibre is available in continuous or chopped lengths. Fibre lengths of up to 35 mm are used in spray applications and 25-mm lengths in pre-mix applications.

Glass fibre has high tensile strength (2 to 4 GPa) and elastic modulus (70 to 80 GPa) but has brittle stress-strain characteristics (2,5 to 4,8% elongation at break) and low creep at room temperature. Claims have been made that up to 5% glass fibre by volume has been used successfully in sand-cement mortar without balling.

Glass-fibre products exposed to outdoor environment have shown a loss of strength and ductility. The reasons for this are not clear and it is speculated that alkali attack or fibre embrittlement are possible causes. Because of the lack of data on long-term durability, GRC has been confined to non-structural uses where it has wide applications. It is suitable for use in direct spray techniques and premix processes and has been used as a replacement for asbestos fibre in flat sheet and pipe applications as well as in a variety of precast products. GRC products find wide use in agriculture; architectural cladding and components; and small containers.

21.2.2 Steel

Steel fibres have been used in concrete since the turn of the century. The early fibres were round and smooth and the wire was cut or chopped to the required lengths. The use of straight, smooth fibres has largely disappeared and modern fibres have either rough surfaces, hooked ends or are crimped or undulated through their length. Modern commercially available steel fibres are manufactured from drawn steel wire, from slit sheet steel or by the melt-extraction process which produces fibres that have a crescent-shaped cross section. Typically steel fibres have equivalent diameters (based on cross-sectional area) of from 0,15 mm to 2 mm and lengths from 7 to 75 mm. Aspect ratios generally range from 20 to 100. (Aspect ratio is defined as the ratio between fibre length and its equivalent diameter, which is the diameter of a circle with an area equal to the cross-sectional area of the fibre.)

Carbon steels are most commonly used to produce fibres but fibres made from corrosion-resistant alloys are available. Stainless steel fibres have been used for high-temperature applications.

Some fibres are collated into bundles using water-soluble glue to facilitate handling and mixing.

Steel fibres have high tensile strength (0,5 to 2 GPa) and modulus of elasticity (200 GPa), a ductile/plastic stress-strain characteristic and low creep.

Steel fibres have been used in conventional concrete mixes, shotcrete and slurry-infiltrated fibre concrete. Typically, content of steel fibre ranges from 0,25 to 2,0% by volume. Fibre contents in excess of 2% by volume generally result in poor workability and fibre distribution, but can be used successfully where the paste content of the mix is increased and the size of coarse aggregate is not larger than about 10 mm.

Steel-fibre-reinforced concrete containing up to 1,5% fibre by volume has been pumped successfully using pipelines of 125 to 150 mm diameter. Steel fibre contents up to 2% by volume have been used in shotcrete applications using both the wet and dry processes. Steel fibre contents of up to 25% by volume have been obtained in slurry-infiltrated fibre concrete.

Concretes containing steel fibre have been shown to have substantially improved resistance to impact and greater ductility of failure in compression, flexure and torsion.

Similarly, it is reported that the elastic modulus in compression and modulus of rigidity in torsion are no different before cracking when compared with plain concrete tested under similar conditions. It has been reported that steel-fibre-reinforced concrete, because of the improved ductility, could find applications where impact resistance is important. Fatigue resistance of the concrete is reported to be increased by up to 70%.

It is thought that the inclusion of steel fibre as supplementary reinforcement in concrete could assist in the reduction of spalling due to thermal shock and thermal gradients.

The lack of corrosion resistance of normal steel fibres could be a disadvantage in exposed concrete situations where spalling and surface staining are likely to occur.

21.2.3 Synthetic fibres

Synthetic fibres are man-made fibres resulting from research and development in the petrochemical and textile industries. Currently there are two different synthetic fibre volumes used in application, namely low-volume percentage (0,1 to 0,3% by volume) and high-volume percentage (0,4 to 0,8% by volume). There are two different physical fibre forms: monofilament fibres, and fibres produced from fibrillated tape. Most synthetic fibre applications are currently at the 0,1% by volume level. At this level, the strength of the concrete is considered unaffected and crack control characteristics are sought in the plastic state, ie while concrete has not achieved significant strength, in order to control plastic shrinkage and crack formation.

Fibre types that have been tried in cement concrete matrices include: acrylic, aramid, carbon, nylon, polyester, polyethylene, polypropylene and polyvinyl alcohol. Table 21.1 summarises the range of physical properties of some synthetic fibres.

Table 21.1: Selected synthetic fibre types and properties* [21.1]

Fibre type	Equivalent diameter, µm	Relative density	Tensile strength, MPa	Elastic modulus, GPa	Ultimate elongation, %	Ignition temperature, °C	Melt, oxidation, or decomposition temperature, °C	Water absorption per ASTM D 570, % by mass
Acrylic	13 - 104	1,16 - 1,18	270 - 1 000	14 - 19	7,5 - 50,0	-	220 - 235	1,0 - 2,5
Aramid I	12	1,44	2 900	60	4,4	high	480	4,3
Aramid II†	10	1,44	2 350	115	2,5	high	480	1,2
Carbon, PAN HMΔ	8	1,6 - 1,7	2 500 - 3 000	380	0,5 - 0,7	high	400	nil
Carbon, PAN HT§	9	1,6 - 1,7	3 450 - 4 000	230	1,0 - 1,5	high	400	nil
Carbon, pitch GP**	10 - 13	1,6 - 1,7	480 - 790	27 - 35	2,0 - 2,4	high	400	3 - 7
Carbon, pitch HP††	9 - 18	1,8 - 2,15	1 500 - 3 100	150 - 480	0,5 - 1,1	high	500	nil
NylonΔΔ	23	1,14	970	5	20	-	200 - 220	2,8 - 5,0
Polyester	20	1,34 - 1,39	230 - 1 100	17	12 - 150	600	260	0,4
PolyethyleneΔΔ	25 - 1 000	0,92 - 0,96	75 - 590	5	3 - 80	-	130	nil
PolypropyleneΔΔ	-	0,90 - 0,91	140 - 700	3,5 - 4,8	15	600	165	nil

Notes

* Not all fibre types are currently used for commercial production of FRC.

† High modulus.

Δ Polyacrylonitrile based, high modulus.

§ Polyacrylonitrile based, high tensile strength.

** Isotropic pitch based, general purpose.

†† Mesophase pitch based, high performance.

ΔΔ Data listed is only for fibres commercially available for FRC.

Acrylic

Acrylic fibres have been used to replace asbestos fibre in many fibre-reinforced concrete products. In this process fibres are initially dispersed in a dilute water and cement mixture. A composite thickness is built up in layers using a pressure forming process and vacuum dewatering. Acrylic fibres have also been added to conventional concrete at low volumes to reduce the effects of plastic-shrinkage cracking.

Aramid

Aramid fibres are two and a half times as strong as glass fibres and five times as strong as steel fibres per unit weight. Due to the relatively high cost of these fibres, aramid-fibre-reinforced concrete has been primarily used as an asbestos cement replacement in certain high-strength applications.

Carbon

Carbon fibre is substantially more expensive than other fibre types. For this reason its commercial use has been limited.

Carbon fibres are manufactured by carbonising suitable organic materials in fibrous forms at high temperatures and then aligning the resultant graphite crystallites by hot-stretching. The fibres are manufactured as either Type I (high modulus) or Type II (high strength) and are dependent upon material source and extent of hot stretching for their physical properties. Carbon fibres are available in a variety of forms and have a fibrillar structure similar to that of asbestos.

Carbon fibre made from petroleum and coal pitch is less expensive than the conventional carbon fibre made from fibrous materials. The Type I and II carbon fibres produced by carbonising suitable organic materials other than petroleum-based types are 20 to 40 times stronger and have a modulus of elasticity up to 100 times greater than the pitch-based carbon fibre.

Carbon fibre is available as continuous strands or as individual chopped fibres. Continuous strands are normally pre-placed and aligned to provide the optimum fibre orientation during fabrication. Chopped fibres are generally incorporated during the mixing process and are therefore orientated randomly throughout the mix. A satisfactory mix of chopped carbon fibre, cement and water is difficult to achieve because of the large surface area of the fibre. Research has shown that uniform dispersion of discontinuous low-modulus carbon fibre has been achieved using an omnimixer and admixture. Carbon fibre has high tensile strength and modulus of elasticity and a brittle stress-strain characteristic.

Additional research is needed to determine the feasibility of carbon-fibre concrete on an economic basis. The fire-resistant properties of carbon-fibre composites need to be evaluated, but ignoring economics, structural applications appear promising.

Nylon

Nylon is a generic name that identifies a family of polymers. The properties of nylon fibres are imparted by the base polymer type, addition of different levels of additive, manufacturing conditions and fibre dimensions. Currently only two types of nylon fibre are marketed for concrete. Nylon is heat stable, hydrophilic, relatively inert and resistant to a wide variety of materials.

Nylon is particularly effective in imparting impact resistance and flexural toughness and sustaining and increasing the load-carrying capacity of concrete following first crack.

Polyester

Polyester fibres are available in monofilament form and belong to the thermoplastic polyester group. They are temperature sensitive and above normal service temperatures their properties may be altered. Polyester fibres are somewhat hydrophobic. Polyester fibres have been used at low contents (0,1% by volume) to control plastic-shrinkage cracking in concrete.

Polyethylene

Polyethylene has been produced for concrete in monofilament form with wart-like surface deformations. Polyethylene in pulp form may be an alternate to asbestos fibres. Concrete reinforced with polyethylene fibres at contents between 2 and 4% by volume exhibits a linear flexural load deflection behaviour up to first crack, followed by an apparent transfer of load to the fibres permitting an increase in load until the fibres break.

High molecular weight (elastic modulus in the region of 70 to 120 GPa) polyethylene fibres have been used successfully to modify the tensile mechanical behaviour of cement-based composites. Significantly increased ultimate tensile strength and especially toughness of concrete and mortars can be obtained with fibre contents of 1,5 to 2% by volume [21.2, 21,3].

Polypropylene

Polypropylene fibre was first used to reinforce concrete in the 1960s. Polypropylene is a synthetic hydrocarbon polymer, the fibre of which is made using extrusion processes by hot-drawing the material through a die. Polypropylene fibres are produced as continuous monofilaments with circular cross section that can be chopped to required lengths or fibrillated films or tapes of rectangular cross section.

Polypropylene fibres are hydrophobic and therefore have the disadvantages of poor bond characteristics with cement matrix, low melting point, combustibility and a relatively low modulus of elasticity. Long polypropylene fibres can prove difficult to mix due to their flexibility and tendency to wrap around the leading edges of mixer blades.

Polypropylene fibres are tough but have low tensile strength and modulus of elasticity; they have a plastic stress-strain characteristic.

Monofilament polypropylene fibres have inherent weak bond with the cement matrix because of their relatively small specific surface area. Fibrillated polypropylene fibres are slit and expanded into an open network thus offering a larger specific surface area with improved bond characteristics. Polypropylene fibre contents of up to 12% by volume are claimed to have been used successfully using hand-packing fabrication techniques, but volumes of 0,1% of 50-mm fibre in concrete have been reported to have caused a slump loss of 75 mm.

Polypropylene fibres have been reported to reduce unrestrained plastic and drying shrinkage of concrete at fibre contents of 0,1 to 0,3% by volume.[21.4]

Polyvinyl alcohol

In the medium volume content range (2 to 3%) Polyvinyl alcohol (PVA) fibres have been used successfully to modify the tensile and flexural mechanical behaviour of cement-based matrices containing fine aggregate. The fibres have high tensile strength and elastic modulus, which enable crack bridging capability. The fibres also have a strong bond with cement-based matrices, with average bond strengths of between 2 and 4 MPa. In fact, it has been shown to be beneficial to reduce this bond by treatment of the fibre surface, in order to allow controlled slippage from the matrix of fibres bridging cracks, resulting in tough tensile response. For high modulus polyethylene fibres, which are hydrophobic, surface treatments have been performed to increase the bond for more effective crack bridging. For mortars containing PVA or polyethylene fibres modified in this way, in the volume percentage range (2 to 3%), full tensile resistance for tensile strain levels of 3 to 5% is achieved.[21.5]

21.2.4 Natural fibres

Natural reinforcing materials can be obtained at low cost and low levels of energy using local manpower and technology. Utilisation of natural fibres as a form of concrete reinforcement is of particular interest to less developed regions where conventional construction materials are not readily available or are too expensive. Sisal-fibre-reinforced concrete has been used for making roof tiles, corrugated sheets, pipes, silos and tanks. Elephant-grass-reinforced mortar has been used for low-cost housing projects. Wood-cellulose-fibre-reinforced cement has commercial applications in the manufacture of flat and corrugated sheet and non-pressure pipes. Typical properties of natural fibres are shown in Table 21.2.

Natural fibres can be either unprocessed or processed.

Unprocessed natural fibres

Products made with unprocessed natural fibres such as coconut coir, sisal, sugarcane bagasse, bamboo, jute, wood and vegetable fibres have been tested in a number of countries. Problems have been reported on the long-term durability of some of the products.

The properties of concrete made using unprocessed natural fibres depend on a number of factors including the type and length of fibre as well as the volume fraction. To show some improvement in mechanical properties, the minimum fibre content is of the order of 3% by volume.

Table 21.2: Typical properties of natural fibres [21.6]

Fibre type	Coconut	Sisal	Sugar cane bagasse	Bamboo	Jute	Flax	Elephant grass	Water reed	Plantain	Musamba	Wood fibre (Kraft pulp)
Fibre length, mm	50 - 100	N/A	N/A	N/A	175 - 300	500	N/A	N/A	N/A	N/A	2,5 - 5,0
Fibre diameter, mm	0,1 - 0,4	N/A	0,2 - 0,4	0,05 - 0,4	0,1 - 0,2	N/A	N/A	N/A	N/A	N/A	0,025 - 0,075
Relative density	1,12 - 1,15	N/A	1,2 - 1,3	1,5	1,02 - 1,04	N/A	N/A	N/A	N/A	N/A	1,5
Modulus of elasticity, GPa	19 - 26	13 - 26	15 - 19	33 - 40	26 - 32	100	5	5	1,5	1,0	N/A
Ultimate tensile strength, MPa	120 - 200	275 - 570	180 - 290	350 - 500	250 - 350	1 000	180	70	90	80	700
Elongation at break, %	10 - 25	3 - 5	N/A	N/A	1,5 - 1,9	1,8 - 2,2	3,6	1,2	5,9	9,7	N/A
Water absorption, %	130 - 180	60 - 70	70 - 75	40 - 45	N/A	N/A	N/A	N/A	N/A	N/A	50 - 75

Notes

N/A Properties not readily available or not applicable.

Processed natural fibres

Wood cellulose is the most commonly used natural fibre. It is most commonly obtained using the Kraft process. This process involves cooking wood chips in a solution of sodium hydroxide, sodium carbonate and sodium sulphide. Different grades of wood-cellulose fibre containing more or less of the three main constituents, cellulose, hemicellulose and ligna can be obtained by bleaching.

Wood-cellulose fibre has relatively good mechanical properties compared with many manmade fibres such as polypropylene, polyethylene, polyester and acrylic.

Delignified cellulose fibre can be produced with tensile strengths up to approximately 2,0 GPa from selected grades of wood, and using suitable pulping processes. Fibre tensile strengths of 500 MPa can be routinely obtained using a chemical pulping process and the more common, less expensive, grades of wood.

Using conventional mixing techniques, the amount of fibre that can be incorporated into the cement matrix at low water contents is limited by the capacity of the fibres to be mixed uniformly into the matrix. Fabrication techniques that involve mixing fibre with the matrix at initially high water contents and then using dewatering procedures are therefore effective and common.

Wood-cellulose fibre that has not been delignified can adversely affect the curing of the cement matrix. This is because leaching of sugar and other organic impurities into the cement matrix can retard or completely inhibit cement set. Results obtained from autoclaved wood-cellulose cement composites indicate that such products can be sensitive to moisture content.

Published information on the performance of wood-cellulose fibre composites is conflicting. However Bentur and Mindess[21.7] state "Although the strength and other properties of the cellulose-pulp fibre are inferior to those of many other fibres, such as asbestos, they are highly cost effective. This, combined with their compatibility with processes for producing asbestos cement, makes the cellulose-pulp fibres an attractive alternative to asbestos. As a result of intensive research and development, cellulose-pulp fibres are now used in some places as partial or full replacement for asbestos in cement composites."

21.3 Theoretical considerations

21.3.1 Introduction

For the purpose of this discussion fibre-reinforced concrete (FRC) is defined as a composite made with hydraulic cement, aggregate of various sizes, and incorporating discrete discontinuous fibres.

Fibres currently being used in concrete can broadly be classified into two types.

Low-modulus, high-elongation fibres, such as nylon, polypropylene and polyethylene, are capable of large energy absorption characteristics. They do not lead to strength improvement but help control cracking in plastic concrete and impart toughness and resistance to impact loading.[21.8]

High-strength, high-modulus fibres, such as steel, glass, asbestos and carbon, produce strong composites. They impart strength and stiffness to the composite and, to varying degrees, dynamic properties.[21.8] It is also possible to produce composites with ductile tensile mechanical behaviour, through appropriate matching of the cement-based matrix, fibres and consideration of the interface between these two phases. Such behaviour is brought about by multiple cracking of the matrix, but with the cracks controlled to widths in the micro-range by effective crack bridging. Thereby the tensile resistance can be maintained and even increased despite the multiple fine matrix cracks. This ductile tensile response is ended when fibres break or pull out from one of the cracks, leading to a localised final crack, with reduced, softening resistance.

21.3.2 Fundamentals of fibre reinforcement

Consider a simple composite material containing fibres running through the length of the sample in one direction and assume that fibres and matrix are bonded together so that they stretch equally under load. The law of mixtures indicates that the stress in the composite σ_c, is divided between fibre and matrix as follows: [21.9]

$$\sigma_c = (E_f V_f + E_m V_m)\ \varepsilon_c \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where

E_f, E_m = Young's modulus of fibre and matrix respectively

V_f, V_m = volume fraction of fibre and matrix respectively

ε_c = strain in the composite

Although this equation has to be modified to take into account various factors in practical composites (such as fibres of finite length, random orientation of fibres and flaws

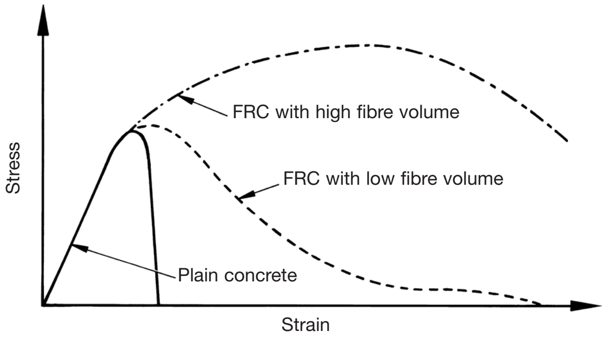

Figure 21.1: Typical stress-strain curves for FRC [21.9]

in fibres) it indicates two factors that are important in the use of fibres. The first is that the fibre stiffness should be substantially higher than the matrix stiffness. The second is that the fibre content by volume should be as high as possible (see Figure 21.1).

In practice, the fibres are incorporated in discrete lengths and load has to be transferred from the matrix into the fibre. Figure 21.2 shows how a relatively stiff fibre modifies the deformation of the matrix, setting up shear stresses in the matrix and at the interface. This results in the tensile stress carried by the fibre rising rapidly with increasing distance from the end of the fibre.

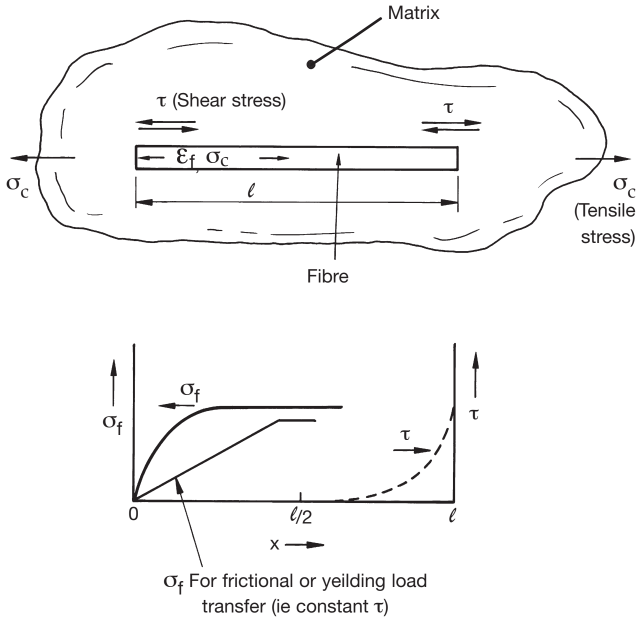

Figure 21.2: Diagrammatic representation of load transfer mechanism[21.10]

Simplified treatments which assume a circular fibre cross section and a constant shear stress value at the interface give equations in the form:[21.11]

$\pi r_f^2\, \sigma_f = 2\pi\, r_f\, \tau x$ .... (2)

where

r_f = radius of fibre

σ_f = tensile stress in fibre at distance x from fibre end

τ = bond stress in shear between fibre and matrix

This equation draws attention to three additional factors that are important for effective fibre reinforcing. They are a good fibre-matrix bond; a sufficient length of fibre; and a high ratio of perimeter to cross-sectional area of fibre $(2\,\pi\, r_f)\ /\ (\pi\, r_f^2)$.

Considering the above, the following rules are applicable for effective use of fibres:

- Fibres should be significantly stiffer than the matrix.
- Fibre content by volume must be adequate.
- There must be a good fibre-matrix bond. However, an upper limit exists for which fibres break, rather that pull out of the matrix, which may lead to brittle fracture.
- Fibre length must be sufficient.
- Fibres must have a high aspect ratio, ie they must be long and thin.

21.3.3 Fibre effectiveness

When examining the effectiveness of fibres, it should be noted that in order to obtain composite properties significantly different from those of the matrix, high volume concentrations of fibres are required. Understandably, much of the published research deals with high volume concentrations. However, for economic reasons, the current trend is to try to minimise the fibre volume to the point at which improvements in properties are sometimes only marginal.

For the quantities of fibres typically used (less than 1% by volume for steel, and about 0,1% by volume for polypropylene for plastic-shrinkage control), the law-of-mixtures approach predicts that the fibres will have no significant effect on the strength or elastic modulus of the FRC, and this is found to be true in practice. It is thus important to evaluate published test data and manufacturer's claims carefully.

21.3.4 Stress-strain behaviour

The idealised tensile stress-strain curve may take one or other of the two basic forms (rising or failing stress tail) shown in Figure 21.3. The extent and distinctiveness of the

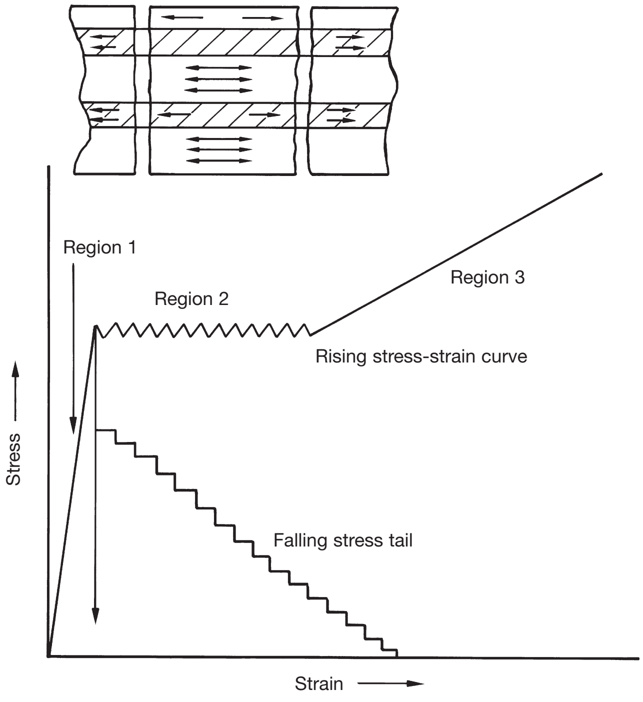

Figure 21.3: Idealised stress-strain curves for fibre-reinforced concretes [21.11]

various regions may differ considerably depending on the particular composite.

In region 1, matrix and fibres act together more or less elastically, the stiffness of the composite and the sharing of the load being governed by the law of mixtures.

Depending on the fibre content, failure of the matrix at the end of region 1 probably occurs at stress levels at or above those which would be attained with well-cured good matrix material. At the end of region 1, the matrix cracks and the behaviour will be very different from that of the unreinforced matrix, depending on fibre content, fibre strength, bond strength and aspect ratio.

If the fibre content and strength are below critical values given by [21.12]

$$K_1 \sigma_f V_f < \sigma_{cc} \quad \text{.......} (3)$$

where

K_1 = factor for fibre orientation

σ_f = tensile stress in fibre

V_f = volume fraction of fibre

σ_{cc} = composite stress when matrix cracks

then the composite will fail suddenly by fibre breakage as indicated by the vertical line in Figure 21.3.

If the criterion given in equation (3) is exceeded, but fibre content, bond strength and aspect ratio are such that [21.12]

$$K_2 V_f \tau \frac{\ell}{d} < \sigma_{cc} \quad \text{.......} (4)$$

where

K_2 = fibre orientation factor

V_f = volume fraction of fibre

τ = bond strength in shear between fibre and matrix

ℓ = fibre length

d = fibre diameter

σ_{cc} = composite stress when matrix cracks

then the composite will fail by fibre pull-out generally in a gradual manner as indicated by the failing stress tail.

If however, fibre content, strength and bond are all sufficient, then the load in the composite is thrown onto bridging fibres (see insert Figure 21.3) and regions 2 and 3 of the rising stress-strain curve result.

Moving away from the crack face, load is transferred back from the bridging fibres to the matrix by shear forces at the fibre-matrix interface, until the stress in the matrix has again risen to such a level that a new crack is formed. This process continues until the material is traversed by an array of very fine cracks which are quite often difficult to see, giving region 2 of the idealised rising stress-strain curve: a region of considerable extension and energy absorption providing toughness for the composite.

As more fine cracks are formed there comes a stage at which it is no longer possible, in the short distance between existing cracks, to transfer sufficient load back into the matrix to raise the stress in the matrix to breaking point. Region 2 is then complete and all further load is carried entirely by the bridging fibres, accompanied by opening of the existing cracks as the fibres stretch. This gives region 3, in which the stiffness and strength of the composite are controlled entirely by the fibre properties, and final failure occurs when the fibres either break or pull out of the matrix.

21.3.5 The concept of toughness

Fibres are added to concrete primarily to alter the post-cracking behaviour of the FRC. How can this improvement be measured? This is perhaps best done by introducing the concept of toughness, which may be defined as the area under the load-deflection (or stress-strain) curve. As may be seen from Figure 21.1, adding fibres to concrete greatly increases the toughness of the resulting material. That is, fibres substantially increase the ability of the concrete to sustain load at deflections or strains well beyond those at which cracking first appears in the matrix.

One way to determining toughness is to determine the energy involved in deforming a specimen to some specified deflection, by measuring the area under the load-deflection curve (in tension, compression, or flexure).

A relative measure of the improvement in toughness involves the comparison of the area under the load-deflection curve for the FRC with that area for the matrix alone.

The most widely used approach, which defines toughness in relative values, is the toughness index. This is defined as the ratio of the area under the load-deflection curve to some multiple of the first-crack deflection, compared with the area under the curve to the point of first cracking. The major problem here is that it is very difficult to determine the onset of cracking, and to obtain an accurate load-deflection plot.

21.4 New developments

21.4.1 High-performance fibre-reinforced cement-based composites

A development of the last few decades has been significant research activity and increasing application of high-performance fibre-reinforced cement-based composites (HPFRCC). This has led to design recommendations being proposed for these materials recently in Japan.[21.16, 21.17] Particular classes are ultra high performance (UHPFRC)[21.14, 21.15] and strain-hardening (SHCC) fibre-reinforced cement-based composites.[21.2, 21.3] These composites are designed for particular applications varying from the requirement of high strength to that of high ductility. For instance UHPFRC have been designed for and applied in thin bridge decks or bridge deck overlays, with compressive strengths in the range 120 to 180 MPa and flexural strengths in the range 20 to 40 MPa. On the other hand, the requirement of energy dissipation in earthquake-resistant buildings has led to the use of highly ductile SHCC in coupling beams of cores of high rise reinforced concrete buildings in Japan. Other uses of SHCC include direct exploitation of its tensile deformability in bridge deck movement joint replacement, and protection of reinforced concrete structures by its multiple, fine cracking nature, which significantly retards the ingress of moisture, gas and chlorides. An example of this application is the thin SHCC overlay of an existing dam face.

21.4.2 Textile concrete

Textile concrete is the use of woven or welded textiles to act as embedded reinforcing in a concrete system. Normally it is used in thin-walled elements such as permanent forms, decorative cladding units, containers, signs, and the like.

Fabrication of these systems is normally by immersing/saturating the textile in mortar and placing several layers over a form or mould. Pressure can be applied to remove excess water and/or to achieve a given finish on the surfaces.

The advantage of this approach is that the fibre can be used at far higher concentrations than achievable in traditional wet-poured approaches. The fibre can also be orientated in the direction of applied stresses, making it more efficient. The cross fibres in the fabric help to tie back the fibres straddling cracks, thereby reducing the amount of pullout that occurs, significantly increasing the toughness of system. (See Figure 21.4).

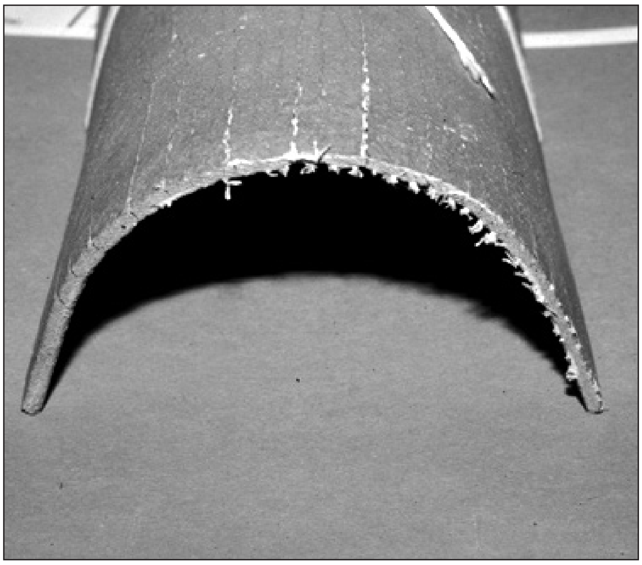

Figure 21.5: Curved sheet of textile concrete
Photograph courtesy of Dr P. Taylor

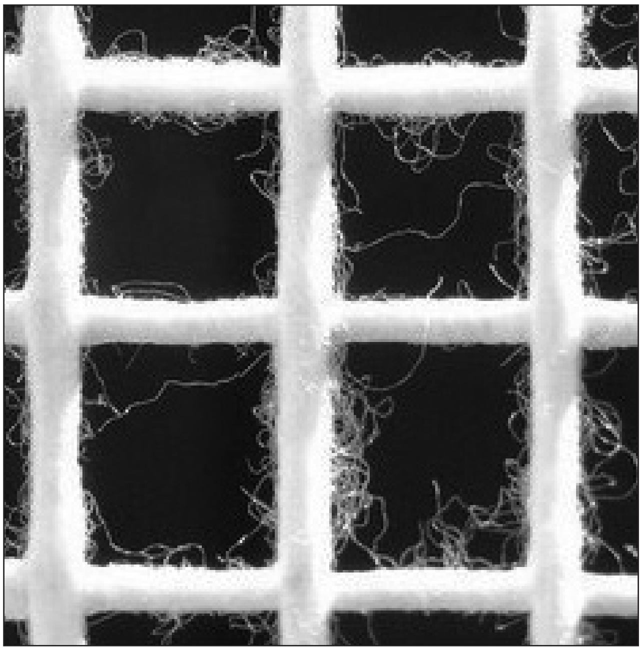

Figure 21.6: Example of textile reinforcement.
Photograph courtesy of D. Hourahane

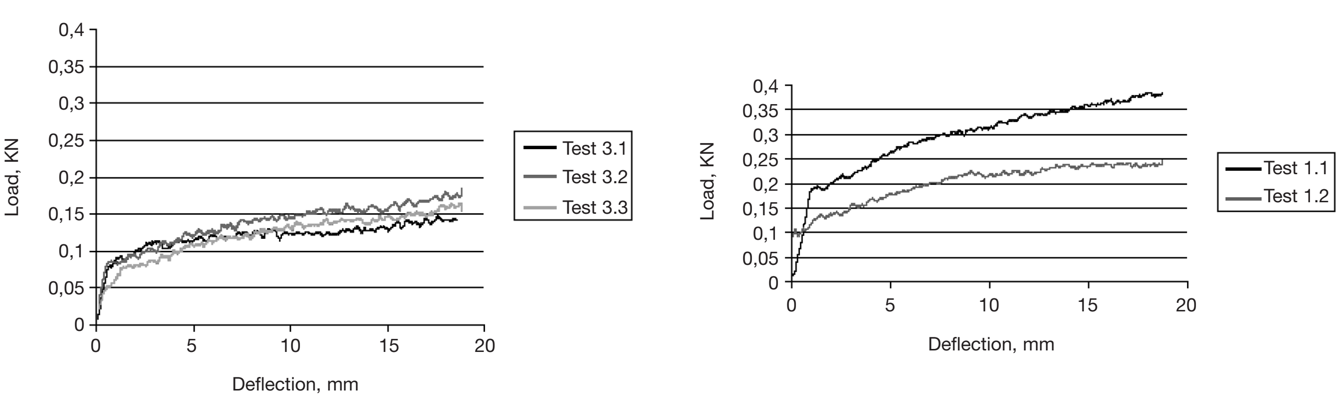

Figure 21.4: Deflection versus load for bonded and unbonded textiles [21.18]

Systems fabricated using this approach can effectively be "flexible" with deflections of up to 5% (see Figure 21.5). The matrix cracks, but the fibres prevent the system from falling apart, while distributing the cracks so that there are a large number of relatively small cracks rather than a single large crack.

Commonly used fibres include polypropylene, cellulose, polyethylene, glass fibre and sometimes carbon fibre, alone or in combinations. Many of these fibres are not prone to corrosion, therefore potentially improving the durability of systems containing them. Figure 21.6 shows a locally available product which uses a compound system that comprises a core fibre shrouded with a fluffy coating that can be periodically bonded to the core fibre. This has the effect of increasing bond between the paste and the core fibre. This is beneficial when using materials that are hydrophobic or exhibit poor bond with cementitious pastes.

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Chapter 22

Precast concrete products

John Cairns

22.1 Introduction

The aim of this chapter is to describe briefly the characteristics and manufacture of precast concrete products and to give guidelines for the selection of materials and proportioning of mixes. Scope is limited to manufacture by mechanical means; moulding by hand is not discussed.

22.2 Masonry units and paving blocks

Concrete masonry units are used extensively for walling in lieu of more traditional units such as stone and burnt clay brick. Concrete masonry units have properties that differ from those of other masonry units and of conventional concrete.

Concrete block paving consists of small concrete elements laid close together on a prepared formation to provide a structural layer, which:

- Accepts a range of traffic types
- Can carry imposed loads
- Is durable
- May be laid using attractive colours and patterns

Both concrete masonry units and concrete pavers are made on similar equipment using semi-dry concrete mixes.

22.2.1 Characteristics

Masonry units

A concrete masonry unit is a preformed building element of rectangular shape, of brick or block size, either solid or hollow, formed from a mixture of cement, aggregate and water, and used in the construction of walls, piers, retaining walls, etc.

The units are made in a range of sizes, shapes, colours, textures and profiles, and are designed to meet various requirements when used in walls such as strength, thermal and acoustic criteria, resistance to fire and aesthetic appearance.

Concrete masonry units are normally slightly less dense than conventional concrete as a result of the manufacturing process employed. Nevertheless, good-quality units have adequate strength for normal use, and a surface texture on bedding faces for the development of good bond with mortar.

When selecting units for a project, the appropriate unit should be used with a view to cost and desired properties.

The standard specification for concrete masonry units is SANS 1215.[22.1] This covers physical requirements such as compressive strength, dimensional tolerances, drying shrinkage, moisture expansion and appearance.

Units are available in a wide range of compressive strengths ranging from 3,5 MPa (hollow units) and 7 MPa (solid units) to 21 MPa. Strength is maximum load divided by gross or overall area. Generally units with strength above 10,5 MPa are solid.

Shrinkage of units results from loss of moisture and carbonation and should not exceed 0,06%. (Units, once curing is complete, should be allowed to dry out and should be laid in a dry state in order to reduce possible shrinkage in the wall.)

Moisture expansion, which may occur when crushed clay brick, some slags and clinker are used as an aggregate, should not exceed 0,02%.

Tolerances, squareness, surface texture and appearance are also covered by SANS 1215.[22.1]

One of the important characteristics of concrete bricks and blocks is the close tolerances to which units can be made. This means that fair face surfaces can be obtained on both sides of a single-leaf wall.

The range of face masonry units available will differ considerably from one manufacturer to another, depending on local needs and building practice. Units of various colours, textures and profiles ranging from plain, close-textured faces to split-faced, exposed-aggregate and fluted surfaces are produced.

Acoustic, thermal and fire resistance properties of concrete masonry depend on the thickness of the units, the density of concrete, the quality of aggregates and other factors.

Concrete paving blocks

Paving blocks (or pavers) are made with materials, and on equipment, similar to those used for concrete masonry units.

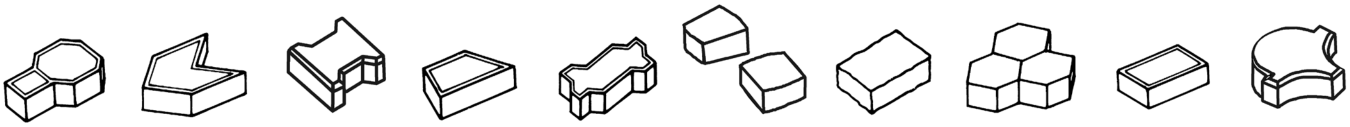

Figure 22.1: Examples of shapes of pavers

Paver shape (see Figure 22.1) is either:

- Dentated
- Rectangular
- Square
- Curved

Typically, pavers of dentated shape are between 200 and 250 mm long, with widths between 100 and 112 mm and thicknesses normally range from 50 to 80 mm with occasional thicknesses of 100 and 120 mm. Pavers of rectangular or square shape in plan may be larger in size.

Pavers are made in various sizes, shapes, colours, textures, and with and without chamfers. To reduce cost of manufacture and to improve performance in service, pavers are often made with a base of lower strength and a topping of greater strength. The topping may be pigmented.

The topping surface may be based on visual requirements such as colour and texture or on high resistance to abrasion.

The technique of using a topping has been in use since the late 1970s and has proved to be satisfactory, provided sound materials are used in appropriate concrete mix proportions with the right manufacturing techniques.

The standard specification for concrete paving blocks is SANS 1058.[22.2] This covers:

- Physical requirements such as strength (class) and abrasion resistance
- Water absorption
- Dimensional tolerances
- Chamfers and general appearance, eg free from cracks and other defects

Pavers are specified in two strengths, referred to as "class of block."

Three shapes of block are specified. Type S-A allows geometrical interlock between all vertical faces of adjacent blocks, Type S-B allows geometrical interlock between some vertical faces of adjacent blocks while Type S-C allows no geometrical interlock between the vertical faces of adjacent blocks.

22.2.2 Manufacture

The technology for the manufacture of preformed concrete units, ie blocks, bricks and pavers differs from that of conventional concrete in that:

- The fresh mix has a relatively dry crumbly consistence. (But as much water as possible should be used without creating production problems.)
- The units are manufactured in a mould under high compactive forces using high-frequency vibration and/or pressure.
- The units are demoulded within a few seconds after compaction and the shape is maintained by the interlock of the aggregate, capillary forces and surface tension.
- Units are manufactured on *block machines.*

Machine operation

After the concrete has been distributed in the mould, which usually accommodates more than one unit at a time, it is compacted with vibration and/or pressure, and the moulded units are ejected, either onto a pallet (a stationary machine) or onto a prepared concrete slab (an egg-laying machine). There are generally five separate operations in the cycle; these are described below. Cycle time is generally between 10 and 30 seconds though pavers cycles are usually faster due to the large area and lower depth of mould to be filled with concrete, vibrated and extruded.

Feeding loose concrete into the mould

In automatic machines, concrete is placed in a feeder box that passes backwards and forwards over the mould to fill it. The method of feeding the concrete into the mould does not always give an even distribution; often the outer sides and the end away from the supply of concrete receive less material than the other parts. Because all units are compacted to the same final depth controlled by the movement of the compacting or tamper head, these units in outer areas of the mould may differ in density from the rest. Thus, important properties of units may therefore differ in a single mould "drop." Particular attention must therefore be given to feed arrangements, and units should be tested to determine the variability of the products made within one mould. This may be done by weighing units and comparing results. Moulds for solid units are easier to fill than moulds for hollow units.

With pavers that are to be topped, the mould is filled with base concrete to 5 to 10 mm below the top of the mould, and a feeder box containing the topping mix then passes over the mould. Because of the large plan area of the paver, moulds are quickly and easily filled.

Preliminary vibration

During preliminary vibration, the loose concrete in the mould is compacted. The degree of compaction depends on how long it is vibrated. The period of vibration ranges from tenths of a second to ten seconds. The longer the vibration period, the greater the amount of material that needs to be fed into the mould. The uniformity of this initial compaction depends very much on the uniformity and the workability of the concrete fed into the mould.

Second filling of the mould

This is a topping-up process with some vibration to bring the concrete flush with the top of the mould. With pavers, the topping up process may involve a stronger pigmented concrete mix.

Compaction of concrete in the mould

The compacting or tamper head is brought into contact with the top of the concrete and the concrete is compacted under vibration and/or pressure. The plan dimensions of the units reflect the accuracy of the mould dimensions while the height as made is sensitive to the operation of the head and the level to which it is allowed to fall; this is controlled to limit differences in height or the degree of compaction.

Extrusion of units from mould

When units are extruded from the mould, the mould can be lifted while the compacting head remains static, or the compacting head can push the units out of the mould while the latter remains static. The relative movements of the compacting head and mould must not cause any cracking or deformation of the units.

Newly de-moulded units are easily damaged. The movement of pallets should therefore be smooth to avoid either racking (jarring) or vibratory movements. The thickness of the slab on which an egg-laying machine operates must be such that only a minimal proportion of the vibration used to compact one drop of units is transmitted to other recently de-moulded units.

The depth of travel of the tamper head is controlled, either at a certain height, or by duration of vibration. In the former case, the height of the units will be reasonably accurate, but concrete densities will differ if the amount of concrete fill in the mould differs. In the latter case, the density of the units will be constant, but their heights may differ.

22.2.3 Development of strength

Various stages at which strength of the units is critical are:

- At de-moulding: Because the units are extruded from the mould within seconds of compaction, and the hydration of the cement at this stage does not contribute towards the unit strength, the strength of the unit is dependent on the interlock between the aggregate particles, capillary forces and surface tension. (It is for this reason that a fairly dry mix is used.)
- At de-palletising or removal of units from slab: Stresses in handling of units with inadequate strength may cause breakages, as well as chipped corners and arises.
- At delivery: The strength of the units at delivery should meet the requirements of SANS 1215 *Concrete masonry units*[22.1] or SANS 1058 *Concrete paving blocks*.[22.2]

22.2.4 Water content

The amount of water added to the mix depends on aggregate characteristics, the mix proportions and the type and operation of the block machine. In general, as much water should be used as possible without creating production problems and causing the units to slump and distort after they have been extruded from the moulds.

Dry mixes will usually be difficult to compact fully because of high internal friction and resistance to movement, and will produce units of high porosity, low green strength and low compressive strength. Over-cohesive or wet mixes may tend to stick to the moulds and be difficult to extrude and/or stick to the compacting head. It has also been noted that with wetter mixes the units made by egg-laying machines tend to stick to the concrete surface on which they are laid, and difficulty is experienced in removing the units the following day without causing breakages.

The final adjustment and control of the water content of the mix is usually done by eye and is normally between 6 and 9% of the mass of the green units.

A rough indication of the optimum moisture content of the mix in the mixer can be obtained by taking a handful of the mix and rubbing it with a 16-mm-diameter smooth steel bar. If the concrete shows water sheen, and ripple marks of moisture are evident on the bar, the moisture content is about right. Also when units are extruded there should be water sheen on the vertical sides of the unit.

Probably the most common fault in manufacture is to make the mix too dry. If the fines content of the mix is high, the mix becomes sticky before the optimum moisture content is reached. In such cases, adjust the aggregate grading and the mix proportions until the stickiness is eliminated. This is particularly important in the manufacture of high-strength units where the increased cement content contributes to the stickiness of the mix.

22.2.5 Mix proportions

The final quality of the unit is affected by the:

- Combination of aggregates
- Ratio of blended aggregate to cement

- Size and shape of the mould
- Thickness of the shell and web of hollow masonry units
- Amount of water
- Mixing time
- Compaction capabilities of the machine
- Degree and extent of curing

It is not possible to determine the final proportions of a mix theoretically or in a laboratory to meet specific requirements. A thorough investigation on actual production should be carried out to assess all the variables affecting unit quality. Nevertheless, samples of aggregate should be submitted to a recognized laboratory for an assessment of their physical and chemical properties.

With masonry units, the cement:aggregate ratio by mass may range from 5 to 20%.

With pavers, the base concrete mix for a Class 2 paver has a cement:aggregate ratio by mass of 14 to 18% while the topping mix ratio is 18 to 23%.

Pigments may be used to colour the concrete. The final choice of pigment and the appropriate dosage will be made on a visual examination of dry concrete in which the pigment has been used. Usually, in the preliminary stages, concrete is made with a range of dosages of pigments to determine the optimum dosage for the required colours. Generally dosages of pigment, by mass of cement, are from 3 to 5% with a maximum of 9%.

Admixtures may be used in the concrete to improve the properties of the fresh and/or hardened concrete. The type of admixture to be used should, in the first instance, be based on the manufacturer's advice. The final decision should be based on trial mixes and cost comparisons.

22.2.6 Determining the best blend of aggregates

Knowing the basic properties of the aggregates, viz. consolidated bulk, loose bulk and relative density and grading, a first estimate of the likely blend of the aggregates is made.

The first estimate of blends can be based on a number of factors, or combinations, thereof, such as:

i) Minimum voids of the aggregate blend. This is determined from laboratory tests of the consolidated bulk density of the blend from which the solid content, and thus the voids is calculated.

   The voids content of a blend reflects all the important characteristics of aggregates, viz. particle shape, grading and surface texture, which affect the aggregate's performance in concrete.

   A concrete mix based on minimum voids in the aggregate blend ensures a dense concrete that is economical in cement and cement-paste requirements.

ii) Reference to grading limits, some of which are shown in Table 22.1. Grading limits take account of particle size distribution only and do not describe particle shape and surface texture, which are important characteristics in assessing aggregates for semi-dry concrete.

   The aggregate grading for the base concrete of pavers is as for normal units in Table 22.1.

Table 22.1: Grading limits

Sieve size, mm	Percentage passing			
	Normal units	Target grading split face masonry units, texture		
		Coarse	Medium	Fine
19,0	100			
13,2	90 - 100			
9,5	75 - 100	100	100	
4,75	60 - 85	80	95	100
2,36	40 - 65	60	85	95
0,300	10 - 25	15	25	30
0,150	5 - 15	4	15	15
0,075	2 - 10			
Fineness Modulus	3,2 - 4,2	3,65	2,75	2,5

   The maximum aggregate size for the topping mix for pavers will depend on the finish required: 3 mm is most common; 4,75 mm is used for some products.

iii) Reference to fineness modulus. An estimate of initial blend is based on a target fineness modulus – usually around 3,7 for coarse textured solid masonry and paving units, and around 2,8 for fine-textured hollow units. A check on the blend for material passing the 0,300-, 0,150- and 0,075-mm sieves is made and the blend adjusted.

iv) Reference to arbitrary proportions of various fractions of coarse, medium and fine materials. A rough classification of these fractions is:

• Coarse (chips 9,5 or 6,7 mm)	25 to 40%
• Medium (coarse river sand)	35 to 70%
• Fine (plaster or building sand)	5 to 20%

Weighing the unit while green will give an indication of its quality: heavier units indicate denser compaction and are usually stronger.

The surface texture of the units should be noted for each blend. If the texture is too smooth, some fines should be removed from the blend. If the texture is too coarse, more fines should be added.

22.2.7 Curing

The type and extent of curing depends primarily on the overall manufacturing process and equipment available. It ranges from low-pressure steam curing in special curing chambers to no curing at all. Curing will assist units to gain strength faster and ultimately achieve greater strength. Green units should be protected as soon as possible from drying out. Curing should commence when there is no likelihood of units being damaged by the particular curing process. With low-pressure steam curing, there is a delay period of three or more hours before steam is introduced. Temperatures above 65ºC are not advisable.

Mist water spraying of units, especially if intermittent, may result in subsequent efflorescence. With coloured units, water curing is not recommended, but units should not be allowed to dry out as this might also cause efflorescence. Plastic shrink-wrapping prevents units in wrapped cubes from drying out.

The type and extent of curing affects the colour of units, better curing resulting in lighter colours.

Curing of the topping mix of pavers is particularly important to ensure high resistance to abrasion when in service.

22.3 Concrete retaining blocks

Concrete retaining block (CRB) units are widely used for the construction of retaining walls by dry stacking them on top of each other, usually at a slope of 70º to the horizontal. They are far more economical and easier to build than vertical reinforced concrete retaining walls.

The design of a CRB Wall should always be undertaken by a qualified engineer.

22.3.1 Characteristics

There are various shapes and sizes of CRB units with the criterion being that they can be handled and positioned by one person. A variety of colours and textures are available. In many cases after constructing the wall, the holes of the units are filled with soil and suitable plants are placed in the earth-filled cavities. Within one or two years the retaining wall is no longer visible, making for an aesthetically pleasing and environmentally friendly walling solution.

The standard specification for CRB units is SANS 508.[22.3] This covers physical requirements such as strength, dimensional tolerances and general appearance. Notwithstanding the fact that units comply with the requirements of the standard, the engineer should always satisfy himself that the unit is suitable for the particular application.

22.3.2 Manufacture

The units are manufactured by two methods. Most commonly they are produced on machines in the same manner as concrete masonry units and paving blocks.

However if the shape of the CRB unit does not allow for this method of manufacture, they are then produced in individual moulds upside down and then rotated onto a pallet and the mould removed – rather like making a sand castle.

22.4 Precast concrete suspended slabs

Precast concrete suspended slabs can be used for a variety of applications from domestic housing to high-rise buildings and industrial structures. They can be supported by structural masonry, in-situ concrete beams or steel beams.

The benefits of using precast concrete slabs include:

- Reduced slab weight due to voids and/or prestressing
- Faster erection
- Minimal propping
- Guaranteed quality

22.4.1 Characteristics

There are two major types of precast concrete slab systems:

- Hollow-core (reinforced and prestressed)
- Prestressed beam-and-block

The hollow-core slab is a complete precast slab with cores that are either circular or elliptical. They may be reinforced or prestressed, and the cores afford a reduction in self-weight of 30% or more when compared with a solid slab of the same depth.

The beam-and-block system comprises prestressed rectangular precast concrete beams and non-structural hollow concrete blocks, which form voids in the slab. A light steel mesh is placed on top of the blocks and structural in-situ concrete with a minimum strength of 25 MPa is poured on top of the beams and over the top of the blocks to create a series of T-beams. Nominal propping at 1/3 span points is required with this system, before the in-situ concrete is cast.

The standard specification for precast concrete suspended slabs is SANS 1879.[22.4] This covers physical requirements such as dimensional tolerances, serviceability limit state and ultimate load of beams and hollow core slabs, and construction loads of blocks.

22.4.2 Manufacture

Reinforced hollow-core slabs are manufactured in fixed moulds. They are generally 900 mm wide and 150 mm deep, incorporating a number of 72-mm deep cores.

Units are made in a range of lengths up to about 7,8 m. They are generally manufactured for stock and then cut to suit the specific contract.

The reinforcing steel is positioned in the mould and the concrete, which is a fairly dry high-strength mix, is poured into the moulds and vibrated.

The core-formers and side shutters are removed and the slab is put into a steam chamber for curing.

Prestressed hollow-core slabs are manufactured on long-line continuous beds generally between 100 and 150 m long. The two methods of manufacture are extrusion or slipform.

The slabs are generally 1 200 mm wide with depths of 120, 150, 200 and 250 mm. Units are made to order in lengths of up to 12 m. The number and positioning of the prestressing tendons varies according to span and loading. The slabs are produced in a continuous length and then saw cut to length after initial curing.

The manufacturing machine moves continuously along the bed and is filled with dry, high-strength concrete from a central batching plant. The machine levels and vibrates the concrete and the core-formers are incorporated in the machine. It is possible to stand on the fresh concrete immediately behind the machine (even above a core) without causing any collapse of the concrete.

Cantilevers can be accommodated in both reinforced and prestressed hollow-core slabs. For reinforced slabs longitudinal top reinforcement can be cast in, and for prestressed slabs the tops of a number of cores are opened for a specific length, the reinforcing steel is placed and the cores filled with concrete.

The concrete strength for hollow-core slabs is around 50 MPa and the manufacturing process requires at least 30 MPa at detensioning for prestressed slabs. Slabs can be cast and delivered to site within a few days.

The soffits of hollow-core slabs are smooth as they are produced on a steel base and do not therefore require plastering. The top surface is roughened to receive either a screed or structural topping, which is sometimes required to increase the effective depth for superimposed loads.

Hollow-core slabs are a generally a complete slab (except where a structural topping is required) and require no propping or additional strengthening once placed. However, because of the weight of the slabs, they require a crane for erection. The longitudinal edges of the precast units are designed and profiled to receive grout in the joints and create a shear interlock, which provides load transfer between adjacent slabs and prevents differential deflection.

The prestressed beams for the beam-and-block slab system are generally produced in a long-line bed of up to 150 m in length in fixed moulds. The number of prestressing wires can vary to accommodate the span and loading. The beams can be produced continuously and saw cut to length after curing, or stop ends can be inserted along the line to the exact requirements of the project. The concrete strength of the beams is around 50 MPa.

The hollow in-fill blocks, which are non-structural but simply void formers, are produced in exactly the same way as hollow masonry blocks. The height of the blocks can range from 60 to 350 mm to give an overall slab depth from 110 to 400 mm. The blocks are rebated so that they “sit” on the beams. The only structural requirement of the blocks is that they must be able to support the weight of a person and a wheelbarrow full of concrete during construction.

The beam and block system can generally be economical up to a 6-m span and can be designed as a simply-supported single span or as a continuous slab by providing longitudinal reinforcing steel in the topping over internal supports.

The units are light enough to be erected manually so that no crane is required for erection and complicated shapes can easily be accommodated.

22.5 Concrete pipes

Concrete pipes are the most frequently used and accepted product for stormwater drainage, outfall sewers and many other applications.

There are currently two South African national standards applicable to concrete pipes:

- SANS 676: 2003 - Reinforced concrete pressure pipes[22.5]
- SANS 677: 2003 - Concrete non-pressure pipes [22.6]

Pipes are classified in terms of their crushing strength when subjected to a vertical knife-edge test-load.

Pipes are generally produced with external diameters ranging from 300 to 1 800 mm and in four pipe classes namely 25D, 50D, 75D and 100D depending on the loading and application.

22.5.1 Manufacture

There are five basic methods of producing concrete pipe. Four of the methods use mechanical means to place and compact a dry concrete mix into the form. The fifth method uses a more conventional wet mix and casting procedure.

The five methods are:
- Centrifugal
- Dry cast
- Packerhead
- Tamp
- Wet cast

Centrifugal or spinning process

The centrifugal or spinning process uses an outer form that is rotated in a horizontal position during the pipe making process. Vibration and compaction can be used in combination with centrifugation to consolidate the concrete mix. While the mix is wetter than some pipe mixes, water is extracted from the concrete by the centrifugal forces, which develop as the pipe is spinning. As the form is rotated concrete is fed into the form by a conveyor system that is capable of distributing concrete throughout the form length. The finished pipe, still in the form, is moved to the curing area and cured in the form.

Dry cast process

There are several variations of the dry cast process, but all use low frequency-high amplitude vibration to distribute and densely compact the dry mix in the form. The form is removed immediately as the newly formed pipe can support itself. To ensure that the desired vibration reaches all points, different techniques are utilised.

In one method, vibrators are attached directly to the exterior form. The mix is fed into the form and the vibrators are operated at various stages during this process. On completion of the process the pipe and form are lifted off the machine and moved to the curing area where the form is removed.

Another variation of the dry cast method has a central core that moves up and down and provides vibration and compaction. In this method when the process is completed the core retracts and the pipe is stripped and moved to the curing area.

Packerhead process

The packerhead process uses a device rotating at a high speed that forms the interior surface of the pipe. It is drawn up through the exterior form as mix is fed in from above. The head has rollers or deflectors mounted on the top, which compact the mix. When compaction is complete the form and pipe are moved to a curing area where the exterior form is removed.

In some packerhead processes a vibrating core follows the packerhead through the pipe-making sequence. The core is mounted in a pit below the pipe machine and is retracted before the pipe is moved.

Tamp process

The tamp process uses direct mechanical compaction to consolidate the concrete mix. Inner and outer forms are placed on a rotating table and the concrete mix is fed into the forms. As the form is rotated and filled the tamper rises automatically. There are usually multiple tampers so that the mix on each side of the reinforcement can be compacted. The pipes are removed from the machine with either the inner or outer form and moved to the curing area where the form is removed.

Table 22.2: The effect of processes on pipe properties

Property	Horizontal casting	Vertical casting
Flow capacity	For high strength pipes wall thicknesses are increased by adding material to the inside of the bore. This reduces the actual diameter below the nominal diameter of the pipe.	Vertically cast pipes have a constant diameter irrespective of strength requirements.
Load carrying capacity	Can produce pipes capable of carrying all loads specified in SANS 676 and 677	
Sewer corrosion	• Additional material in the form of a sacrificial layer can be added inside the bore of the pipe. • Methods have been developed for the incorporation of an impervious PVC lining to the bore of the pipe. • Sacrificial material cannot be added, the design of the pipe must be changed to make use of more steel and a thinner wall.	Linings cannot be incorporated
Joint compatibility	• Centrifugal action forces the bore of the socket away from the former, and therefore for joints requiring close tolerances, the socket must be ground. • The difficulty of casting concrete into the socket section can lead to density problems in this zone. Sockets are very well compacted and accurately formed.	
Material utilisation	• Makes optimum use of the potential to vary wall thickness. • It is not possible to vary wall thickness and for this reason the lighter load range makes use of more material.	

Wet cast process

Wet casting of concrete pipe, as the name implies, uses a concrete mix that is wet relative to the mixes used in the other processes. The mix usually has a slump of less than 100 mm. The wet cast process is most commonly used for production of large diameter pipe which is manufactured, cured and stripped at one location. Inner and outer forms are most commonly mounted in a vertical position but some pipe is cast in horizontal forms. The latter is called prebed pipe because the exterior pipe wall has a flat side where the form is open to allow filling.

With vertical forms, a cone attached to the inner form is used to direct the concrete mix. The mix is transported to the form by crane conveyor or similar equipment. As the mix is placed in the form it is vibrated using internal and, in many cases, external vibrators. After the form has been filled the cone is removed and the pipe cured in the form. This is frequently done using canvas tents and steam. Following the curing period, usually overnight, the forms are removed and the pipe is moved to the storage area.

The effects of casting pipes in either the horizontal or vertical positions on their properties are summarised in Table 22.2.

22.6 Concrete roof tiles

Over the past 30 years concrete roof tiles have revolutionised the roofing industry. Their success can be attributed to their consistent high quality, durability, affordability, aesthetic appeal and ease of installation.

Due to their competitiveness and their acceptability amongst specifiers and end-users, concrete roof tiles are increasingly being used in mass housing projects - a market previously dominated by fibre-cement and metal roof sheeting.

Ongoing research and technical innovation have made concrete roof tiles a true example of a product that has evolved to meet market needs and to exploit potential new applications.

22.6.1 Characteristics

Concrete roof tiles are produced in a range of profiles and colours. The most common profile is Double Roman, and there are also flat tiles and more bold profiles.

Pigmented colour is used for the whole tile so that the colour is uniform throughout the body of the roof tile. Only synthetic iron oxides are recommended for use in concrete roof tiles in order to ensure colour uniformity, tinting strength and availability.

Mineral granules are used for textured surface finishes. They stabilise the coloured pigment slurry giving it improved resistance to climatic and atmospheric influences. South Africa has an abundance of natural coloured rock from which these mineral granules are produced.

The standard specification for concrete roof tiles is SANS 542,[22.7] which covers the physical requirements such as dimensional tolerances, transverse strength and permeability.

22.6.2 Manufacture

Concrete roof tiles are produced on highly specialised single-purpose plants in a large variety of profiles, colours and finishes.

To meet quality, dimensional accuracy and weathering stress requirements, concrete roof tiles are the product of exact precision in moulding, material formulations and knowledge of concrete technology.

The manufacturing process, including the various aspects of raw material selection, processing, manufacturing, quality control and distribution, must be viewed as an operation.

Tile production

The key to successful production is the design and control of the flow system, which feeds the machines. These systems require careful preparation and planning, ingenious engineering, synchronised control and constant monitoring, which is often computerised.

The production line operates on a closed-loop system whereby on the one side wet tiles are produced and on the opposite side dry tiles are depalleted from the moulds, which are returned via conveyors and turntables to the tile machine, making the whole operation a continuous process.

Unlike ceramic and clay tiles, concrete roof tiles are not pressed but are extruded on top of intricate aluminium pallets, which shape, form and support the green (wet) tile.

To produce a tile, pallets are fed as an endless band through an oiling unit into the tile machine. The tile machine pushes the pallets into the extrusion box, which receives the concrete mix from the top. Here the forming and compaction of the fresh concrete takes place under high pressure by means of a rotating paddle, shaft, profile rollers and profiled extrusion slippers, which give the tile its final shape and smooth surface.

The compacted and profiled concrete exits the extrusion box on top of the supporting pallets as an endless ribbon to be cut into single tile lengths by synchronised knives at the joint of the individual pallets. Extremely close tolerances are maintained to ensure that every tile is identical to its predecessor.

In the case of granular tiles, the green tiles supported on their aluminium pallets then pass through an automatic slurry and granular applicator before being collated, racked and transported into specially constructed curing chambers where they remain for a minimum of eight hours.

In the curing chamber, temperature, humidity and the time cycle of curing and drying are closely controlled. Curing temperatures are ± 40ºC and the humidity is generally kept at 95%.

Through the mechanism of hydration, sand and cement become tightly enmeshed in a gradual and permanent increase in compressive and tensile strength. Concrete roof tiles will continue to gain strength as they age.

At the end of the curing cycle the tiles are taken out of the curing chamber and go through the process of deracking and depalletising.

In the automatic depalleter the dry tiles are mechanically separated from the pallets by means of rotating knives. The aluminium pallets are returned to the tile machine for re-use, and the cured tiles receive their final surface treatment before being conveyed into the stacking yard for final curing of between seven and 21 days before being loaded for delivery to their final destination.

Further reading

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22.5 SANS 676:2003 **Reinforced concrete pressure pipes,** Pretoria: South African Bureau of Standards, 2003.

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Chapter 23

Shotcrete

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23.1 Introduction

Shotcrete is concrete conveyed through a hose and pneumatically projected at high velocity onto a surface. The force of the jet impacting on the surface compacts the material. Shotcreting is done by using either the “dry-mix” or the “wet-mix” process. With the dry-mix process, a relatively dry mixture is generally used and the material is capable of supporting itself without sagging or sloughing. With the wet-mix process, concrete with pump-mix consistency is used. Both systems can be used for vertical and overhead applications, provided that the layer thickness is either limited to about 75 mm or special admixtures are used.

Shotcrete has been referred to by such terms as pneumatically applied concrete, sprayed concrete, spraycrete, air-blown concrete (or mortar), gunite and gunned concrete. In South Africa, the term shotcrete implies the use of a mix containing coarse aggregate, up to about 12 mm in size. Gunite is the term used for a sand-cement mortar with maximum aggregate size limited to about 6 mm. In this chapter, the term shotcrete is used in a generic sense, ie irrespective of aggregate size, unless stated otherwise.

Shotcrete is suitable for various types of new construction, repair work and fireproofing[23.1, 23.2] of structures as well as in underground support systems. However, its properties and performance are largely dependent on the conditions under which it is placed, the capability of the particular equipment selected, and especially on the competence of the application crew.[23.3]

Research and development is ongoing, mainly in the fields of local aggregates, cementitious materials, retarders, accelerators, early strength development and fibre-reinforced shotcrete (FRS) as well as leaching behaviour and environmental impact of shotcrete. These developments have been reported in papers presented at a number of shotcrete conferences held at the University of Innsbruck in Austria.[23.1, 23.2, 23.4, 23.5, 23.6]

Some experimentation with equipment such as the use of peristaltic and worm-type wet-mix shotcrete machines is also being undertaken. Dust reduction for dry-mix shotcreting is an area of research that still requires attention.

This chapter provides a brief overview of shotcrete practice in South Africa. Shotcrete uses, processes, and materials as well as design and some important aspects of manufacture, placing, testing and properties are addressed. Future trends and developments in local shotcreting are not detailed, other than to note increasing use of (retarded and reactivated) wet-mix shotcrete, as opposed to dry-mix application, and FRS, made with steel or polymeric fibres.

23.2 Uses of shotcrete[23.3, 23.15]

Shotcrete offers advantages over conventional concrete in certain applications.

It is frequently more economical than conventional concrete because: thin layers and/or variable thicknesses can be placed resulting in reduced volumes of materials; formwork is not required; and only a small portable plant is needed for manufacture and placement in the case of the dry system. High compressive strengths can be achieved with shotcrete. Shotcrete is capable of excellent bond with a number of materials and this may be an important consideration.

The uses of shotcrete include:

- New structures: roofs (particularly curved or folded sections), thin shells, walls, prestressed tanks, reservoir linings, canal linings, swimming pools, boats, tunnel linings, sewers, shafts and other shaped sections.
- Support, both temporary and permanent, of tunnels, mines, slopes, basements, etc
- Coating over brick, masonry, concrete, rock and steel to achieve longer fire protection or a better fire rating.
- Encasement of structural steel for fire-proofing
- Strengthening of concrete slabs and concrete and masonry walls
- Repair of deteriorated concrete in structures such as bridges, culverts, sewers, reservoir linings, dams, tunnels, shafts, elevators, waterfront structures and pipes and others
- Repair of earthquake and fire damage to masonry and concrete structures
- Refractory linings to stacks, furnace walls, boilers and other structures

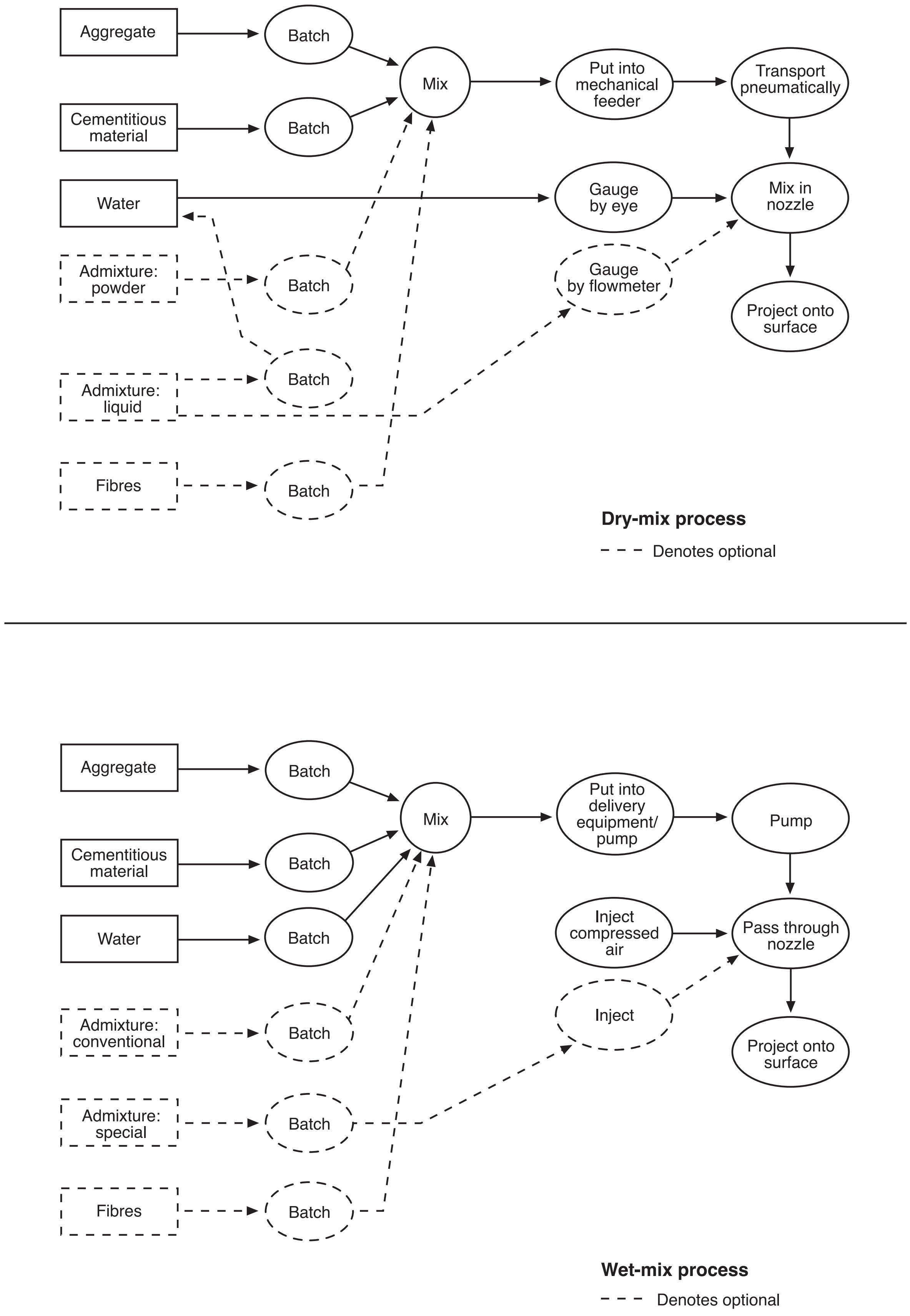

Figure 23.1: Flow charts of shotcrete processes

23.3 Shotcreting processes

There are two basic shotcreting processes: dry-mix and wet-mix.[23.3] These are described below and shown diagrammatically in Figure 23.1.

23.3.1 Dry-mix process

This process consists of the following steps:

1. Cementitious material and dry aggregates are thoroughly mixed. (A total water content of about 3% has been found to reduce dust emissions, but not eliminate them. Some of the recently developed dry-mix shotcrete systems can handle up to 12% moisture).
2. The mixture is fed into a special mechanical feeder or gun.
3. The mixture is metered into the delivery hose by a feed wheel, feed cylinder or pneumatic distributor.
4. The mixture is carried by compressed air through the delivery hose to a special nozzle. The inside of the nozzle is fitted with a perforated manifold through which water (and possibly admixture) is introduced under pressure and intimately mixed with the other ingredients.
5. The concrete is jetted from the nozzle at high velocity on to the surface to be shotcreted.

Admixtures may be introduced, under strict control, in either dry form at the feeder, or in solution at the nozzle by using a dosing system.

23.3.2 Wet-mix process

This process consists of the following steps:

1. All of the ingredients, including mixing water and admixtures, but excluding accelerator or activator or special admixtures, if used, are thoroughly mixed at a plant which is either located next to the pumping equipment or at a remote plant with subsequent transport to the pump.
2. The concrete is introduced into the chamber of the delivery equipment.
3. The mixture is pumped through the delivery hose to a nozzle.
4. Accelerator/activator and other special admixtures, if used, are added at the nozzle.
5. Air is injected at the nozzle to increase the velocity and improve the gunning pattern and/or compaction of the placed shotcrete.
6. The concrete is jetted from the nozzle at high velocity onto the surface to be shotcreted.

The choice of process depends on operational features, cost of equipment and other considerations, some of which are given in Table 23.1.

Traditionally, users of shotcrete in South Africa have tended to prefer the smaller dry-mix shotcrete machines. Now that small wet-mix machines and/or retarded systems are available, their use has increased. There are also smaller dual wet/dry-mix machines available.

Either process can be adapted, by adding specially manufactured fibres to produce fibre-reinforced shotcrete (see section 23.4.5).

Table 23.1: Comparison of dry- and wet-mix processes

Dry-mix process	Wet-mix process
Instantaneous control over mixing water and consistence of mix at the nozzle to meet variable field conditions Better suited for low-density aggregates, refractory materials and shotcrete needing early strength properties Batches of mixed material are capable of being transported longer distances before application Start and stop placement characteristics are better with minimal waste and greater placement flexibility Low water:cement ratios result in higher strengths, but this can vary significantly depending on the operator Very operator-sensitive Higher environmental impact through dust Lower production rate	Mixing water is controlled at the point of manufacture and can be accurately measured Better assurance that the mixing water is thoroughly mixed with other ingredients Less dust generation and cement loss occurs at the shotcrete machine Lower rebound resulting in less material waste Capable of greater production Special mixing and transporting equipment is needed (recent developments also allow longer transport distances before application) Potentially higher wastage with non-continious operations Higher cost of placed material

23.4 Materials

23.4.1 Cementitious materials

Common cements, of all types and in all available strength classes, complying with SANS 50197-1,[23.7] are normally suitable. The suitability of a cement for a specific application should be established through site trials.

Incorporating condensed silica fume (CSF) in shotcrete makes it possible to substantially reduce rebound, improve adhesion and cohesion, build up thicker layers on both vertical and overhead surfaces, improve resistance to "wash out" and improve density. Typically substitution of CSF for cement ranges from 5 to 10% by mass. CSF should comply with SANS 1491: Part 3.[23.8]

A recent development is the use of ultra fine fly ash, in conjunction with CSF. This has been found to be beneficial, especially in retarded wet systems. Ultra fine fly ash should comply with SANS 1491: Part 2.[23.9]

The optimum selection of the cementitious material and strength class will depend to a large extent on specific requirements.

For special applications (refractory or high abrasion resistance) the use of Calcium Aluminate Cement may be considered. (See section 2.3.1). In all cases where this cement is used, advice should be obtained from the manufacturer.

Other cementitious materials not mentioned above (eg white portland cement, low-gypsum cement) may be used subject to their suitability being demonstrated in site trials.

23.4.2 Aggregates

Shotcrete is usually classified according to the process used: dry- or wet-mix; and sometimes the type of aggregate used: fine or coarse.

Except for grading requirements, aggregate should comply with requirements for use in conventional concrete, ie SANS 1083,[23.10] although rounded aggregates are preferable to the crushed concrete aggregates generally used in South Africa. Continuous grading of the fine aggregate is preferred; if necessary, this can be achieved by blending of natural sands with crusher sand. In general, all soft, friable, flat, elongated or laminated aggregates may have a deleterious effect on shotcrete. Crushed aggregates tend to increase the rate of wear of moving parts in the shotcrete equipment.

Recommended grading envelopes for dry shotcrete are shown in Table 23.2. The table, which is based on a large number of dry shotcrete applications in South Africa, should be regarded only as a starting point; fine and coarse aggregates lying outside the recommended envelopes may give satisfactory results. Site testing with specific materials and shotcreting equipment is the only way to establish suitability.

Although some modern shotcrete machines accept large stone sizes (up to 30 mm), the use of stone larger than 10 mm is not favoured in South Africa. This is because the flaky nature of the coarse aggregates used results in a high proportion of rebound and excessive abrasive wear of the shotcreting equipment.

Table 23.2: Recommended grading limits for dry shotcrete aggregates

Sieve size, mm	Percentage by mass passing sieve	
	Max. aggregate size 4,75 mm	Max. aggregate size 13,2 mm
13,2	-	100
9,5	-	90 –100
6,7	-	80 – 95
4,75	100	70 – 85
2,36	80 – 100	50 – 70
1,18	50 – 85	35 – 55
0,6	25 – 60	20 – 35
0,3	10 – 30	10 – 20
0,15	2 – 10	2 – 10

The selection of aggregates for wet shotcrete should follow the principles of designing a concrete mix suitable for pumping, using a relatively small coarse aggregate.

Alkali-silica reaction

Due to the high cement contents normally used in shotcrete, care must be taken with the selection of aggregates for shotcrete. If reactive aggregates have to be used, the risk of ASR in shotcrete can be minimised by using a cementitious material that contains, by mass, not less than either 40% GGBS, 20% FA or 15% CSF. (See Chapter 10).

23.4.3 Water

Requirements are as for conventional concrete (see also Chapter 4).

Recycled or waste water, which is often used in South African mines, must be tested for suitability.

23.4.4 Admixtures

Admixtures are usually used in the dry-mix system only where high early strength is required, eg to achieve support or to stop the ingress of water. These rapid-set accelerators, which are significantly different from accelerators for conventional concrete, contain sodium carbonate, sodium aluminate or other chemicals like aluminium sulphate. The dosage of rapid-set accelerators is extremely sensitive and has to be fine-tuned to the chemistry of the specific binder used. Admixtures are added either in powder form at the mixing machine or in liquid form at the nozzle.

In addition to the above accelerators, the wet-mix system uses normal concrete plasticisers, retarders, activators and others, all in liquid form.

An overview of admixture systems used in shotcrete can be found in reference 23.6. Admixtures specifically for shotcrete are covered by ASTM C1141.[23.11]

23.4.5 Reinforcement

In South Africa, the civil engineering industry tends to use weldmesh, while the mining industry uses weldmesh and diamond mesh (chainlink mesh) to reinforce shotcrete. Weldmesh should consist of 3- to 4-mm wires welded together at 100- to 300-mm centres. In no case should reinforcing wires be spaced less than 50 mm apart because closer spacing promotes the formation of "shadows" (voids) behind the mesh.

Fibre-reinforced shotcrete is generally technically superior to weldmesh-reinforced shotcrete and often allows smaller layer thicknesses. It is tougher, with higher tensile strength, and is recommended wherever it is practicable and economically feasible. Imported and locally produced steel fibres and, to a larger extent, polymeric fibres are available in South Africa. Advice on dosage, fibre aspect ratio (length divided by equivalent diameter), toughness performance and energy absorption capability (panel test) should be obtained from the manufacturer. Techniques to prevent balling of fibres and ensure uniform mixing must be tested by site trials. (See also Chapter 21.)

23.5 Shotcrete mix design

Because of the many job-related factors involved, shotcrete mixes cannot be designed in the same way as conventional concrete. The design of wet-mix shotcrete mixes is similar to that of a concrete suitable for pumping. Reference 23.14 provides guidelines for the design of shotcrete mixes.

23.6 Manufacture and placing of shotcrete

A detailed description of equipment and methods is outside the scope of this discussion: reference 23.12 as well as manufacturers of equipment and material suppliers may be consulted for more information. Points of particular importance are set out below.

23.6.1 Mixing and batching

Batch quantities must be measured out properly, either by volume or by mass.

When batching by volume, base each batch on one or more whole bags of cement and take the volume of a bag of cement to be 33 ℓ A 1:5 mix by volume therefore consists of 1 bag of cement and 165 ℓ of aggregate.

Wet-mix shotcrete should preferably be mass batched.

Monitoring the moisture content of aggregate stockpiles is especially important because of the low water:cement ratios used, particularly in dry-mix shotcrete.

Pre-bagged mixes, which are now available from a number of suppliers in South Africa, especially for the mining industry, facilitate accurate batching.

Cement (binder) and aggregates to be used in the dry-mix system need to be premixed.

24.6.2 Placing time

In the absence of retarders, shotcrete, whether wet- or dry-mix, should be placed within 90 minutes of batching.

The placing time of retarded and reactivated mixes can be extended to 24 hours and beyond. Specific advice must be obtained from the admixture supplier.

23.6.3 Placing equipment

There is a large variety of locally manufactured and imported placing equipment available in South Africa. For information and developments on wet-mix shotcrete placing equipment (including shotcrete nozzles) see reference 23.13.

23.6.4 Surface preparation

It is imperative that shotcrete bonds to the rock surface creating a deep beam effect. Therefore, surfaces to be shotcreted should be cleaned of all loose material, oil, etc, and should be damp (but not saturated) immediately prior to placing.

23.6.5 Reinforcement

Mesh reinforcement

It is imperative that mesh be firmly fixed (generally by means of pins), as any vibration of the mesh during placing will result in voids and sloughing off of the shotcrete. Mesh should be free of oil, loose rust, grease and other deleterious materials.

Steel-fibre reinforcement

At the time of batching, steel-fibre reinforcement should be free of rust, oil and other deleterious materials and fibres must not adhere to each other. Fibres should be added continuously and in small quantities during the mixing cycle so as to prevent balling.

Polymeric-fibre reinforcement

Similar precautions as mentioned above must be taken during the mixing cycle.

23.6.6 Addition of water

For dry-mix shotcrete, water added at the nozzle must be carefully monitored by the nozzleman to ensure that the water:cement ratio is maintained.This is not easy and nozzlemen of proven experience will facilitate the process.

For wet-mix shotcrete the water addition can be established in the laboratory and at site trials. Water addition must always be strictly controlled.

23.6.7 Rebound

The phenomenon of rebound is peculiar to shotcreting and contributes markedly to the uncertainties attached to the process. Rebound has two different effects:

- It causes an overall loss of materials.
- It tends to change the mix proportions.

The first effect may mean a loss of 10 to 50% of the input, depending upon circumstances, eg shooting horizontally or vertically upwards, thickness already placed, skill of the operator, nature of the mix, etc.

The second effect results in a change to the cement: aggregate ratio in the placed mix. In the dry-mix process, a richer in-situ ratio is achieved. This is because the larger particles are more easily lost through rebound.

The effect of rebound on the properties of placed wet-mix shotcrete is not yet fully established.

Shooting vertically either upwards or downwards is usually more difficult than shooting horizontally. When shooting downwards to form horizontal or nearly horizontal surfaces, special precautions should be taken to exclude rebound from the placed shotcrete. It is good practice to use an air lance to blow away rebound that may collect in inaccessible areas. Shotcrete should never be placed over rebound.

Rebound must be completely removed from the area being shot, and be discarded. Under no circumstances must rebound material be re-used as shotcrete aggregate.

The impact on the environment by dumping rebound must be considered, especially when using accelerators containing alkalis. Further details can be found in reference 23.15.

23.6.8 Curing

Shotcrete needs effective damp-curing. As a general rule, shotcrete should be wetted with a fine mist spray every four hours for at least three days.

Whilst moist underground conditions are normally favourable for curing, the effect of drafts due to ventilation systems should not be neglected.

23.6.9 Safety, health and environment

Attention should be given to protection of workmen from rebound material (particularly in respect of eye protection) and high dust levels. Additional ventilation and/or dust masks are frequently needed to ensure acceptable environmental conditions for the shotcreting crew. Skin protection measures should be stipulated when handling any cementitious material, and particularly when using caustic accelerators. Lighting levels must be sufficient for safe working conditions.

Further attention must be given to safeguard the shotcrete crew from moving parts of equipment and high pressures in both wet- and dry-process delivery lines. Detailed information regarding hose couplings, environmental issues, impact on soil and impact on water can be found in reference 23.15.

23.6.10 Shotcrete operators

The importance of good workmanship and proper supervision cannot be overemphasised. Nozzlemen and their backup teams should be trained and tested, with emphasis placed on surface finish, water control and the results of any specified tests (strength, density, etc) before any permanent work is undertaken.

The hand-applied system (nozzleman) requires the regular rotation of operators whilst robotic arm systems are less strenuous on the operator.

Additional information can be found in reference 23.15.

23.7 Testing

23.7.1 Thickness control

During placing, thickness control is achieved by using depth pins, and may be tested immediately after placing by probing. In the case of accelerated shotcrete, probing may not be possible and holes should be drilled to determine thickness.

Another way to control thickness is to use the grid-pattern and in-fill spraying method, where the thickness of the applied layer can be observed by the operator.

23.7.2 Surface finish

Shotcrete is naturally rough and surface finish is therefore difficult to control. The most common test is to measure the deviation from the specified line using a template. The maximum allowable deviation will depend on the application.

Steel-fibre shotcrete linings might require a final thin layer without fibre for protection of equipment and personnel moving through underground workings. A final thin layer can also be used to achieve the required finish, especially in water transfer tunnels where friction losses from a rough surface are undesirable.

Another development is the use of rolling formwork in conjunction with a modified shotcrete mix design for either wet- or dry-mix shotcrete. With this system it is important that the shotcrete remain plastic for about one minute but set two and a half to three minutes later. This allows a shotcrete layer of 300 – 400 mm to adhere to a rock surface, even in an overhead position without support from the rolling formwork.[23.16] For wet-mix shotcrete special accelerators have to be used, whilst the dry-mix process makes use of low-gypsum cements.

23.7.3 Compressive strength

In common with conventional concrete, strength is a useful index of shotcrete quality. It is not possible to mould cubes of dry-mix shotcrete. An alternative method of assessing strength, and the relative criterion, should therefore be specified before the start of a contract. The following methods can be used:

In-situ testing
Cores are drilled from the in-situ shotcrete and crushing strength is measured after trimming and capping. This method is suitable only where shotcrete thickness is sufficient; say 75 mm. Core diameter and length:diameter ratio as capped or ground should be specified.

Normally 28-day strengths are specified. Testing at earlier ages should be done if early strength development is important.

Test panels
Shotcrete panels or boxes should have angled sides with a recommended bottom measurement of 750 mm x 750 mm x 200 mm deep. These panels can be set up alongside the working area in the position corresponding to the surface to be shotcreted (preferably vertical or overhead) and filled with shotcrete before the actual work is started.

When the shotcrete has developed sufficient strength, the sample can be de-moulded; tests are carried out on beams (tensile strength and toughness) or cores (compressive strength) sawn or drilled from the sample.

SANS Method 5865[23.17] may be used as a guideline in drawing up specifications for the first two methods. A 100-mm core diameter should be satisfactory for shotcrete made of 13,2-mm maximum-size aggregate.

With the wet-mix shotcrete process, additional cube samples can be taken either at the batching or placing site.

In-situ early-strength testing
This is carried out using a penetrometer in the initial stages and the Hilti gun method as the strength increases.[23.14]

Testing can also be undertaken on shotcrete placed into panels. These panels can be used afterwards for core drilling.

24.7.4 Workability of fresh shotcrete

Workability and cohesiveness can be evaluated with a slump cone and/or the flow table.

Further information can be found in reference 23.15.

23.7.5 Density

Density is an important aspect of shotcrete control, particularly with regard to effectiveness of placing. Quality control is usually achieved by a combination of tests for boiled absorption and volume of permeable voids using ASTM C642 [23.18] test procedures.

These parameters are normally monitored only on major projects or where durability is important.

23.7.6 Toughness and energy absorption

Fibre-reinforced shotcrete can either be tested for toughness in accordance with ASTM C1018,[23.19] or for energy absorption using test panels as described in references 23.15 and 23.20.

23.7.7 Bond strength

The determination of bond, ie adhesion to the substrate, strength is of particular importance when shotcrete is used for structural concrete repairs. The bond strength is normally determined by partial core testing as described in reference 23.15.

23.8 Properties of hardened shotcrete

Properly applied shotcrete is a structurally adequate and durable material and it is capable of excellent bond with concrete, masonry and some other materials. However, these favourable properties are dependent on proper planning and supervision, careful selection of shotcrete materials, proper mix design, favourable placing conditions, correct equipment selection, and the competence and experience of the application crew.

Many references to shotcrete are found in the literature; the available data show a wide range of values. This is attributable partly to actual variations in quality and partly to the difficulty in obtaining representative samples for testing. However there is every reason to expect that the engineering properties of shotcrete vary only in the normal manner of concretes with the characteristics of the mix after it has been placed.

23.8.1 Water content

The evaluation of a large number of dry-mix shotcretes (locally and internationally) has indicated that the water: cement ratio of the placed mix is about 0,55 (ranging from 0,50 – 0,60).

The water content of the wet-mix shotcrete depends on a large number of variables and can be determined in the laboratory.

The water contents in both systems are comparable with normal concretes and account for the low permeability and good potential durability of shotcrete.

23.8.2 Compressive strength and variability

The variability of dry shotcrete can be greater than that of normal concretes in comparable conditions, probably because shotcrete properties are so dependent on workmanship. Properly applied wet-mix shotcrete usually shows less variability then dry shotcrete.

In view of the abovementioned variability, dry-mix shotcrete should generally not be specified where the required strength is higher than 30 MPa. Wet-mix shotcrete can be specified for strengths of 50 MPa or even higher.

23.8.3 Density

On major projects or projects where the long-term durability of shotcrete is important, the boiled absorption and volume of permeable voids are routinely tested. For shotcrete without accelerator, upper limits of around 8% and 17% are specified for boiled absorption and volume of permeable voids respectively. For accelerated mixes the corresponding figures are 9% and 19%.

23.8.4 Drying shrinkage

Drying shrinkage of shotcrete depends on the mix proportions used but is generally somewhat higher than comparable conventional concrete. The reason for this is the utilisation in shotcrete of higher cement contents and smaller aggregates.

The higher shrinkage of shotcrete could result in more shrinkage cracking, compared with conventional concrete. Shotcrete may require more reinforcing wires or fibres, than conventional concrete, to control shrinkage cracking, particularly in shotcrete used for concrete repair or corrosion protection.

23.9 Fibre-reinforced shotcrete

Testing has shown that steel-fibre-reinforced shotcrete (SFRS) has shown a ductile behaviour superior to that of weldmesh- and diamond-mesh-reinforced concrete. Steel-fibre reinforcement can also improve energy absorption, impact resistance, fatigue resistance, and crack resistance of shotcrete. In addition, SFRS continues to carry load after cracking.

Some of the abovementioned properties can also be achieved through the use of polymeric fibres which have been gaining popularity in South Africa, mainly in the mining industry.

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Chapter 24
Roller-compacted concrete

Quentin Shaw and Bryan Perrie

24.1 Introduction

Roller-compacted concrete, or RCC, is a zero-slump concrete that is transported, spread and compacted using large, earth-moving plant. As a consequence of the application of high-capacity plant and equipment, RCC is most suited to use in large-scale construction and for mass concrete works. Since the early 1980s, RCC has gained general acceptance as an appropriate material and method for the construction of dams and by the end of 2008 more than 400 large RCC dams had been completed worldwide. It has also been successfully used for pavement construction, most notably in Canada, the USA and Spain.

In this chapter, section 24.3 on RCC for dams was authored by Quentin Shaw and section 24.4 on its use in pavement construction by Bryan Perrie.

During its early development, RCC was perceived as a low-quality, low-strength mass material. It has since become possible to produce a range of concrete qualities by roller compaction, with the most common product being a dense, high-quality and relatively high-strength concrete.

While RCC mix design technologies for dams developed through two divergent "geotechnical" and "concrete" approaches, the lower strength, geotechnical variations of the material are now more commonly categorised as "Hardfill" or "Cement Stabilised Gravel" (CSG). Whereas some of the early "rollcretes" involved the addition and mixing of cement after spreading, modern RCCs are now accurately batched and mixed for a controlled period at a central plant before being conveyed to the point of placement.

Today, RCC is used principally in dams and pavements, including industrial pavements, haulage roads in mines, and airport aprons. Generally, it has an application wherever large quantities of concrete have to be placed quickly. Other applications include canals, chutes, river crossings, protective works and protection of embankment dams.[24.1]

24.2 Properties of RCC

Experience and testing have demonstrated the properties of hardened RCC to be similar to those of conventional mass concrete.[24.2] Differences are more apparent in the fresh state and in RCC, relate primarily to lower water and cement contents and a higher aggregate content.

Differences in the hardened concrete include:

- The volume change of RCC after set is reduced, as a result of less drying shrinkage (due to less water in the mix), less heat of hydration (because of lower cement content) and a lower effective long-term temperature drop.
- Creep is reduced as a result of lower total paste content, lower thermal stresses and an apparent early capacity to carry low levels of compression stress.
- Higher densities are achieved due to the use of continuously-graded aggregates and tighter control on aggregate shape. Consequently the total aggregate content is increased.

More detailed information is available in the references listed at the end of this chapter.

24.3 RCC for dams

24.3.1 Introduction

In principle, RCC is placed and compacted in horizontal layers, adopting a similar approach to that applied for earth- and rockfill dams. RCC dams are often constructed at rates of around 10 m in height per month. During construction of the 195-m (first phase) Lontan Dam in China, maximum placement rates of 18 900 m^3 in a day and 400 754 m^3 in a month were achieved.

The main benefits of RCC for dam construction are increased economy and more rapid implementation.

To minimise cementitious material contents and to take advantage of the fact that the critical zones of a large dam do not generally experience stress for an extended period after placement, characteristic strengths for RCC are specified at ages of up to one year, and generally not less than 90 days.

The majority of RCC dams contain mineral admixtures, most commonly fly ash, as an active constituent of the concrete.

24.3.2 Modern RCCs

Modern RCCs are primarily designed in accordance with two different approaches:[24.3]

- The "overall" approach, which relies on the dam body for water-tightness through high-quality concrete and treatment to ensure well-bonded layer and lift joints.
- The "separate" approach, which relies on an independent impervious barrier, which is usually placed on the upstream face.

These can be broken down further into the following classifications:

- The lean RCC dam, for which the cementitious materials content is < 100 kg/m^3.
- The RCD method (roller-compacted dam) unique to Japan, for which the cementitious materials content is generally 125 kg/m^3, but only the hearting zone of the dam is RCC.
- The high-paste RCC dam, for which the cementitious materials content is > 150 kg/m^3.

Although the above classifications are essentially based on cementitious material content, each method has a slightly different philosophy in respect of dam design.

The lean RCC dam uses an upstream face watertight membrane, which generally comprises an attached PVC geomembrane, but has also included conventional immersion-vibrated facing concrete, or precast concrete panels.

In the case of the RCD method, the final dam structure is very similar to a traditional conventional concrete gravity dam, with 15-m wide monoliths, although these are formed by cutting as opposed to forming with shuttering.

For the high-paste RCC dam, the RCC material itself provides the watertight barrier and must be designed for an in-situ permeability equivalent to that of traditional dam mass concrete. The RCC and the associated construction methods must further be designed to ensure effective bond between layers. Various facing systems are applied for high-paste RCC dams, all with the simple objective of creating a good and durable surface finish. Transverse joints are induced at pre-determined intervals, which are generally wider than is the case on a conventional vibrated mass concrete dam.

An additional category would include medium-cementitious RCC dams, for which the cementitious material content is between 100 kg/m^3 and 149 kg/m^3. There is, however, a noticeable trend away from this type of RCC as it offers few of the benefits of either the lean or high-paste approaches.

In early RCC dam construction, a particular problem was recognised as low bond between successive placement layers. While a relatively high shear friction angle could generally be assured under all circumstances, low cohesion and tensile strengths were compounded by high permeability when a new layer was placed on an excessively mature existing layer.

Development in the interim has included the use of set-retarding admixtures and the use of sloped and non-continuous layer placement methods to ensure the freshness of the underlying RCC layer when the subsequent layer is placed. While such practices are only implemented where required as part of the dam design, the result is a seamless bond between successive RCC placement layers, with joint properties equivalent to the parent RCC properties. These aspects will be discussed in more detail in section 24.3.6.

The facing systems used in RCC dam construction have changed in recent years and, while early construction saw the application of conventional concrete skins, extruded kerbs and precast concrete elements, modern practice increasingly involves the modification of RCC to allow compaction by immersion vibrator. In GE-RCC (grout-enriched RCC), grout is added to the uncompacted RCC against the formwork from the top after spreading. For GEVR (grout-enriched vibrated RCC), grout is added at the bottom of the layer against the formwork before RCC spreading. In both cases, a width of 400 to 500 mm of RCC against the formwork is modified with grout and compacted in the same manner as conventional concrete, while the interface between this material and the unmodified RCC is compacted using a small vibratory roller.

24.3.3 Materials

Aggregates

The selection of aggregates and the control of aggregate gradings are important factors influencing the quality and properties of RCC for dams. Coarse and fine aggregates can either be blended to achieve the required properties, or a continuously-graded all-in aggregate may be used.

In order to allow reduced water and cementitious materials contents in a more cohesive mix, a trend towards tighter specifications on aggregate shape has developed.

For all aggregate sizes, the allowable percentage of flat and elongated particles will be limited, depending on the quality requirements of the specific RCC.

Coarse aggregates

Coarse aggregates must be hard enough to withstand crushing during compaction by vibrating rollers. A minimum 10% FACT value of 110 kN is regarded as acceptable. Generally, the same physical property requirements for aggregates in conventional concrete are applied for RCC.

The maximum aggregate size is usually limited to 50 mm, although sometimes 75-mm maximum size aggregate is used. The use of smaller maximum size aggregates tends to lead to an increase in cementitious materials, sand and water contents, while aggregates larger than 37,5 mm tend to segregate when transported and dumped.

Both natural and crushed coarse aggregates have been used for RCC, but generally the quality requirements typically applied in current practice imply that crushed aggregates, separated into a number of discrete grading bands, are used. The grading requirements of SANS 1083:2006, *Aggregates from natural sources – Aggregates for concrete* are appropriate.

Fine aggregates

The following recommendations have been made for fine aggregates:

- Grading conforming to the requirements of SANS 1083:2006
- Fineness modulus: 2,0 to 2,8
- Percentage voids: 48% maximum

An important feature of the grading limits given above is that the material fraction passing the 0,6-mm sieve must not be less than 40%, and should preferably be 60%.

All-in grading limits

Continuous gradings have been found to produce more workable concrete, requiring less water and cementitious material, and achieving better compressive strengths than mixes made with gap-graded aggregates.

As a result, fine and coarse aggregates often have to be combined to provide specified grading limits. In addition, if different sizes of coarse aggregate are required, each size should have its own grading.

Table 24.1 shows grading limits for total aggregate content that can be used for initial guidance in arriving at a suitable grading for RCC dams.

Table 24.1: Grading limits for total aggregate content for RCC for dams

Sieve size, mm	% passing
53,0	95 - 100
37,5	70 - 90
19,0	55 - 70
9,5	40 - 55
4,75	35 - 45
2,36	30 - 40
1,18	25 - 35
0,60	17 - 30
0,30	10 - 22
0,15	5 -15
0,075	2 - 8

Cementitious materials

RCC can be manufactured with any of the basic types of cement and, more usually, a combination of cement and a mineral admixture or pozzolan. The use of mineral admixtures has a number of beneficial effects, which can be summarised as follows:

- Reducing the cement content, which reduces cost and total heat of hydration
- A slower rate of heat development
- A slower rate of strength development, which can improve the bond between successive placement layers and reduce thermal stress
- The addition of fine particles, which increase the paste content and accordingly the overall RCC workability

In South Africa, mineral admixtures used have included fly ash (FA), ground granulated blastfurnace slag (GGBS) and Corex slag. South African Corex slag has been found to be highly reactive and will not necessarily reduce the rate of heat and strength development. Internationally, natural pozzolans, such as volcanic ash, pumice, tuffs, opaline, cherts, shales and some diatomaceous earths have been used in RCC. In addition, milled basaltic and calcareous rock has been used, primarily as filler material, to increase the paste content in lean-mix RCC.

As a consequence of the size of the concrete blocks in a large dam, a very extended period can be required to fully dissipate the heat evolved during hydration. This may be of little consequence in the case of a gravity dam, but is more significant for an arch, or arch/gravity dam. Accordingly, it is often considered beneficial to minimise the total heat of hydration and low- or moderate-heat-generating cements are often specified for RCC, together with the maximum proportions of mineral admixtures commensurate with strength requirements at the age specified. Cementitious material blends commonly used in South Africa include:

- 30:70 blends of CEM I and FA
- 30:70 blends of CEM I and GGBS
- 50:50 blends of CEM I and FA

Total contents of cementitious material in South Africa have ranged from 110 to 200 kg/m^3. Internationally, cementitious material contents in RCC for dams have ranged from 70 to 250 kg/m^3, with an increasing trend towards the higher contents as higher RCC dams become prevalent and the benefits associated with a cohesive mix and the design advantages associated with high-strength concretes become better understood and increasingly exploited.

Admixtures

Both water-reducing and set-retarding admixtures are quite commonly used in current RCC construction practice. Water-reducing admixtures reduce the water content for a specific mix consistency, the consequential advantages being increased strength and compaction density. The advantages of set retardation are improved inter-layer bond and reduced cold joint treatment, particularly during hot weather. Because of the low water content of RCC, the effectiveness of an admixture is often compromised. However, set retardation can generally now allow RCC to remain fresh for a period exceeding 24 hours.

24.3.4 Mix parameters

Beyond the basic requirement of strength, a modern RCC mix is defined by the following mix parameters:

- Paste:mortar (p:m) ratio
- Sand:aggregate (s:a) ratio
- Maximum size aggregate (MSA)
- Modified Vebe time

The paste fraction is generally defined as cement + water + air. Sometimes this is extended to include non-plastic aggregate fines, ie inert fillers such as rock flour. The mortar fraction is the paste + fine aggregate.

These parameters relate essentially to the required material properties as follows:

- Paste:mortar (p:m) ratio relates to the achievable density (and impermeability). For the "overall" approach, mixes are designed for maximum density, with a paste:mortar ratio of at least 0,37 being required to achieve a minimum density of 98,5% of the theoretical maximum solid density.
- Sand:aggregate (s:a) ratio refers to the achievable compaction ratio. While early RCC testing suggested that a lower sand:aggregate ratio was possible for RCC, compared to conventional vibrated concrete (CVC), practical experience in the interim has demonstrated that quality control and the maintenance of RCC consistency is much more realistically achieved in an RCC with a sand:aggregate ratio exceeding 0,35.
- Maximum size aggregate (MSA) relates to the tendency of the constituent materials to segregate during handling. Aggregate sizes larger than 37,5 mm demonstrate a tendency to segregate in an RCC mix during handling operations. Accordingly, a trend toward reducing the MSA from the original 75 mm has become evident, and most RCCs now specify an MSA of 50 or 53 mm.
- Modified Vebe time. The workability of RCC is determined by testing with the Vebe apparatus, which is modified to include a surcharge mass of 19 kg. For workable RCC, the modified Vebe time should lie between 10 and 20 seconds.

Under construction conditions, the aforementioned properties determine workability and the difference between a permeable, stony RCC with planes of weakness and a cohesive, seamless, watertight and dense RCC.

24.3.5 Mix proportioning methods

During the early development stages of RCC, a number of mix proportioning methods were applied. While all methods sought to realise a low-cost mix that would produce a concrete of adequate durability, strength and density, some approached the problem in a similar manner to concrete, while others took an approach applying soil-compaction concepts. The latter techniques are now more commonly used for "Hardfill" and "CSG", and current practice generally involves designing RCC in a similar manner to conventional mass concrete for dams.

The starting point for RCC mix design will be the applicable dam design approach, whether "overall", or "separate".[24.3]

In the case of the "overall" approach:

- A minimum paste:mortar ratio of 0,37 is necessary to ensure adequate density and consequently impermeability.
- For cohesive, compactable RCC, a minimum sand: aggregate ratio of approximately 0,35 is usually also required.
- Depending on the quality of the sand available for the RCC being designed, the water content necessary to achieve a modified Vebe time of between 10 and 20 seconds will vary between 85 litres/m^3 for a very good natural sand, and 125 litres/m^3 for a poor, manufactured sand.
- The cementitious material content is then calculated based on the p:m and s:a ratios. In the case of typical aggregates, a s:a ratio of 0,35, a p:m ratio of 0,37 and a water content of 105 litres/m^3 will indicate the need for a minimum cementitious material content of approximately 150 kg/m^3, ignoring sand fines as part of the paste fraction. In the case of a lower cementitious material content, non-plastic sand, mineral, or manufactured fines will generally be required to achieve an RCC of maximum density.

With the water content, the cementitious material content and the minimum required values for the p:m and s:a ratios, the aggregates will be proportioned for the anticipated final RCC density ie approximately 98,5% solid density for a p:m ratio of 0,37; higher density for a higher p:m ratio and lower density for a lower p:m ratio.

In the "separate" approach, the cementitious material requirement is calculated from the required w:c ratio for strength and durability.

As is the case for all concrete mixes, the designs are only validated with laboratory trials. With RCC however, the mixes are only finalised on completion of large-scale practical construction trials, whose purpose is to prove workability, cohesiveness, consistency, constructability, compactability, adhesion between layers, etc under realistic, practical conditions.

The following steps illustrate the "overall" mix proportioning method:

1. **Target RCC strengths**
   The target strength for RCC needs to be determined taking the following requirements into account:
   - Compressive strength
   - Sustained elastic modulus in both compression and tension
   - Direct tensile strength
   - Direct shear strength

   Strength requirements are normally specified at 90 days.

2. **Additional specified RCC parameters**
   The project specification typically prescribes limiting values to the following mix parameters:
   - Water:cement ratio
   - Minimum cement content
   - Minimum extender content
   - Target Vebe time
   - Maximum aggregate size
   - Minimum sand:aggregate ratio
   - Minimum paste:mortar ratio

   The mix proportions must also be evaluated on the basis of these additional requirements.

3. **Mix proportions**
   The mix needs to be proportioned to minimise cement content for economy and to achieve maximum density. A theoretical mix is designed by observing the following steps:
   - Choose a theoretical cement content based on the typeof cement and required proportion of extender toachieve the target 90-day strength.
   - Choose a water content that will produce the desired workability (Vebe time of 10 to 20 seconds) with the parti-cular set of aggregates to be used.
   - Calculate the water:cement ratio from the theoretical cementitious and chosen water contents.
   - Based on the above, calculate the paste volume (volume of water + volume of cementitious materials) using the relative densities of the water and cementitious materials.
   - Choose a paste:mortar ratio to achieve adequate density, and calculate the mortar volume.
   - Calculate the volume of the fine aggregate by subtracting the paste volume from the mortar volume. The mass of the fine aggregate is then calculated using the relative density of the sand.
   - Calculate the coarse aggregate content for a target density of 98,5%.

4. **Trial mix assessment in the laboratory**
   Prepare the mix in the laboratory and assess the Vebe time. A conventional vibrated mass concrete would exhibit a modified Vebe time of approximately 5 seconds, which would represent excessive workability for RCC mixes.

   Should it not be possible to achieve a modified Vebe time of less than 20 seconds with the trial mix, the mix proportions need to be revised, with the inclusion of additional water. The sand:aggregate ratio can also be increased to increase workability. Either of these modifications is likely to increase the overall cementitious material content.

   Great emphasis should be given to keeping the total cementitious material content as low as possible to minimise deleterious hydration-heat build-up, as well as for reasons of project economy.

   Another alternative is to increase the extender content. It should be noted that FA for example, will act effectively to enhance overall workability, but will contribute very little to RCC strength above a certain proportion.

   While an increased cement content can be effectively applied to counter the requirement for more sand and/or water, it will also lead to increased heat of hydration. An increase of more than 85 kg/m^3 is not recommended.

   If the above mix reflects a Vebe time of less than 10 seconds, the mix proportions need to be revised by reducing the water content. In this instance, particular attention should be paid to the fines content of the sand to ensure that the effective paste content (taking fines into account) is not too low to achieve adequate density.

   Should the mix modifications be insufficient to produce a mix of the required workability and consistence, it may be necessary to review the coarse aggregate particle shape and grading distribution, or to evaluate the possible use of other types of cement extenders.

5. **Large-scale construction trial**
   As is the case for all concrete mixes, designs are only validated with laboratory trials and with RCC, mixes are only finalised on completion of large-scale practical construction trials, which aim to prove that the

required properties are achievable under realistic, practical conditions.

The aim of the materials engineer or concrete technologist should be to minimise the use of cementitious materials for economy and to investigate a paste:mortar ratio of up to 0,40 for workability and a water:cement ratio of down to 0,45 for strength and durability.

24.3.6 Construction

Batching and mixing

Batch mixers are preferred for all major RCC production. During the early period of RCC development, continuous batching and mixing plants were favoured because of their apparently higher output capacity. However, poor consistency, low accuracy, high wear and extended down time has generally resulted in a very low production/capacity ratio. In conjunction with more difficult quality control, this has resulted in continuous batching and mixing plant being used progressively less frequently for major RCC dam construction. Preference is usually given to twin-horizontal-shaft pan-type concrete mixers. Transit mixers are not suitable for RCC under any circumstances.

Conveyance

While the preferred method for RCC transportation is by conveyor to the dam wall and by dump truck on the dam wall, in certain instances all-conveyor, or all-dump truck systems are used. Dump trucks with narrow discharge chutes should be avoided and the best dumper systems allow the load of RCC to slide off the load bed as far as possible as a single body.

Regardless of the method of transportation, RCC should always be deposited on previously spread but uncompacted material, and must be deposited in a position where it will be moved/worked/turned before compaction. RCC must never be compacted where deposited.

Placing and spreading

Early RCC dam construction applied horizontal placement, starting at one abutment and working continuously across the valley to the other, generally with a compacted layer depth of 300 mm, although 250 mm was used in South Africa. More recently, non-continuous layers have been utilised as a means to improve inter-layer bond and to reduce surface treatment requirements.

Using a non-continuous layer placement, RCC is placed in a number of successive layers over a restricted area of the surface of the dam. This placement approach can either be achieved using sloped layers (see Figure 24.1) or discontinuous horizontal layers (see Figure 24.2). The latter method, which effectively breaks each lift across the full dam surface into a series of separate placements, offers a significant advantage in easier start-up and in limiting the occurrence of potentially deleterious feathered edges that run in an upstream-downstream direction.

RCC is usually spread using bulldozers, with rotating lasers to control the layer thickness. As part of the spreading

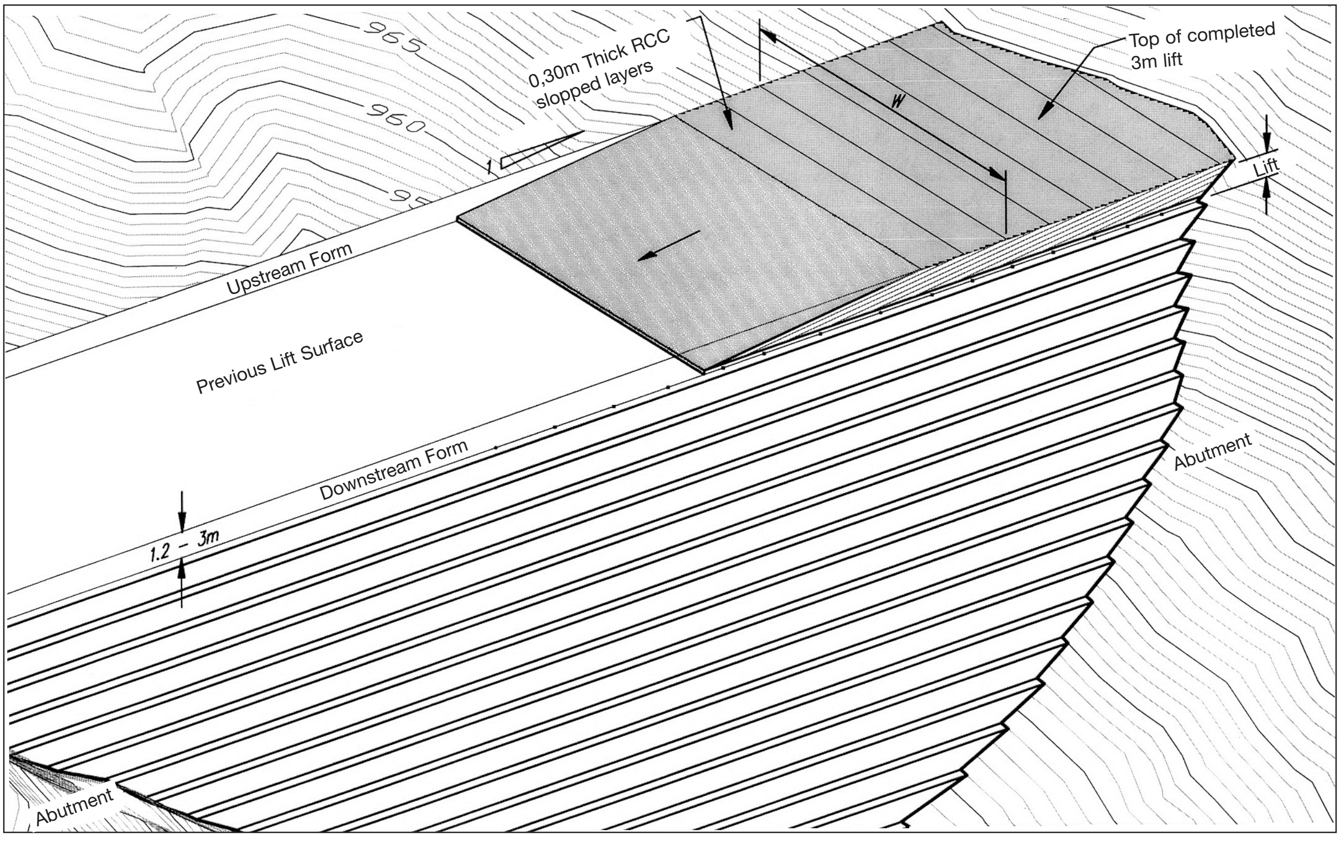

Figure 24.1: Sloped layer placement

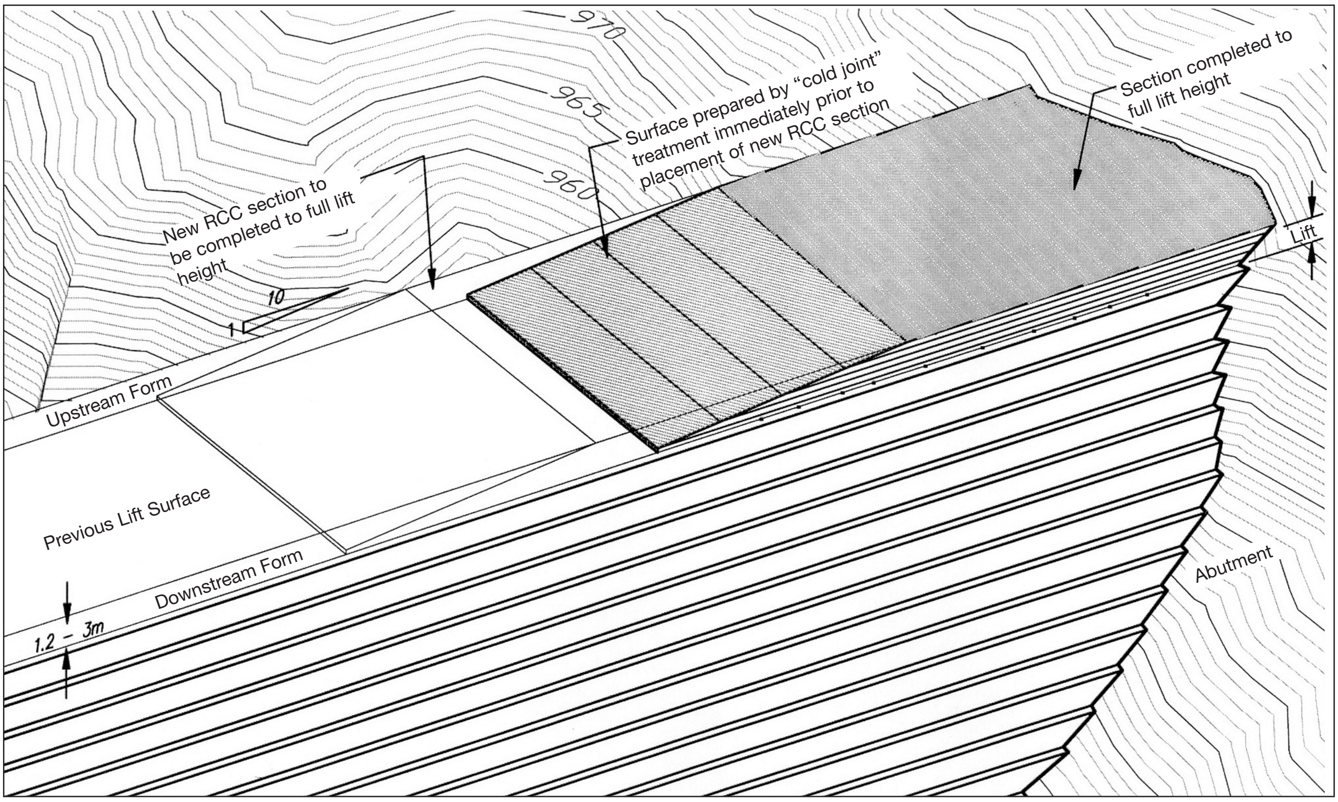

Figure 24.2: Non-continuous layer placement

equipment, compact loaders are essential to ensure re-mixing of segregated material and to facilitate the delivery of RCC to areas where restricted space prevents access with larger plant.

At the commencement of each placement layer, a starter RCC mix (standard RCC without +37-mm aggregate) is used to create a starter platform from which the standard RCC mix can subsequently be spread. Further layers are then created by dumping RCC on the surface of the advancing, uncompacted layer, approximately 2,5 m behind the advancing face, and bulldozing this material off to its final location for compaction.

Compaction

All RCC is compacted using self-propelled vibratory rollers. The bulk placement is usually compacted with single-drum rollers with a gross weight of between 10 and 15 tonnes. Restricted areas and the interface between the GE-RCC/GEVR and the RCC are generally compacted using a self-propelled vibratory roller with a gross mass of approximately 1 500 kg. Typically four to six passes of the vibratory roller are adequate to achieve the specified density. The RCC mix is usually modified if more than 10 compaction passes are required. RCC against formwork, abutment foundations, drainage galleries and any pipework is compacted as GE-RCC/GEVR.

Horizontal joints and joint treatment

The treatment used for the surfaces of the RCC layers will depend on the structural design requirements and the condition of the surface at the time of placement of the subsequent layer. For this purpose, layer joint surfaces are generally defined as "Hot", "Warm" or "Cold".

A layer surface will be considered as "Hot" before the first set of the RCC has occurred. A layer surface will be considered to be "Warm" if first set has occurred, but the surface cannot yet be considered to be "Cold". A layer surface will be regarded as "Cold" when it is judged that little or no penetration of the aggregate from the new placement into the previously placed layer will occur.

Different treatment measures are applied for each of the three conditions of surface maturity.

"Hot" RCC surfaces are generally treated in preparation for the subsequent layer by:

1. Keeping the full lift surface continuously moist.
2. Protecting the surface from damage, or contamination.
3. Removing all loose contaminants, or deteriorated RCC from the surface.

"Warm" RCC surfaces are generally treated in preparation for the subsequent layer by:

1. Using all the treatment measures for a "Hot" joint.
2. Covering the surface with a layer of bedding mortar or concrete within 15 minutes of placement of the RCC for the new layer.

"Cold" RCC surfaces are generally treated in preparation for the subsequent layer by:

1. Keeping the full lift surface continuously moist.
2. Removing all surface mortar, laitance and contaminants by high pressure washing or wet sandblasting while the surface is still green.
3. Cleaning the surface to remove all loose material, dust and dirt using labour, high-pressure water and air washing, a mechanical broom or a vacuum. The green cutting process exposes, but does not undercut coarse aggregate.
4. On resumption of RCC placement, covering the clean surface with a layer of bedding mortar, grout or concrete within 15 minutes of placement of the RCC for the new layer.

Forms and facings

Upstream and downstream faces can be unformed, or formed using conventional formwork, precast concrete elements or concrete kerbs. However, since the introduction of GE-RCC and GEVR, RCC is generally placed directly against climbing steel formwork, while forms of up to 2,1 m in height are used to minimise the requirements for "Cold" surface joint treatment.

Allowable placement conditions, curing and protection from weather

For RCC in dam construction, restrictions on the allowable maximum placement temperature and placing during adverse weather conditions are generally imposed. While these restrictions will be specific to the site and dam design, it is common to use ice cooling for RCC on large dams, as well as wet belt cooling for coarse aggregates.

Curing practice for RCC in dam construction is essentially the same as that for conventional mass concrete, with measures limited to maintaining all surfaces in a damp condition and minimising temperature gradients in particularly cold/extreme climates.

Contraction joints

In modern RCC dam construction practice, contraction joints are not formed, but induced on pre-determined alignments and isolated with waterstops. Joints are generally induced after partial or full compaction of a layer by cutting into the RCC and inserting a debonding plate, which can take a variety of forms, across the full width of the dam. Debonding plates can be inserted in all, in every second or in every fourth layer, depending on the design approach. A system of isolation waterstops, together with drains and sometimes grouting systems, are installed in the upstream GE-RCC/GEVR on the alignment of the induced joint.

On RCC arch dams, which have to date only been constructed in South Africa and China, systems for inducing and grouting contraction joints have been successfully developed and implemented.

24.3.7 Quality control

General

As a consequence of the pace of construction, the substantial volumes, the extended age specified for strength and the inability to return to remove unsatisfactory material, quality control for RCC dam construction is of particular importance. Cube, cylinder and core strength results (even accelerated curing tests) serve the purpose of providing a final verification record only, and RCC quality control programmes are designed around the approach of verifying the quality and consistency of the constituent materials, the production facilities and the fresh RCC. In addition, a comprehensive programme of measurement of in-situ compacted density and moisture content is implemented.

Tests

While all plant for RCC must be of an adequate quality, be well maintained and subject to regular verification testing, a good stock of all necessary spare parts must be maintained.

Typical quality control for RCC includes the following conformance testing:

- Testing cement and FA
- Sampling and testing aggregates
- Los Angeles abrasion testing of aggregates
- Aggregate particle shape testing
- Sampling fresh concrete (moisture content, etc)
- Concrete uniformity (Vebe)
- Density
- Air content
- RCC temperature
- Making and curing cubes/cylinders
- Compressive testing of cubes/cylinders

A full-scale trial is an essential component of the RCC QC programme. In addition to testing all plant, production and placing equipment, and training staff and operators, the testing done during this exercise will form the basis against which all later testing during placement on the dam wall will be evaluated.

Quality control of fresh concrete

Quality control during placement will involve inspection and testing. Inspection provides the first opportunity to observe a problem and institute corrective measures. The frequency of QC tests will be determined on the basis of the size of the project, the sensitivity of the design to variations in quality and the rate of RCC production. Often the frequency of testing will be high during early placement and will be progressively reduced as the statistical database increases and the consistency of the test results improves.

Quality control of fresh concrete involves careful judgment and consideration of the following aspects:

- Spreading and compaction
- Density and moisture content
- Layer and lift joint bonding
- Curing
- Placement temperature

Quality control inspectors should pay particular attention to the following aspects of construction:

- Foundation cleanup
- Placement and compaction of the interface concrete (or RCC) against the abutments
- Consolidation of the concrete around waterstops and other important embedded items
- Cleanliness of each lift and the moisture condition of the surface
- Placing and spreading of bedding mixes, if used, and the coverage of this layer with RCC
- Time periods between mixing and placement, and spreading to completion of compaction
- Air and RCC temperatures, and changing weather conditions

When unsatisfactory work is observed, the quality-control inspector must immediately bring this to the attention of the contractor's supervisor. All these items, and others, are essential to the quality of RCC construction, as it progresses at a rapid rate.

Density control is particularly important for RCC. Density is typically controlled by means of nuclear densitometers, which measure density at different depths within the layer. It is envisaged that in-situ density will be measured at a frequency of 1 test per 500 m^3 of RCC placed.

Quality control of lift joint bonding comprises the detection of possible contamination at the surface and the identification of cold joints. The first will be accomplished by specifying cleanliness of equipment at the RCC surface, the method and equipment to be used on lift surface cleanup and preparation, and by visual inspection prior to placing the next layer. It is common practice for large RCC projects to limit all traffic to vehicles staying on the dam or to clean the tyres of vehicles entering the concrete surface.

Records should be kept to identify cold joints and distinguish between joints which may or may not require special treatment. This record may be based on the maturity factor obtained from preliminary testing (full-scale trials) or simply the time elapsed since the placement of the previous layer.

Temperature control during construction is particularly important in the construction of large RCC dams. RCC temperatures will be measured when dumped, using an electronic probe, and in the compacted state using thermocouples.

Quality control of hardened concrete

Methods for quality control of hardened RCC in the dam are the same as those employed in the full-scale trial. Concrete cubes or cylinders are made at the time of placement, cured and then tested for strength and elastic modulus. Further specimens are obtained by coring and are subject to the same tests. Core samples contain the joints between lifts that will be subjected to detailed inspection and strength testing.

The permeability of the placed RCC is verified by water-pressure testing in holes drilled vertically into the dam body, and an appropriate programme for core recovery and core-hole permeability testing is compiled on the basis of approximately 1 m of core drilled for each 1000 m^3 of RCC placed. Generally, cores are drilled vertically from the crest, through the full height of the dam and across the foundation contact.

Test results from the RCC are evaluated statistically and compared with the design requirements.

Strength control of RCC is reviewed on the basis of the coefficients of variation (CoV) of the in-situ concrete indicated through testing on cores recovered from the structure.

Variations in RCC strength may be due to a number of factors including variations in:

- Cementitious materials content
- Water (moisture) content
- Fines content
- Control of compaction (density)
- Quality control at the concrete plant
- Mixer efficiency

Careful curing and handling of the test specimens can reduce the CoV of strength. Reducing the variability of each of these factors will reduce the overall variability of the in-situ compressive strength.

24.4 RCC for pavements

Much of the material in this section is drawn from the publications listed as references 24.4 to 24.9 at the end of this chapter.

24.4.1 Introduction

Roller-compacted concrete pavement (RCCP) comprises dry, high-strength concrete that is placed using construction procedures normally associated with asphalt construction rather than the construction techniques traditionally used with concrete pavement.

The literature suggests that RCCP is 10 to 30% more economical than conventional concrete pavements. RCCP's savings result mainly from cheaper construction techniques.

24.4.2 Materials

Aggregates

Aggregates suitable for RCCP generally comprise materials that meet the requirements of Clause 7102 (c) of the 1998 COLTO *Standard specifications for road and bridge works for state road authorities.* [24.10]

Aggregate gradings strongly influence construction difficulty and surface quality, and as problems have been encountered with segregation, pavement smoothness and poor surface, there has been a trend towards smaller maximum-size aggregates and well-graded aggregates, eg coarse aggregate with a maximum nominal size of 19 mm or 13,2 mm, and a combined grading as shown in Table 24.2. However, RCC pavements have been successfully constructed with aggregates with a variety of gradings.

Table 24.2: Combined aggregate grading for RCCP (based on US Corps of Engineers' requirements) [24.5]

Sieve size, mm	% passing
26,5	100
19,0	83 - 99
13,2	72 - 93
9,5	66 - 85
4,75	51 - 69
2,36	38 - 56
1,18	28 - 46
0,60	18 - 36
0,30	11 - 27
0,15	6 - 18
0,075	2 - 8

The US Corps of Engineers requires fine and coarse aggregates to be supplied separately. Other requirements stipulate a spherical or cubical shape for both coarse and fine aggregates, with the quantity of flaky particles limited to 20% in a standard test.

Most aggregates meeting the 10% FACT values or the Aggregate crushing values for crushed stone base courses in Section 3600 of the COLTO specification will be suitable for RCC.

Cementitious materials

Suitable cements for RCCP construction include CEM I and CEM II A complying with SANS 50197-1.[24.11] Alternatively, blends of CEM I and ground granulated blastfurnace slag (GGBS) or fly ash (FA) may be used. GGBS and FA should comply with the requirements of SANS 1491.[24.12] FA is often substituted for 15 to 20% of total cementitious content.[24.13] In Australia, FA contents up to approximately 30% by mass of portland cement have been used.[24.14]

An experimental section of road base constructed near Umhloti on the Natal North Coast utilised a 2:1 ratio of PC 15 SL (a factory blend of about 85% portland cement and 15% FA, now superseded by CEM II A-S) and FA.[24.15] FA contributes to cohesion and workability of concrete, readily responds to vibratory roller compaction; and is beneficial to the achievement of a uniform surface.[24.16]

The addition of 20 to 30 kg/m^3 of condensed silica fume (CSF) has, in some cases, been found to increase strength by about 30%. In Australia, use of CSF in the proportion of about 10% of total cementitious content (about 300 kg/m^3) has resulted in compressive strengths of 40 to 50 MPa at 28 days, and 50 to 60 MPa at 90 days. CSF should comply with the requirements of SANS 1491. [24.12]

Admixtures

Both plasticisers and retarders have been used. Plasticisers have been used to allow a reduction in water content in order to obtain an increased strength. Retarders have been used to delay setting in order to avoid cold construction joints as far as possible. Australian experience however suggests that the use of admixtures tends to be limited.

24.4.3 Mix proportioning

RCC for highways, airport aprons and industrial pavements typically has a compressive strength similar to that of conventional highway concrete. A characteristic (cube) compressive strength of at least 30 MPa at 28 days may be assumed. The corresponding elastic modulus is about (30 GPa) The design of RCC for pavements is based on flexural strength (as for any other concrete pavement) and the same values are generally used.

No consensus exists on the most appropriate concrete mix proportioning method for RCCP. A number of different approaches exist.

Proportioning by soil-compaction methods

This method consists of determining the relationship between moisture content and dry density for a specific cement and aggregate combination at a laboratory compactive effort, usually modified AASHTO. This approach identifies the optimum moisture content for achieving the maximum dry density of a specific material at the selected compactive effort. Once the results of optimum moisture content tests are available, mixes can be made covering a range of water: cement ratios, water contents and various additions of FA, GGBS or CSF (as appropriate)

Proportioning by soil evaluation of consistency tests

This method involves proportioning the RCC for optimum workability at the required level of strength using an apparatus such as the modified Vebe.

Proportioning using the optimal paste volume method[24.17]

This method is based on a theoretical model that permits a high packing density. The steps involved are determination of the ideal grading, selection of the paste volume and selection of the w:c ratio.

Test specimens

Specimens for unconfined compressive strength, flexural strength, indirect tensile strength and stiffness (E-modulus) can be manufactured for testing, usually at ages of seven and 28 days. Trial mix proportions can be selected based on the data and considerations of the water content strength and water content density relationships at different w:c ratios.

Generally, RCCP is placed with a w:c ratio between 0,30 and 0,40. The cementitious content is usually between 285 and 370 kg/m^3.

24.4.4 Thickness design

Both the Portland Cement Association (PCA) and the US Army Corps of Engineers have design methods for determining thickness of RCC pavements. These methods have been compared by Hiller and Buch who recommend an alternative method which addresses the shortcomings of both the aforementioned methods.[24.18]

Piggott[24.19] has prepared a report detailing the performance of a large number of RCC pavements under different loading and environmental conditions.

24.4.5 Subgrades and subbases

Design parameters for subgrades and subbases under RCCP are similar to those for conventional concrete pavement. However, White suggests that a strong foundation (subgrade modulus greater than 1,35 MPa) will result in less working of cracks because of improved pavement support; and that compaction on a weak resilient foundation may result in low density. [24.20]

The subbase of RCCP fulfils one or more of the following functions:

- Provision of a stable, firm working platform to facilitate construction
- Provision of uniform support
- Prevention of subgrade pumping

A detailed discussion of pumping and subbases is contained in reference 24.21.

High-quality cement-treated or lean concrete subbases are recommended for heavy traffic. This is because the provision of load-transfer devices, such as dowels, tiebars and keyways, presents practical difficulties in RCCP; hence, load transfer at joints in an RCC pavement designed for heavy traffic has to be obtained by aggregate interlock at joints with support from a cemented subbase.

24.4.6 Control of transverse cracking

The control of transverse cracking has been approached in two ways: either the pavement is left to crack randomly, which is the current practice in the USA, or transverse contraction joints are sawn, as is often the case in Spain, France and Australia. Joint sawing is considered to be more acceptable in terms of providing aggregate interlock and therefore load transfer at joints. The spacing of joints should be limited to a maximum of 30 x slab thickness, or 4,25 m, whichever is the least. The recommended depth of sawcut is one third of the slab thickness.

Where joints are to be sealed, the sealant groove may be sealed in two stages, ie an initial sawcut 3 mm wide, followed later by a widening of the upper portion of the sawcut to dimensions appropriate to the particular sealant to be used.

As previously noted, the stiff consistence of RCC does not lend itself to the installation of load-transfer devices, such as dowels or keyed joints.

24.4.7 Joint sealing

The purpose of sealing joints is to prevent intrusion of fine incompressible material and water into joints. Field-moulded sealants, such as silicone, are considered to be best suited to RCCP. Manufacturers' recommendations regarding sealant reservoir dimensions, procedures for joint preparation and priming (if any) and installation procedures should be followed.

24.4.8 Construction

Batching, mixing and transporting

Batching of the concrete is accomplished in either a continuous-mixing or a weigh-batch plant. There is general agreement that to disperse the relatively small amount of water evenly through the matrix, a twin-shaft pugmill mixer is required. Batches are typically hauled from the mixer to the paver(s) in dump trucks equipped with protective covers to obviate adverse environmental effects (eg rain, heat or extreme cold). The concrete is deposited directly into the paver hopper. Transit mixers are not suitable.

The maximum recommended haul time between mixing and placing is 15 minutes, which may necessitate a mobile mixing and batching plant in certain situations.

Placing

In Spain and France graders and bulldozers have been used successfully to spread concrete on low-volume and secondary roads. But RCCP is best placed with a modified asphalt paver equipped with an oscillating tamping-type screed with a vibrating pan for finishing. Commonly used pavers are ABG models 410 and 420. The paver must be equipped with an automatic grade-control device, such as a travelling ski or electronic string line. It should also have a large hopper and augers to ensure an even spread. The edge of each lane may be finished vertical or at 15° from vertical. Fresh concrete is placed and compacted within 45 minutes of adding water to the batch. The maximum period between placing lifts or adjacent lanes is 60 minutes, unless treatment for cold joints is to be provided. It is advantageous to have two or more pavers operating in echelon, since this eliminates longitudinal cold joints.

The placing width is limited to between 3,6 and 4,3 m to avoid mix segregation. Compacted lift thickness is limited to a maximum 250 mm, and a minimum of 100 mm.

Subgrade or subbase layers must be dampened prior to placing, since the loss of even a small amount of mixing water can result in incomplete hydration of the mix.

Compaction

Final compaction of RCCP is effected by means of steel-wheeled vibratory rollers and non-vibratory rubber-tyred rollers. The steel-wheeled roller should be of the dual-drum type, with a mass of about 10 000 kg, a minimum frequency of 1 500 cycles per minute and an amplitude variable between 0,4 and 1 m. Rubber-tyred rollers should have two axles with at least three wheels per axle, offset such that the front and rear tyres do not track in the same path; be capable of being operated at a tyre pressure of between 345 and 620 kPa; and with a wheel load between 14 and 20 kN.

A light dual-drum non-vibratory roller may be required to remove any roller marks left on the surface.

No more than ten minutes should elapse between placing and the beginning of rolling. Rolling should be completed within 45 minutes after mixing, except during hot weather. In hot weather, rolling should begin within five minutes of spreading and, except for joints, be completed within 30 minutes of the start of mixing.

A consistence check is first carried out by making one or two passes (one back-and-forth motion is two passes) of the steel-wheeled roller, in a non-vibratory mode, within (300 mm) of an edge of a lane and observing the material during the two passes to ensure that undue displacement does not occur. If the mix is too dry or too wet for compaction after placing, the water content should be adjusted. However, only minor changes in water content from the design mix should be made; otherwise, a new mix design may be needed.

After the consistence check, the RCC is compacted with the steel-wheeled roller ensuring adequate compaction at the joint between adjacent lanes.

During vibratory compaction the roller should never stop on the pavement in the vibratory mode. Vibration should be turned on only after the roller is in motion and discontinued before the roller stops. The stopping points of successive rolling passes should be staggered to avoid a depression in the RCCP surface. Rolling should be at the speed, amplitude and frequency to achieve maximum compaction and at a speed not exceeding 3 km/hour.

Vibratory compaction is followed immediately with two or more passes of a rubber-tyred roller to achieve a tight surface texture. Rolling by means of a light dual-drum roller may be necessary to remove any roller marks on the surface. All exposed surfaces of the RCCP must be kept moist with a light water spray after rolling until completion of curing.

Compaction is usually measured using nuclear density testing equipment. Quality control on mix proportioning is carried out in a number of ways. Compressive strength can be determined on cubes compacted using a small vibrating breaker on a steel plate or on cores drilled from the pavement. Indirect tensile strengths can be determined on cores drilled from the pavement. Flexural strength is often determined on beams cut from the pavement or manufactured in the laboratory using the same means of compaction as indicated above. These beams should be tested as cast or as cut and not on their sides as with conventional concrete.

Construction joints

Construction joints are necessary when delays occur between placement of adjacent lanes or sections, such as at the end of the day, or when required time limits for placing have been exceeded. For these situations, the longitudinal edge of the hardened concrete is trimmed by sawing to a depth of half of the depth of the pavement and removing the remaining unsawn concrete with light power tools to produce a face within 15° of vertical.

In continuing placing, the longitudinal sawn/trimmed vertical face is dampened. The paver screed is positioned to overlap the longitudinal construction joint by approximately

75 mm, and approximately 25% higher than the desired thickness of the compacted concrete. Excess fresh concrete should be pushed back to the edge of the fresh concrete with rakes and rounded off so that a minimal amount of fresh material remains on the surface of the hardened concrete. The edge of the fresh lane adjacent to the hardened concrete is rolled first in the static mode with about 300 mm of the roller on the fresh concrete.

Transverse construction joints are constructed in a similar manner, with a static pass made in the transverse direction across the first 300 mm of the fresh concrete.

In two-lift construction, the top layer is placed so that joints in the top and bottom layers coincide. If the edge of the upper layer is not even with the edge of the lower layer, the lower layer should be cut back even with the edge of the upper layer. Where required time limits between placing of successive layers are exceeded, the US Corps of Engineers requires the removal of the lower layer. However, technology is available to achieve adequate bond (a bond shear strength of 1,38 MPa) between successive layers; the Portland Cement Association has produced guidelines for obtaining adequate bonding between layers.[24.22]

The effect of multiple-lift construction or design can be evaluated using concrete overlay design concepts. An important aspect of multi-layer construction is that surface tolerances on the second layer are superior to those achieved in single-layer construction.[24.16]

Surface finish

As the consistence of roller-compacted concrete does not readily lend itself to the application of a surface texture, unsurfaced RCCP is generally not suitable for roads carrying high-speed traffic.

For heavy-duty, low-speed and medium-speed pavements, RCCP is usually left unsurfaced, the surface in these cases resembling that of a weathered asphalt pavement. Abrasion resistance is satisfactory.

For high-speed applications, a bituminous surface treatment may be applied to improve the skid resistance to an acceptable level, eg in France, for medium-traffic roads, a coating of fine gravel emulsion consisting of about 0,6 ℓ/m^2 of residual bitumen and 4 to 6 mm gravel, is applied to RCCP on the day of construction.

For major roads on which good rideability is especially important, a bituminous layer is placed on the surface to improve smoothness.

Where a bituminous layer is placed on the surface of RCCP, as in Spain and France, the surfacing is placed after a delay of one to two months to permit shrinkage cracking to occur. During this period the pavement is surfaced with emulsion, as described above.

Curing

RCCP requires early and effective curing, particularly since little free water is available in the mix. Curing should be maintained for a minimum period of seven days. The following alternative methods are used.

- The RCCP is fog-sprayed for 24 hours, followed by continuous water spraying from a sprinkler system. Fog spraying is implemented immediately after rolling; drying of the RCCP must not be allowed. In some cases wet burlap and moist sand have been used to assist curing.

- Water is applied in the form of a fine power-driven spray from a water tanker, taking into account the need to keep the pavement surface continuously wet.[24.13]

The use of bituminous or asphalt emulsion curing agents without initial moist curing or after 24-hour moist curing, has also been successful. On the other hand, curing compounds of the type used with conventional concrete paving have been found to be inadequate because they do not completely cover the surface.

Reflection cracking

In jointed RCCP it is likely that the joints will reflect through an asphalt surfacing. This may also occur in sprayed seals, but depends on the type of bitumen used.

Long-term experience on heavily-trafficked jointed concrete roads in Australia has shown that such cracks may not lead to undue distress or maintenance of the asphalt. Various crack-alleviating materials (eg bitumen-impregnated fabrics and geofabrics) placed across joints before asphalt paving can be used to minimise cracking.

Opening to traffic

On completion of compaction, light vehicles not exceeding 300 kg gross mass and travelling at slow speed may be permitted to use the pavement. Heavy traffic should be kept off for at least seven days; if this is not possible, traffic speeds should be very low and carefully monitored to limit damage to the pavement surface.

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Chapter 25
No-fines concrete

Steve Crosswell

25.1 Introduction

No-fines concrete (NFC) consists of coarse aggregate and cement paste. See Figure 25.1. Aggregate particles are each covered with a thin layer of cement paste and are in point-to-point contact with each other. At each point of contact the paste forms a small fillet; in the hardened state these fillets hold the particles together and give strength to the concrete.

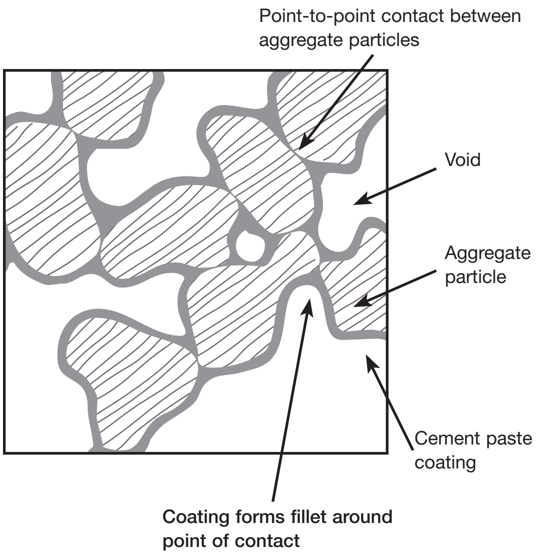

Figure 25.1: The structure of no-fines concrete

NFC is characterised by:

- A porous structure with relatively large interconnected voids distributed throughout
- Density and drying shrinkage values lower than those of ordinary concrete

NFC is used in ways that exploit these properties.

This chapter deals with applications, materials, mix proportions, manufacture and properties.

25.2 Applications

NFC is suitable for the following applications:

- Drainage layers; under reservoir floors; behind retaining walls (placed in situ or as precast blocks); beneath basement floors; in subsoil drains ("French" drains); in combined storm-water and subsoil drains (as cast-in-place pipes or channels)
- Backfill: between walls and excavated ground
- Stabilisation of earth surfaces: as wave and current protection; as earth-slope protection (grass will grow through the no-fines cover)
- Porous dams: for soil conservation and donga reclamation (gravity section or arched)
- Floors: base course for light duty, especially domestic floors
- Damp-proof coursing: in walls and under floors
- Roof screeds: insulating; below waterproofing system
- Walling: for system-built housing and garden walls (cast in place)
- Low retaining walls: for gardens (can be overgrown with grass and plants)
- Paving: for walkways, tennis courts and play areas - usually smoothed by a topping which may itself be permeable for quick drainage. (NFC is not suitable for paving to take vehicular traffic as its flexural strength is too low.)

Cautionary note: In some parts of South Africa, notably the mountainous regions of the Western Cape Province, ground waters may be soft or acidic, or both. NFC drains are not recommended under such conditions because leaching of calcium hydroxide from the NFC weakens the concrete; the downstream deposition of the leached lime can block outlet pipes, apart from causing an unsightly stain below weep-holes.

25.3 Materials

25.3.1 Cement

CEM I 42,5N and CEM II 32,5N (or higher) complying with SANS 50197-1 are recommended for all normal uses.

CEM I 42,5R and CEM I 52,5 are useful where high early strength or reduction of curing time is required.

CEM III, CEM IV and CEM V type cements should be used with caution, as should equivalent site-blended cements that contain high proportions of extenders and consequently have slower strength development. They also require longer periods of damp-curing and, because NFC tends to dry out quickly, this becomes impracticable.

C&CI currently has no information on the performance of CEM III, CEM IV and CEM V cements in NFC.

Masonry cements should not be used for NFC.

25.3.2 Water

Water suitable for making ordinary concrete should be used. (See Chapter 4.)

25.3.3 Aggregates

While a range of materials can be used in NFC, they should fulfil the following basic requirements:

- All particles should be more or less of the same size and there should be a minimum of elongated, flat or flaky particles. This will ensure that the packed aggregate has an open structure.
- All particles must be sufficiently strong for the purpose envisaged and must be durable and stable. The aggregate must not crush under the loads that will be imposed on the concrete, nor must the particles break down during the mixing process.
- The surface of the particles must be clean and strong to enable hardened cement paste to adhere firmly.

It is an advantage if the aggregate has a low absorptiveness. However, if a porous and absorbent aggregate must be used, it should be saturated beforehand by soaking in water and then should be allowed to drain for about an hour before use.

The most commonly used aggregate is single-sized crushed stone.

Crushed clinker or hard ash (resulting from the burning of coal in industrial furnaces) is normally variable and unreliable, and may degrade in service; it may be used only if found acceptable by laboratory testing. Mixes rich in cement are usually necessary with clinker aggregates.

NFC has been made successfully with aggregates ranging in nominal size from 6,7 mm to 75 mm, but by far the most common material is a nominal 19-mm stone.

The minimum thickness of NFC should be not less than about five times the nominal size of the aggregate. With aggregates smaller than 19 mm, cement consumption for a stated strength is appreciably higher than that of concrete made with 19-mm aggregate, while the use of aggregates larger than 19 mm reduces cement consumption slightly.

Mixes made with small aggregate – nominal size 13,2 mm or 9,5 mm – are easier to place and give a relatively fine surface texture.

25.4 Mix proportions

The practical limits of mix proportions are 200 to 500 ℓ of aggregate per 50-kg bag of cement. For normal construction, however, recommended volumes of aggregate per bag of cement range from 300 ℓ for a concrete requiring moderate strength, to 400 ℓ for one where strength is of lesser importance, to 450 ℓ where maximum economy is required (see Table 25.1).

The amount of mixing water used is critical. It depends mostly upon the size, shape, surface texture and absorptiveness of the aggregate. The correct amount to use is that which is just sufficient to enable the cement-water paste to coat the

Table 25.1: Recommended range of mix proportions, and resulting characteristics, of NFC made with 19-mm aggregate (Data from C&CI tests.)

Aggregate per 50 kg bag of cement, ℓ	Aggregate:cement ratio		Compressive strength, MPa		Cement content, kg/m^3	Water content, ℓ per	
	By mass	By volume	At 7 days	At 28 days		m^3	50-kg bag of cement
300	8,6	9,0	3,4	4,8	175	68	19
350	10,0	10,5	2,7	4,1	150	62	21
400	11,4	12,0	2,2	3,6	135	56	21
450	12,8	13,5	1,9	3,1	120	52	22
Aggregate requirement = 1,05 m^3/m^3 approximately							

Table 25.2: Batch quantities for NFC, made with 19-mm aggregate, to suit mixer size (Mixes as in Table 25.1)

Aggregate per 50-kg bag of cement, ℓ	300	350	400	450
For every 100-ℓ capacity of mixer, use:				
Volume of aggregate, ℓ	100	100	100	100
Volume of cement measured in the loose state, ℓ	13	11	9,5	8,5

aggregate uniformly with a layer of paste that runs together at the points of contact of aggregate particles. Too much water yields a paste that is too fluid and therefore tends to flow off the aggregate particles, weakening the bond in the upper portion of the work and filling the voids in the lower part. Too little water, on the other hand, results in a paste that does not coat the aggregate particles properly.

For given materials and proportions, the amount of water required differs very little from batch to batch. It is therefore recommended that, at the start of the work, the exact amount of mixing water required per batch be established by trial and inspection, and that this amount of water be supplied to all subsequent batches without reliance on the judgement of the mixer operator.

25.5 Manufacture

The manufacture of NFC should generally follow normal good practice for concrete-making. Aspects of special interest or difference are dealt with in the following sections.

25.5.1 Batching and mixing

Machine mixing is preferable to hand mixing. The batch size is governed by the capacity of the mixer; the volume of aggregate per batch should not exceed the rated capacity of the mixer. (See Table 25.2.)

If it is necessary to mix by hand, each batch should consist of one whole bag of cement and the appropriate volume of aggregate as given in Table 25.1. The cement and the water should be mixed to a thick slurry in a vessel such as a cut-down oil drum and then poured over and mixed with the aggregate on an impervious surface.

25.5.2 Placing

NFC should be placed in the forms with the least possible delay after mixing, as evaporation of water tends to be rapid.

The concrete should be deposited in, or as near as possible to, its final position. It should be placed in layers of about 150 mm deep, each layer being rodded sufficiently to force the aggregate particles firmly together and into full contact with the forms. Heavy tamping is not required, and on no account should vibrators be used.

Note that as freshly mixed NFC is not plastic or fluid, direct access to all parts of the formwork is necessary during compaction.

25.5.3 Construction joints

Construction joints, and "cold" joints, are highly undesirable in NFC construction because cement paste tends to accumulate and strength is reduced at such joints. These joints should be avoided especially in walling that is to be weatherproof and in load-bearing NFC. Where joints must be provided, they should be so arranged that they are subjected only to compressive stresses.

The position of construction joints should be planned beforehand, having particular regard to placing schedules and design of formwork. Sloping construction joints should not be permitted.

25.5.4 Formwork

Formwork for NFC must be designed on the same principles as those for normal concrete. The horizontal pressures exerted on formwork by NFC may be taken as one third of those for a normal concrete made with the same coarse aggregate and placed under similar conditions.

Formwork for NFC need not be watertight – it can be made of wire or other mesh on light timber frames, provided that the mesh is fine enough to retain the aggregate. However, the cement-water paste of the concrete tends to embrace and harden around the wires of the mesh, and damage to the mesh usually results when it is stripped from the concrete. Formwork with a smooth interior surface is therefore to be preferred, as is formwork which is wind-proof, to prevent the concrete from drying too quickly.

In view of the difficulties of making strong and weatherproof construction joints in NFC, formwork for the walls of dwelling houses should be of full storey height and should, if possible, extend right round the whole length of the exterior walls, so that these can be placed in a single uninterrupted operation.

Since NFC has virtually no cohesiveness, forms must remain in place until the cement paste has developed sufficient strength to hold the aggregate particles in place. Both the shape of the placement and its height therefore affect striking times. For vertical faces, striking times may be taken from Table 25.3 which refers to walls of storey height.[25.1]

Table 25.3: Striking times for formwork for walls of NFC (when using CEM I or CEM II A)

Air temperature during curing of concrete, °C	Minimum time of striking	Minimum time before applying load
Not lower than 20	24 hours	3 days
Between 10 and 20	2 days	5 days
Between 5 and 10	3 days	10 days
Below 5	Special precautions must be taken	

Note: When CEM IIB, CEM III, CEM IV and CEM V cements are used, these times should be increased to allow for the slower strength development of these cements. (See also section 25.3.1.)

25.5.5 Reinforcement

Because of the very low bond strength of NFC, tensile reinforcement is of limited effectiveness. The potential durability of reinforcement in NFC is doubtful because the concrete offers very little protection to embedded steel. The use of reinforcement in NFC is therefore not recommended.

25.5.6 Curing

As with normal dense concrete, adequate damp-curing is essential. Indeed, with NFC, particular attention should be paid to curing because drying winds can penetrate into the body of the concrete. At normal air temperatures (above about 15ºC) NFC should be kept damp for at least seven days after placing. At lower temperatures longer periods will be necessary.

Where the formwork is such that air cannot circulate through it, leaving the forms in place will prevent drying out and provide adequate curing. Drying out through exposed faces or open-textured formwork should be prevented by covering with damp hessian or with plastic sheeting.

25.5.7 Provision for services

It is difficult to cut chases or openings in NFC without damage to the adjacent work. Generally, it is best to prise out each particle of aggregate separately from the mass; this is a slow, laborious and, therefore, relatively expensive process.

To avoid these difficulties, suitable provision should be made for all services before placing the concrete.

25.5.8 Rendering and plastering

The rough, open-textured surface of NFC provides an excellent key for rendering or plastering. The concrete must be dry when the plaster coat is applied because there is little suction and the plaster tends to slip off wet paste surfaces. Normal plaster mixes are used on NFC.

Ideally, walls should be plastered as soon as the formwork is stripped, assuming the concrete is dry enough. This will prevent NFC from drying out too rapidly.

25.5.9 Screeding of NFC

Screeds are applied to NFC used for light-duty domestic floors; drainage layers under basement floors and reservoir floors; and insulating roof screeds.

Screed mixes are normally 1 bag of cement to 100 or 130 ℓ of concrete sand, mixed to a plastic consistence.

Screeding should be carried out as soon as the NFC layer is strong enough to withstand the screeding operation without damage. It is not possible to be definitive about the time, but in no case should more than 72 hours elapse before the screed is applied.

Screeding is done as soon as possible in order to retain moisture in the NFC layer and thus improve curing conditions and protect the NFC from physical damage and from contamination with sand, mortar, plaster or other detritus found on site.

Wet curing of screeds is strongly recommended as the screed can dry from both sides: into the NFC layer as well as into the atmosphere.

25.5.10 Weep-holes

Wherever an exterior wall of NFC rests on, or includes, a horizontal layer of relatively impermeable material, weep-holes must be left in the exterior rendering just above this layer to enable any moisture which may penetrate the rendering to drain away to the outside of the building. Such weep-holes are required above door and window lintel beams, solid floor slabs, solid foundation walls, membrane damp-proof courses, etc.

25.6 Physical properties

The physical properties of NFC are described below in general terms. This information is primarily for guidance since its applicability to specific cases cannot be guaranteed.

The figures given relate to NFC made in proportions within the range of the mixes shown in Table 25.1 unless otherwise stated, and are based on South African experience.

25.6.1 Compressive strength

The compressive strength of NFC is due to the strength of the fillets of hardened cement-water paste round the points of contact between the aggregate particles, and to inter-particle friction.

Hence, although the intrinsic strength of these fillets varies with water:cement ratio, this is not a major variable because of the need to keep the fluidity of the cement-water paste within quite narrow limits. Strength is controlled primarily by the number, size and orientation of the fillets.

The major factors controlling the strength of NFC made with given materials are therefore aggregate:cement ratio and degree of compaction.

With changes of materials, aggregate particle size and shape are likely to cause substantial shifts in the relationship between strength and aggregate:cement ratio.

Other variables are curing procedure and methods of test. Despite the apparent simplicity of these variables, the results of tests show considerable scatter, more than is found in the testing of dense concretes.

The strengths quoted in Table 25.1 were determined by direct testing in C&CI laboratories. The test methods were according to BS 1881: Part 113: 1983[25.2] except that the specified method of compaction was only simulated, and 100 mm instead of 150 mm cubes were used.

Tests by other South African organisations[25.3, 25.4] gave similar results, as did Jain[25.5] working with Indian materials in his comprehensive investigations.

In contrast, reports of British, American and Australian origin[25.6 - 25.8] give much higher strengths. For instance, the British tests for mixes of the same aggregate:cement ratio, gave strengths 60 to 100% higher, yet cement contents were the same and densities only 13% higher.

The cause of these large discrepancies is not apparent, but may be due in a large part to differences in test procedures, particularly degrees of compaction. One South African report[25.4] for instance recorded an increase of 33% in strength due only to increased compaction. The British tests covered a range of mixes of much lower aggregate:cement ratios than those in Table 25.1. The situation emphasises the need for extreme caution in applying overseas findings to South African conditions.

With regard to the effect of stone size on strength, the information available in South Africa is that the use of 9,5-mm stone instead of 19-mm stone results in an increase in strength of about 25%. This is probably due to the larger number of contact points (cement fillets) per unit volume.

Test methods for compressive strength

Cubes of NFC for compressive strength testing should preferably be made and cured in accordance with BS 1881: Part 113: 1983.[25.2] The preferred cube size is 150 mm.

Where the rather elaborate equipment for compacting samples is not available, it is suggested that compaction be by means of the standard square-faced "compacting bar" of BS EN 12390-2:2000[25.9], This bar is nominally 25 mm x 25 mm in section x 380 mm long. Cube moulds are filled in two layers and each layer should be tamped at least 12 times. Moulds must not be vibrated as the paste runs off of the stone and collects in the bottom of the mould. Testing of the cubes should be carried out as described in BS EN 12390-3:2002,[25.10] no capping of cubes nor any packing between the cube and the machine platens being required.

25.6.2 Other strengths

Since Jain's[25.11] compressive strength observations closely resemble South African findings, some of his observations are quoted:

"If f_c is the cube strength of NFC at 28 days, we have tensile strength = 0,12f_c ; flexural strength = 0,23f_c; cylinder strength = 0,61f_c; and bond strength = 0,19f_c. The maximum variation observed in these values was plus or minus 10%."

25.6.3 Density

The concrete of Table 25.1, made with stone of particle relative density 2,72, had an average density of 1 625 kg/m^3 within plus or minus 2,5%.

25.6.4 Drying shrinkage

There is widespread consensus that the drying shrinkage of NFC with stone aggregate is approximately one half of that of dense concrete. For estimating purposes therefore, drying shrinkage can be taken as 200 microstrain. It will be somewhat higher in NFC made with smaller stone.[25.11]

Because of the low total-water contents and open texture of NFC, it dries out rapidly. 50 to 80 % of full drying shrinkage can therefore occur within 10 days.[25.7]

25.6.5 Absorptiveness

NFC made of dense stone aggregate can absorb about 2,5% by mass of water on soaking for 24 hours.

25.6.6 Capillary action

Where NFC is allowed to stand in water, capillary action will cause the concrete to become damp for some distance above water level. With dense stone aggregate this distance will not exceed two and a half times the nominal size of the aggregate.[25.5] The thickness of damp-proof courses should therefore be not less than five times the nominal size of stone used.

25.6.7 Thermal properties

Linear thermal expansion

Because the aggregate particles in NFC are virtually in point-to-point contact and form a continuous lattice framework, the linear thermal expansion of NFC can be taken to be equal to that of the aggregate. (See 8.7.1 in Chapter 8.) A mean value of 9 microstrain per ºC can be used for most purposes.

Specific heat capacity

Based on the observed average density of the concretes in Table 25.1 (1 625 kg/m^3), the specific heat capacity of these concretes is 1 700 kJ/m^3.K, or 1,05 kJ/kg.K. Average specific heat is therefore 0,25.

The corresponding figures for a dense structural-grade concrete are 2 800 kJ/m^3.K; 1,15 kJ/kg.K; and 0,27, respectively.

Thermal conductivity

The thermal conductivity of NFC may be taken to be about 0,7 W/m.K. For comparison, the thermal conductivity of a dense structural-grade concrete is normally about 2,0 W/m.K.

Thermal diffusivity

The average thermal diffusivity of NFC is about 0,0015 m^2/hr. This is about 40 to 50 % of that of dense concrete.

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25.10 BS EN 12390 - 3:2002, **Testing hardened concrete – Part 3: Compressive strength of test specimens,** London: British Standards Institution, 2002.

25.11 Petersen, P.H. **Properties of a porous concrete of cement and uniform-sized gravel,** Washington: United States Government Printing Office, 1943. (Building Materials and Structures Report No. BMS 96).

Chapter 26

Sand-cement mixes – mortars, plasters, and screeds

Steve Crosswell

26.1 Introduction

Large quantities of cement are used in sand-cement mixes, even more possibly than in concrete, and serviceability failures of these mixes are fairly common, particularly in the cases of plasters and floor screeds. Typical problems with plasters and screeds include debonding, cracking, crazing, softness, poor abrasion resistance and unacceptable surface finish. As far as mortar is concerned common problems include softness, poor bond and leaching of lime from the mortar in face brick masonry.

This chapter covers site-batched mortars, plasters and sand-cement screeds. The chapter is split into two sections; the first section covering mortar and plaster, the second covering floor screeds. It is strongly recommended that the reader refer to the C&CI leaflets listed in Further reading. These are available from the website at www.cnci.org.za.

Proprietary ready-mixed sand-cement mixes are available in some parts of South Africa. These may be dry-mixed in bags, dry-mixed in bulk, or wet-mixed (ie supplied by a ready-mixed concrete supplier). It is recommended that the reader contact the relevant suppliers for advice on the uses and applications of these materials.

"Granolithic" screeds are not covered as they are classified as concrete toppings because they contain 6,7 or 9,5-mm stone.

Because of their specialist nature, cementitious grouts and adhesives are not included and reference should be made to specialist and proprietary literature.

26.2 Mortar and plaster

26.2.1 Materials

Cements

Cements for use in mortar and plaster should comply with SANS 50197-1[26.1] or SANS 50413-1.[26.2] Historically, in South Africa, cements equivalent to CEM I and CEM II A were used in mortars and plasters and caution was recommended when using cements equivalent to CEM II B, and CEM III. In some parts of southern Africa CEM IV and CEM V cements are now obtainable but little data is available regarding their service history in sand-cement mixes. In general it is recommended that users consult their cement suppliers for advice when using highly extended cements.

Masonry cements are available in some parts of South Africa. These cements must be used in accordance with the recommendations made in SANS 10164[26.3] and the relevant clauses of the NHBRC Home Building Manual.[26.4] They are not suitable for use in screeds, concrete toppings or concrete.

White cements are often used in swimming pool and other decorative plasters. White cement is not manufactured in South Africa, but imported cement must still comply with EN 197-1, the European standard from which SANS 50197-1 was derived, or prove compliance with SANS 50197-1. White cement is available in strength classes from 32,5N to 42,5R depending on source. As is the case with grey cement, the colour of white cement varies according to source.

Aggregates

Sand is by far the major constituent of sand-cement mixes and has a significant influence on their performance and material cost.

In South Africa, natural sands, ie pit, river and dune sands, are almost invariably used. An important requirement is that sand should be free of organic matter such as roots, seeds, twigs and humus. This is an absolutely essential requirement in the case of white or pigmented plasters and screeds.

Crusher sands are not generally suitable for use in mortar or plaster due to their angular particle shape. However, crusher sands have been used successfully in rich mixes for special applications such as plastering of squash court walls and pipe lining, and limestone or marble crusher sands are commonly used, with white cement, for plastering swimming pool shells.

"Karoo" sands, which contain a high proportion of disc-shaped dark-coloured particles, should not be used for plastering. This is because they exhibit excessive swelling and shrinkage on wetting and drying respectively. This causes excessive shrinkage cracking in the hardened plaster.

The use of furnace bottom ash as aggregate is also not recommended unless the ash has been thoroughly tested and proven to be sound.

Important physical properties of sands are:

- Clay content
- Grading
- Maximum particle size
- Particle shape

SANS 1090:2002,[26.5] the standard covering sand for plaster and mortar, gives limits for certain properties of sands but these should be regarded as no more than a guide. It has been found that sands meeting this standard do not necessarily produce satisfactory plaster; conversely sands that do not meet this standard may produce acceptable mixes.

Clay content

Only a small proportion of clay can be tolerated in sand used in sand-cement mixes. Clay normally causes a high water requirement and high drying shrinkage.

Sands with high clay content may generally be recognised as follows:

- The fraction that passes a 0,075-mm sieve can, after being moistened, be rolled into a thread about 3 mm or less in diameter.
- Mixes made with such sands:
    - Are very "fatty" and tend to cling to a trowel
    - Have a high water requirement

Specialist advice should be sought if there is any doubt about the content and type of clay in a sand.

Grading

Ideally, the sand should have a continuous grading, from dust up to the largest particles. The fractions passing the 0,150-mm and 0,075-mm sieves ("fines") are important because they significantly influence the water requirement, workability and water retentivity of the mix. Increasing these fractions results in increased water requirement (with consequent lower strength and higher shrinkage), but improved workability and water retentivity. The optimum fines content is therefore a compromise between these properties.

For mortars and plasters, a sand lacking in fines may be used with hydrated builder's lime, mortar plasticiser, or masonry cement, or it may be blended with a fine filler sand.

A sand with excessive fines may be improved by washing or by blending with a suitable coarser sand.

Recommended gradings for mortar and plaster sand are shown in Table 26.1.

Table 26.1: Recommended grading for mortar and plaster sand[26.5]

Sieve size	Percentage passing sieve by mass for plaster sand	Percentage passing sieve by mass for mortar sand
4,75 mm	100	100
2,36 mm	90 -100	90 -100
1,18 mm	70 -100	70 -100
600 µm	40 - 90	40 -100
300 µm	5 - 65	5 - 85
150 µm	5 - 20	5 - 35
Note: Some coarser material may be acceptable, or desirable, for textured decorative work.		

Maximum particle size

For conventional smooth plaster, all the sand should pass through a sieve with 2,36-mm square openings. For coarsely textured decorative work, and mortar, the corresponding sieve size is 4,75 mm.

Oversize particles (and lumps) should be removed by sieving.

Particle shape

Ideally, for good workability, the particle shape should be nicely rounded and the particle surface texture should be smooth. The particle shape of natural sands tends to be rounded due to weathering whereas that of crusher sands tends to be angular or flaky. Some river sands, however, contain newly weathered material with a rough surface texture and angular particle shape. These are normally in the coarser fraction of the sand and should be screened out.

Additives for mortar and plaster

A number of additives are used in the manufacture of sand-cement mixes. These include:

- Hydrated builder's lime
- Mortar admixtures, most commonly plasticisers
- Bonding aids
- Pigments
- Fibres

Hydrated builder's lime

In South Africa both calcitic lime ($Ca(OH)^2$) and dolomitic lime ($Ca(Mg)(OH)^2$) are available depending on location. Builder's lime must comply with SANS 523:2002[26.6] and is used to improve workability, plasticity and water retentivity of mortars and plasters. Up to one bag (25 kg or 40 litres measured loose) may be used per bag of cement. The quantity added depends largely on the fines content of the sand.

Lime tends to increase the water requirement of the plaster and hence reduce the compressive strength slightly, however the improved workability and water retention result in better bond and lower permeability.

As a rule, lime is more expensive than cement in South Africa. South African limes are non-hydraulic and cannot be used to replace cement in the mix.

Mortar admixtures

Chemical admixtures are often used in mortar and plaster mixes to improve workability and water retentivity. They should comply with BS EN 934-1:2008[26.7] and/or 934-3:2003.[26.8]

The most commonly used admixtures are so-called "mortar plasticisers" which are in fact air-entraining agents. The use of these admixtures must be properly controlled in order to avoid excessive air-entrainment and consequent loss of strength. On no account should they be used in concrete or with masonry cements.

Water-reducing plasticisers, retarders and accelerators may also be used in sand-cement mixes, as they are in concrete. Care should be taken with the use of retarders and accelerators in plasters due to their influence on finishing operations.

Bonding aids

Bonding aids are often used in plaster mixes, and in slurry mixes for bonding plasters to their substrates. These are normally based on rubber emulsions or acrylic polymers and should be used strictly in accordance with the manufacturer's instructions.

Some do not tolerate continual exposure to dampness and may re-emulsify under those conditions.

They can be very effective in improving bond but only if the substrate has been properly prepared. By no means are they a substitute for good surface preparation and workmanship. They also impart good workability to the mix but overdosing can lead to low strength.

Pigments

Pigments are used to impart colour to sand-cement mixes as well as to concrete and precast concrete products. Pigments should comply with BS EN 12878:2005[26.9] or an equivalent standard.

Pigments must be alkali-tolerant and ultraviolet-resistant and should not increase the water requirement of the mix unduly. Pigments for use with portland cements are available in powder and liquid forms in a variety of colours, and are normally inorganic metallic oxides.

Caution should be exercised when using carbon black as a pigment. Experience has shown that some grades leach from the cement matrix over time.

For any given combination of cement and sand, the dosage depends on the pigment colour and the desired final colour of the mix. Each pigment has a saturation dosage above which the colour of the mix remains the same. Dosages in excess of the saturation dosage waste pigment and can weaken the mix by increasing the water requirement.

For uniform colour the pigment should be thoroughly mixed with the cement prior to mixing with the sand. Machine mixing is preferable, if not essential.

Variegated colour effects are also possible by partially mixing one or more additional pigments into the mix.

Note that both the sand and cement colour can have a distinct effect on the final colour, as can the amount of mixing water added and it is therefore important, particularly for pigmented plasters, that both the sand and cement are obtained from one consistent source prior to commencing work, and that the batch quantities are carefully controlled.

As already mentioned, an absolutely essential requirement in the case of white or pigmented plasters is that the sand is free of organic matter in the form of roots, twigs, humus, seeds and plant resins as these can cause localised retardation, staining, and pop-outs.

On a practical note, a very important aspect to consider when specifying pigmented plasters is that it is impossible to patch them without the patch being visible. This means that all plumbing, electrical and other fixtures must be installed and tested prior to plastering or screeding.

Fibres

Fibres in the form of animal hair and straw have been used in mortars and plasters since ancient times. Modern fibres used in sand-cement mixes include glass fibres, polypropylene fibres and, in the case of shotcrete (gunite), various types of steel fibre. Fibre-reinforced cement (frc) is discussed in Chapter 21 and shotcrete (gunite) in Chapter 23.

26.2.2 Mix proportions for normal applications

Mortar

SANS 2001-CM1:2007[26.10] Table 7 prescribes the following mix proportions shown in Table 26.2, overleaf.

Note that the prescribed mix for class II mortar in the NHBRC Home Building Manual[26.4] when using masonry cement MC 12,5 is different, being 170 litres of sand to 50 kg of masonry cement.

Table 26.2: Prescribed mix proportions for mortar[26.10]

Cement type	Cement, kg	Lime, kg	Sand, loose damp volume, ℓ	Sand, standard wheelbarrows
Class I mortar				
Common cements	50	0 - 10	130	2
Masonry cement MC 22,5X, MC 12,5	50	Not permitted	80	1
Class II mortar				
Common cements	50	0 - 25	200	3
Masonry cement MC 22,5X, MC 12,5	50	Not permitted	130	2

SANS 2001-CM1:2007[26.10] and SANS 10164[26.3] also permit the use of mortars designed for compressive strength as an alternative to prescribed mix proportions. Table 26.3 shows the strengths specified in Table 8 of SANS 2001.

Table 26.3: Compressive strength requirements for mortar[26.10]

Mortar class	Minimum 28 day compressive strength, MPa	
	Preliminary (laboratory) tests	Works tests
I	14,5	10
II	7	5

Plaster

SANS 2001-EM1:2007[26.11] Table 1 prescribes the following mix proportions for plaster for normal applications:

Table 26.4: Prescribed mix proportions for plaster[26.11]

Type	Common cement, kg	Lime, kg	Masonry cement MC 22,5X or MC 12,5	Masonry cement MC 12,5X	Sand, loose damp volume, ℓ	Sand, standard wheelbarrows
External plaster	50	0 - 25	-	-	150	2,5
	-	-	50	-	130	2
Internal plaster	50	0 - 25	-	-	200	3
	-	-	50	-	150	2,5
	-	-	-	50	100	1,5

Note that the NHBRC Home Building Manual[26.4] does not permit the use of MC 22,5X or MC 12,5X in plaster work for residential construction.

26.2.3 Properties of fresh mortar and plaster

Water requirement

This is the amount of water needed per unit volume of mix to bring it to the required consistence. It is normally expressed in ℓ/m^3.

The water requirement of a mix is significant for two reasons:

- For a given cement:aggregate ratio it directly affects the w:c ratio which in turn determines the potential strength and impermeability of the hardened mortar.
- Drying shrinkage is determined largely by the water content of the fresh mix. For given materials and conditions, the higher the water content, the higher the drying shrinkage and the greater the tendency for sand-cement mixes to crack.

Water requirement for a given consistence is determined by the same factors as for concrete and is discussed elsewhere.

As a guide, for mixes in the range covered in this chapter, a poor-quality sand has a water requirement greater than 400 ℓ/m^3, an average-quality sand around 350 ℓ/m^3, and a good quality sand around 270 ℓ/m^3.

Workability

Workability is the relative ease with which the mix can be placed, compacted and finished. It is the result of intrinsic requirements such as cohesiveness, consistence, plasticity, thixotropy and finishability, and extrinsic factors such as thickness and orientation of application.

Cohesiveness, or stickiness, is the degree to which the mix clings together in a continuous mass or clings to tools such as trowels or to surfaces to which it has been applied. The cohesiveness of mortar is generally superior to that of concrete because of the absence of large stone particles.

Consistence describes mobility or ease of flow and is related to the wetness or dryness of the mix. For a given material it depends largely on the water content of the mixture.

Plasticity is the degree to which a mass of mortar remains in a continuous mass while being moulded or extruded. Plasticity is an important property of sand-cement mixes in applications where it is extruded in the placing process. Examples are bricklaying and plastering where extrusion is effected by the brick and the trowel respectively.

Thixotropy is the property of becoming temporarily liquid when shaken, stirred or otherwise agitated, and returning to a gel state on standing.

Finishability is required of plaster mixes in some applications so that the surface can be floated without tearing or dislodging coarser particles.

Thickness of application, relative to the size of the largest particles in the mix, also influences workability. Thicknesses less than three to four times the maximum particle size are difficult to place.

Orientation of application refers to the angle of the surface, relative to horizontal, to which the mix is applied. Vertical, inclined and overhead surfaces require a greater degree of cohesiveness and thixotropy than horizontal surfaces.

Plastic shrinkage

If a sample of compacted sand-cement mix loses water while in the plastic state, overall volume is reduced and shrinkage will result. If this shrinkage is restrained, for example by the substrate, cracking of the plaster or screed will result. Factors which increase the likelihood and severity of plastic shrinkage cracking include:

- A high initial water content of the mix
- Highly absorbent rigid background or substrate
- Drying conditions such as exposure to direct sunlight and hot, dry winds

Plasters are very prone to plastic-shrinkage cracking due to their large surface-area-to-volume ratio.

Water retentivity

This property is particularly important in mortars used for bedding of masonry units. When a trowelful of mortar is placed on the already-laid units, it should retain its workability long enough to allow bedding down of the next course. If the units are dry and absorbent they will tend to extract water from the mortar. Fired clay stock bricks are absorbent and it is good practice to wet them prior to laying in order to reduce suction. Clay face bricks on the other hand tend to be less absorbent and should be laid dry.

Concrete bricks and blocks should be laid dry to reduce the possibility of cracks due to drying shrinkage, and therefore mortars used with concrete masonry units must have good water retentivity.

Water retentivity is a function of the content and nature of the fines in the mortar. Sands with a high clay content have good water retentivity but high water requirements and subsequent drying shrinkage.

The water retentivity of mortars made with sands that lack fines may be improved by the addition of hydrated builders lime, rock flour (if available) or air-entraining agent (mortar plasticiser). Alternatively masonry cement may be used instead of one of the common cements.

26.2.4 Properties of hardened mortar and plaster

Strength

Compressive strength is measured in the same way as for concrete, by making cubes and testing them in accordance with SANS 5863.[26.12]

As with concrete, the main determinants of measured compressive strength are water:cement ratio, compaction and curing.

For mortars and plasters typical compressive strengths lie in the range of 5 to 15 MPa, which is adequate for most applications. Lower strengths are undesirable for reasons of high porosity and poor adhesion, while stronger mixes tend to shrink and crack more, which is also undesirable.

There are exceptions to the above, for example when plastering soft clay bricks or squash court walls, where lower and higher strengths respectively are desirable.

Adhesion

The adhesion between a sand-cement mix and a background surface is important in most applications. Bedding mortar should adhere well to the bricks on either side of the joint; and plaster should adhere well to walls and ceilings.

Factors which promote good adhesion between cement-based materials and backgrounds are:

- Cleanliness of the background
- Roughness of the background
- Optimum absorptiveness ("suction") of the background: it should take up some of the mixing water from the fresh mix, but not so much as to dry out the mixture.
- Strength compatibility of the background and the applied material: strong applied material on a weak background can cause debonding
- Good curing of the applied material to ensure a high degree of cement hydration
- Smallest possible relative movement between background and applied material, which in practice means using a mixture with a low drying shrinkage

This last factor is significant as sand-cement mixes have considerably higher drying shrinkage than concrete of the same strength. Where adhesion is critical, materials for the applied mixture should be selected for the lowest possible water content that allows full compaction.

Permeability

The permeability of sand cement mixes depends on the same factors as those that affect concrete:

- W:C ratio
- Compaction
- Degree of curing

Low-strength mortars and plasters which are poorly compacted and cured can be expected to have high permeability.

High-strength, well compacted and cured mixes, as used in ferro-cement boat hulls for example, are highly impermeable and show excellent durability.

Dimensional stability

Sand-cement mixes exhibit higher drying shrinkage, higher creep, and lower stiffness than concrete. This is because they contain higher water and cement contents per unit volume and are not restrained by coarse aggregate.

While higher shrinkage tends to cause cracking when restrained, the higher creep and lower stiffness have the opposite effect and reduce cracking somewhat.

26.2.5 Practical assessment of sand quality and workability of mixes

Assessing the suitability of sand for use in mortar and plaster

A sand may be assessed by doing both of the following:

- Comparing grading and maximum particle size and, if necessary, apparent clay content, with the recommendations given.
- Making a mix to assess water requirement and workability. This may be done as follows.
  - (i) Weigh out the following amounts of material (or suitably scaled down proportions):
    - 5 kg of cement
    - 25 kg of dry sand
    - 5 kg (ℓ) of water
    - 1 kg (ℓ) of water
    - 1,5 kg (ℓ) of water
  - (ii) Mix the cement and sand to a uniform colour on a non-absorbent surface.
  - (iii) Add, in succession, each of the amounts of water (5 ℓ, 1 ℓ and 1,5 ℓ) and mix until the mix reaches a consistence suitable for plaster.
    - If 5 ℓ of water is enough, the sand is of good quality
    - If 5 ℓ + 1 ℓ is enough, the sand is of average quality
    - If 5 ℓ + 1 ℓ + 1,5 ℓ is enough, the quality of the sand is poor
    - If more water than that is required, the quality is very poor

    Only "good" sands are suitable for use in all mortar and plaster work.

    "Average" sands may be used for mortar, for interior plaster, and screeds.

    "Poor" and "very poor" sands are not recommended and should be avoided if at all possible.
  - (iv) Assess the workability as described below.

SANS 2001-EM1:2007[26.11] outlines a similar test and states that "sands shall be such that when 2,5 kg of cement is mixed with 12,5 kg of dry sand, no more than 2,7 litres of water is required to bring the mix to a plastering consistency."

Assessment of workability

As is the case with concrete there is no satisfactory test to measure the workability of sand-cement mixes. Workability is therefore assessed by qualitatively assessing the contributory properties above.

Mortar and plaster mixes may be assessed as follows:

- Make up a mixture of sand, cement and water, and lime or admixture where appropriate, to the required proportions. Add the water slowly and mix until the consistence is considered satisfactory.
- Place some of the mix, about 100-mm thick and 200-mm diameter, on a flat non-absorbent surface. Holding the blade horizontal, try to force a trowel down towards the surface. If plasticity is good, the mix will extrude easily around the perimeter of the blade and it will be possible to push it almost on to the surface.
- Place some of the mix on a trowel, tap the trowel a few times to compact the mix then turn it upside down. The mix should cling to the trowel but should fall off if it is shaken.
- The finishability of a plaster can be assessed by floating the surface of some of the mix with the trowel. The surface should not tear if the finishability is good.

26.2.6 Applications

Mortar

The most common application of mortar is to bind masonry units together in order to build structurally sound weatherproof walls. High-strength mortar is also used as a repair material to repair certain defects in concrete.

Class I mortar (see section 26.2.2) is normally used for highly stressed masonry or in severe exposure conditions, for example foundation walls below ground. Class II mortar is used for normal building work.

In some cases, for example when laying weak poorly burnt bricks, it may be necessary to use a leaner mix than class II mortar. In such cases a mix of one bag of cement to two bags of lime to 300 ℓ of damp sand is recommended.

Plaster

Plaster is used to protect the underlying masonry and for decorative purposes. It is also often used as an applied finish to concrete walls, columns, beams and ceilings. In all cases the surface preparation prior to plastering is critical to achieve good adhesion.

Under certain circumstances, for example when restoring old buildings built with poorly burnt bricks, lean plaster mixes with high lime contents are recommended. A typical mix would be one bag of cement to two bags of lime to 300 ℓ of damp sand.

Details on plastering practice can be found in Further reading at the end of this chapter.

More specialised applications, which are outside of the scope of this chapter, are:

- Plastering squash court walls
- Swimming pool linings
- Pipe lining and coating
- Improving fire resistance
- Repairs

It is recommended that the reader consults specialist and proprietary literature in the case of these applications.

26.2.7 Common problems

Mortars

The most common problems found in mortars are:

- Lack of hardness
- Lack of adhesion to masonry units

Lack of hardness is caused by one or more of the following:

- Insufficient cement in the mix.
- Re-tempering (remixing with more water when the mortar starts to stiffen).
- Rapid drying in hot, windy weather.
- The use of a sand with a high water requirement.
- The use of a sand with poor water retention properties.

Lack of adhesion is less common, but may be caused by much the same factors.

Plasters

Defects most frequently noticed in plaster fall into one of the following categories and are discussed in detail in reference 26.13.

- Non-structural cracks
- Structural cracks
- Debonding
- Lack of hardness
- Grinning
- Expansion
- Popping

Other frequent problems are irregular surface finish and inconsistent surface texture.

Note that gypsum-based plaster must never be mixed into plaster made with portland cement. This is because gypsum is a sulphate compound which attacks portland cement paste, especially in damp conditions. This attack causes swelling, softening and disintegration of the plaster.

26.2.8 Estimating quantities and wastage

Table 26.5: Mortar quantities for masonry, Class II mortar.

Cement	Lime	Sand, damp and bulked
2 bags	0 - 2 bags	6 wheelbarrows
2 bags	0 - 80 ℓ	400 ℓ
Depending on wastage this is enough mortar to lay approximately: • 600 bricks or maxi bricks in a cavity or single leaf wall • 500 bricks or maxi bricks in a solid 230-mm wall • 600 hollow blocks (390 x 90 x 190 mm) • 400 hollow blocks (390 x 140 x 190 mm) • 300 hollow blocks (390 x 190 x 190 mm)		

Table 26.6: Plaster quantities, Class II mix.

Cement	Lime	Sand, damp and bulked
2 bags	0 - 2 bags	6 wheelbarrows
2 bags	0 - 80 ℓ	400 ℓ
Depending on wastage this is enough plaster to cover approximately 20 m^2 at a thickness of 15 mm.		

Wastage is difficult to estimate as it depends on site control, workmanship, and type of masonry. It can be as high as 40%, but a reasonable figure to use when estimating would be of the order of 25%.

26.3 Sand-cement floor screeds

26.3.1 Materials

Cements

Cements for use in screeds should comply with SANS 50197-1.[26.1] Historically, in South Africa, cements equivalent to CEM I and CEM II A were used in screeds, and caution was recommended when using cements equivalent to CEM II B and CEM III. In some parts of southern Africa CEM IV and CEM V cements are now obtainable but little data is available regarding service history in sand-cement screed mixes. In general it is recommended that users consult their cement suppliers for advice when using highly extended cements.

Masonry cements are not suitable for use in screeds.

White cements are often used in decorative floor screeds. White cement is not manufactured in South Africa, but imported cement must still comply with EN 197-1, the European standard from which SANS 50197-1 was derived, or prove compliance with SANS 50197-1. White cement is available in strength classes from 32,5N to 42,5R depending on source. As is the case with grey cement, the colour of white cement varies according to source.

Aggregates

Sand is by far the major constituent of screed mixes and has a significant influence on their performance and material cost. In South Africa, natural sands, ie pit, river and dune sands, are almost invariably used. An important requirement is that sand should be free of organic matter such as roots, seeds, twigs and humus. This is an absolutely essential requirement in the case of white or pigmented screeds. Crusher sands are also sometimes used, blended with fine natural sands, in floor screed mixes.

"Karoo" sands, which contain a high proportion of disc-shaped dark-coloured particles, should not be used for screeds. This is because they exhibit excessive swelling and shrinkage on wetting and drying respectively. This causes excessive shrinkage cracking in the hardened screed.

The use of furnace bottom ash as aggregate is also not recommended unless the ash has been thoroughly tested and proven to be sound.

Important physical properties of sands are:

- Clay content
- Grading
- Maximum particle size
- Particle shape

Sands for use in screeds should comply with SABS 1083:2006[26.14] the standard covering aggregates for concrete.

Clay content

Only a small proportion of clay can be tolerated as it normally causes a high water requirement and high drying shrinkage. Sands with a high clay content may generally be recognised as follows:

- The fraction that passes a 0,075-mm sieve can, after being moistened, be rolled into a thread about 3 mm or less in diameter.
- Mixes made with such sands:
  - Are very "fatty" and tend to cling to a trowel
  - Have a high water requirement

Specialist advice should be sought if there is any doubt about the content and type of clay in a sand.

Grading

Ideally, the sand should have a continuous grading, from 4,75 mm down to dust. A sand with excessive fines may be improved by washing or by blending with a suitable coarser sand.

Recommended gradings for concrete sand are shown in Table 26.7.

Table 26.7: Recommended grading for concrete sand[26.14]

Sieve size	Cumulative percentage passing	
	SABS 1083:1994	Preferred limits
4,75 mm	90 - 100	90 - 100
2,36 mm		75 - 100
1,18 mm		60 - 90
600 µm		40 - 60
300 µm		20 - 40
150 µm	5 - 25	10 - 20
75 µm	0 - 5 (10)*(1)	3 - 6 (6 - 15)*

* Crusher sands
(1) These limits may be exceeded if the methylene blue absorption value is < 0,7 and the clay content is < 2,0%

Maximum particle size

The maximum particle size for sands used in screeds should be 4,75 mm. Over-sized material should be removed by screening.

Particle shape

Ideally, for good workability, the particle shape should be nicely rounded and the particle surface texture should be smooth. The particle shape of natural sands tends to be rounded due to weathering whereas that of crusher sands tends to be angular or flaky.

Additives for screed mixes

A number of additives are used in the manufacture of screeds. These include:

- Mortar admixtures, most commonly plasticisers

- Bonding aids
- Pigments
- Fibres

Mortar admixtures

Water reducing plasticisers are sometimes used in screed mixes to improve workability.

Retarders and accelerators may also be used in sand-cement screed mixes, as they are in concrete. Care should be taken with the use of retarders and accelerators in screeds due to their influence on finishing operations.

Admixtures should comply with BS EN 934-1:2008[26.7] and/ or 934-3:2003.[26.8]

Bonding aids

Bonding aids are often used in screed mixes and in slurry mixes for bonding screeds to their substrates. These are normally based on rubber emulsions or acrylic polymers and should be used strictly in accordance with the manufacturer's instructions.

Some bonding aids do not tolerate continual exposure to dampness and may re-emulsify under those conditions.

They can be very effective in improving bond but only if the substrate has been properly prepared. By no means are they a substitute for good surface preparation and workmanship.

They also impart good workability to the mix but overdosing can lead to low strength.

Pigments

Pigments are used to impart colour to sand-cement screeds as well as to concrete and precast concrete products. Pigments should comply with BS EN 12878:2005[26.9] or an equivalent standard.

Pigments must be alkali-tolerant and ultraviolet-resistant and should not increase the water requirement of the mix unduly. Pigments for use with portland cements are available in powder and liquid forms in a variety of colours, and are normally inorganic metallic oxides.

Caution should be used when using carbon black as a pigment. Experience has shown that some grades leach from the cement matrix over time.

For any given combination of cement and sand the dosage depends on the pigment colour and the desired final colour of the mix. Each pigment has a saturation dosage above which the colour of the mix remains the same. Dosages in excess of the saturation dosage waste pigment and can weaken the mix by increasing the water requirement.

For uniform colour the pigment should be thoroughly mixed with the cement prior to mixing with the sand. Machine mixing is preferable, if not essential. Variegated colour effects are also possible by partially mixing one or more additional pigments into the mix.

Note that both the sand and cement colour can have a distinct effect on the final colour, as can the amount of mixing water added and it is therefore important, particularly for pigmented screeds, that both the sand and cement are sourced from one consistent source prior to commencing work, and that the batch quantities are carefully controlled.

As mentioned previously, an absolutely essential requirement in the case of white or pigmented screeds is that the sand is free of organic matter in the form of roots, twigs, humus, seeds and plant resins as these can cause localised retardation, staining and pop-outs.

On a practical note, a very important aspect to consider when specifying pigmented screeds is that it is impossible to patch them without the patch being visible. This means that all plumbing, electrical and other fixtures must be installed and tested prior to screeding.

Fibres

Fibres in the form of animal hair and straw have been used in mortars and plasters since ancient times. Modern fibres used in sand-cement mixes include glass fibres, polypropylene fibres and, in the case of shotcrete (gunite), various types of steel fibre. Fibre reinforced cement (frc) is discussed in Chapter 21, and shotcrete (gunite) in Chapter 23.

26.3.2 Mix proportions for normal applications

The recommended mix proportions for sand-cement floor screeds are 50 kg common cement to 130 ℓ of sand, measured damp and loose.[26.15]

Some publications recommend the use of a richer mix of 50 kg of common cement to 100 ℓ of sand when the screed is covered with a soft or flexible finish such a carpet or vinyl tile as opposed to a rigid tile finish.

Irrespective of the aggregate:cement ratio, sufficient water should be added to make a workable, plastic mix that can be fully compacted and trowelled to the required finish. The use of so-called "dry" or "semi-dry" mixes is a frequent cause of failure of sand-cement screeds.

26.3.3 Properties of fresh sand-cement screed mixes

Water requirement

This is the amount of water needed per unit volume of mix to bring it to the required consistence. It is normally expressed in ℓ/m^3.

The water requirement of a mix is significant for two reasons:

- For a given cement:aggregate ratio it directly affects the w:c ratio which in turn determines the potential strength and impermeability of the hardened mixture.
- Drying shrinkage is determined largely by the water content of the fresh mix. For given materials and conditions, the higher the water content, the higher the drying shrinkage and the greater the tendency for sand-cement mixes to crack.

Water requirement for a given consistence is determined by the same factors as those that affect concrete and is discussed elsewhere.

As a guide, for mixes in the range covered in this section, the water requirement of the sand should not exceed 320 ℓ/m^3.

Workability

Workability is the relative ease with which the mix can be placed, compacted and finished. It is the result of intrinsic requirements such as cohesiveness, consistence, plasticity, thixotropy and finishability, and extrinsic factors such as thickness and orientation of application. These factors are discussed in section 26.2.3.

Settlement and bleeding

Settlement and associated bleeding can occur in sand-cement screed mixes just as it can in concrete mixes, and the causes and remedies are the same as for concrete. It is not normally a problem with screed mixes except where excessive bleeding could cause finishing problems.

Plastic shrinkage

If a sample of compacted sand-cement mix loses water while in the plastic state, overall volume is reduced and shrinkage will result. If this shrinkage is restrained, for example by the substrate, cracking of the screed will result.

Factors which increase the likelihood and severity of plastic shrinkage cracking are:

- A high initial water content of the mix
- Highly absorbent rigid background or substrate
- Drying conditions such as exposure to direct sunlight and hot, dry winds

Screeds are very prone to plastic-shrinkage cracking due to their large surface-area-to-volume ratio.

26.3.4 Properties of hardened sand-cement screed mixes

Strength

In the case of screeds, somewhat higher strengths in excess of 20 MPa give more satisfactory performance. The strength of screeds is dependent not only on the mix proportions but also on the degree of compaction. While the determination of strength using cubes can be used, it is recommended that the quality of the in-situ screed be assessed using the BRE Screed Tester. The use of this instrument is described in detail in reference 26.16.

Adhesion

The adhesion between a screed and the concrete substrate is important in many applications, although un-bonded screeds are also used.

Factors which promote good adhesion are:

- Cleanliness of the background
- Roughness of the background
- Optimum absorptiveness ("suction") of the background: it should take up some of the mixing water from the fresh mix, but not so much as to dry out the mixture
- Strength compatibility of the background and the applied material: strong applied material on a weak background can cause debonding
- Good curing of the applied material to ensure a high degree of cement hydration
- Smallest possible relative movement between background and applied material, which in practice means using a mixture with a low drying shrinkage

This last factor is significant as sand-cement mixes have considerably higher drying shrinkage than concrete of the same strength. Where adhesion is critical therefore, materials for the applied mixture should be selected for the lowest possible water content that allows full compaction.

Permeability

The permeability of sand cement screed mixes depends on the same factors as that of concrete:

- W:C ratio
- Compaction
- Degree of curing

Where screeds are used as a decorative floor finish it is advisable to seal the surface with a proprietary sealant in order to prevent stains from spillages or breakages.

Dimensional stability

Sand-cement screed mixes exhibit higher drying shrinkage, higher creep, and lower stiffness than concrete. This is because they contain higher water and cement contents per unit volume and are not restrained by coarse aggregate.

While higher shrinkage tends to cause cracking when restrained, the higher creep and lower stiffness have the opposite effect and reduce cracking somewhat.

Abrasion resistance
While sand-cement screeds are not suitable to take heavy traffic, foot or otherwise, the reality is that many screeds are placed with the intention of taking traffic.

It has become more common to specify screeds as a decorative floor finish, either plain, or white, or pigmented. It is therefore important that such screeds have good abrasion resistance. Abrasion resistance is improved by using a sand with a low water demand, by fully compacting the screed, by steel floating and, finally, by good curing.

Sealing the surface with a proprietary sealant will also improve abrasion resistance.

26.3.5 Practical assessment of sand quality for screeds and workability of mixes

Assessing the suitability of a sand for use in a screed mix
SANS 10109-2:2004[26.15] states that the sand shall comply with SANS 1083:2006.[26.14]

In addition the sand, in a mix of 3,5 parts dry sand to one part of cement by mass, shall produce "a plastic, easily workable cohesive mortar of plastering consistence, with a water content not exceeding 320 ℓ/m^3."

Assessment of workability
As is the case with concrete there is no satisfactory test to measure the workability of sand-cement mixes. Workability is therefore assessed by qualitatively assessing the contributory properties above.

Mixes may be assessed as follows:
- Make up a mixture of sand, cement and water, and admixture where appropriate, to the required proportions. Add the water slowly and mix until the consistence is considered satisfactory.
- Place some of the mix about 100-mm thick and 200-mm diameter on a flat non-absorbent surface. Holding the blade horizontal, try to force a trowel down towards the surface. If plasticity is good, the mix will extrude easily around the perimeter of the blade and it will be possible to push it almost on to the surface.
- Place some of the mix on a trowel, tap the trowel a few times to compact the mix then turn it upside down. The mix should cling to the trowel but should fall off if it is shaken.
- The finishability of a screed can be assessed by floating the surface of some of the mix with the trowel. The surface should not tear if the finishability is good.

26.3.6 Applications

Sand-cement screeds are intended for light-duty traffic only. They are not suitable for use in industrial environments. They are used to achieve the correct level, flatness, and falls if necessary, on (usually) concrete floors and roofs.

It has become more common to specify decorative screeds for domestic and light commercial applications. These may be plain grey, white, or pigmented, as mentioned above.

26.3.7 Common problems

The four most common problems found with floor screeds are:
- Breaking up of the surface
- Disintegration of the screed itself
- Dusting of the surface
- Debonding

The first two are related to the common practice of making the screed mix too dry, not compacting it fully, and then trowelling a cement-water slurry into the surface to give a hard, smooth surface. The cement-rich slurry mix has a very high drying shrinkage and soon crazes and debonds, whereupon the poorly compacted material below disintegrates under traffic.

Dusting of the surface is normally related to poor curing but may also be caused by using too lean a mix.

Small areas of debonding may not necessarily cause serviceability failure of a screed, but large areas are undesirable unless the screed is designed as a unbonded screed. Debonding is usually caused by inadequate surface preparation or incorrect use of bonding aids.

26.3.8 Estimating quantities and wastage

Wastage is less than for mortar and plaster, possibly of the order of 20%. In the case of screeds (as with plaster) a small variation in thickness over a large area can have a significant effect on the quantities used.

Table 26.8: Quantities for sand-cement floor screeds

Mix	Cement (bags/m^3 screed)	Sand (m^3/m^3 of screed)
1 bag to 130 ℓ sand	9	1,2
1 bag to 100 ℓ sand	11	1,1

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Chapter 27

Concrete repair

Hans Beushausen and Mark Alexander

27.1 Introduction

Concrete is a durable construction material and, if designed and placed properly, will give long service under normal conditions. However, a large number of concrete structures are deteriorating, often prematurely, and require remedial measures to reinstate their safety and/or serviceability. Consequently, the need for repair and protection has grown considerably in recent years. Deterioration may be a result of numerous factors, including design, workmanship, materials used, environmental influences during construction and service life, and loads acting on the structure. One of the main reasons that reinforced concrete structures deteriorate is insufficient cover depth to reinforcing steel, often in combination with poor concrete cover quality and exposure to harsh environments, resulting in reinforcement corrosion and associated cracking and spalling.

While costs associated with repair of deteriorating concrete structures can be substantial, costs resulting from poorly designed or executed repairs may be even higher. The technical and economical success of repair projects depend on a range of factors, including a proper condition assessment of the structure, design and execution of remedial measures, and design and implementation of maintenance strategies.

At the beginning of a repair project, a systematic condition assessment of the structure needs to be carried out to identify the cause(s) of deterioration and the extent of damage. The condition assessment typically starts with visual inspection by an experienced engineer and in many cases would include on-site testing of the state of the structure accompanied by laboratory-based examination of material properties of samples removed from the structure. Repair methods need to be designed with consideration for the anticipated or desired remaining service life of the structure. A distinction must be made between repairs intended to stop deterioration fully and those merely aimed at slowing down deterioration processes for a limited period of time. It must be understood that in most cases a once-off repair will not be successful in reinstating long-term serviceability and that commonly a detailed maintenance strategy must be developed for the remaining lifetime of the structure.

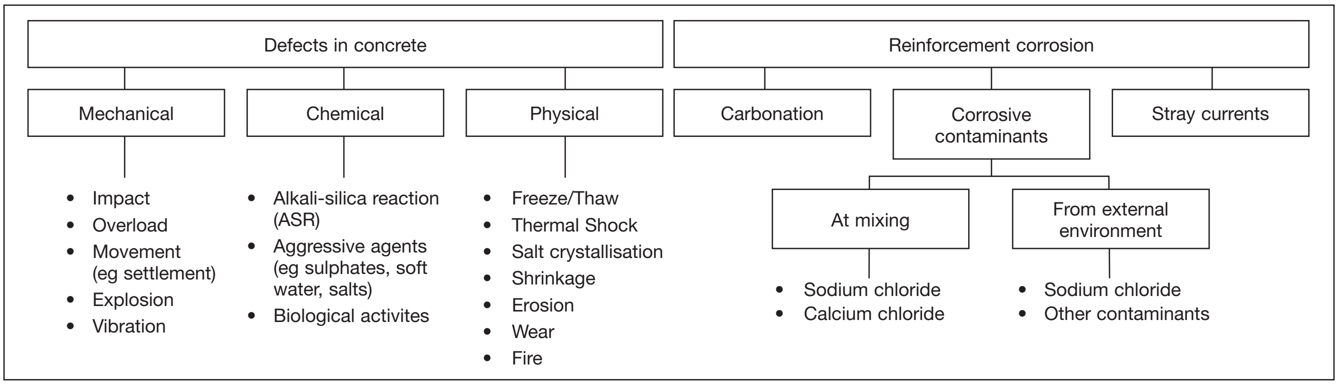
This chapter presents an overview of important topics associated with the field of concrete repair. Condition assessment strategies and common diagnostic test methods, as well as repair principles, methods and materials are discussed. A list of selected references is presented at the end of this chapter for those who would like to explore the topic in more detail. Information presented in this chapter is partly adopted from Mackechnie and Alexander.[27.1]

27.2 Condition assessment of concrete structures

27.2.1 Methodology

Clients and design engineers must understand that a repair project can only be successfully completed if it includes a thorough condition assessment of the structure under consideration. The condition assessment should generally distinguish between defects in the concrete itself and defects caused by reinforcement corrosion.[27.2] The selection and specification of repair methods and strategies must consider the cause (or causes) of deterioration and the extent

Table 27.1: Common defects in concrete [27.2]



of damage experienced. The assessment must be based on sound understanding of the physical, chemical and mechanical actions that lead to deterioration of concrete and concrete structures. Common causes of defects in concrete structures are listed in Table 27.1.

The first step is the planning of appropriate assessment activities and the gathering of available information such as structural, environmental and service data, and, if available, information from previous investigations and tests. The second step is a detailed visual inspection under guidance of a structural engineer, through which information on basic deterioration mechanisms can be obtained. In most cases, the prevailing cause of damage can be identified through crack patterns and the appearance of stains and chemical reaction products on the concrete surface. Visual assessment is the cheapest type of inspection and, with experienced staff, can provide most of the relevant information needed.[27.3]

Based on information from the visual assessment and existing data, a strategy for in-situ and laboratory-based testing can be designed, if necessary. Structural assessment may be required, involving analysis of the load bearing capacity in cases where this may have been lowered by

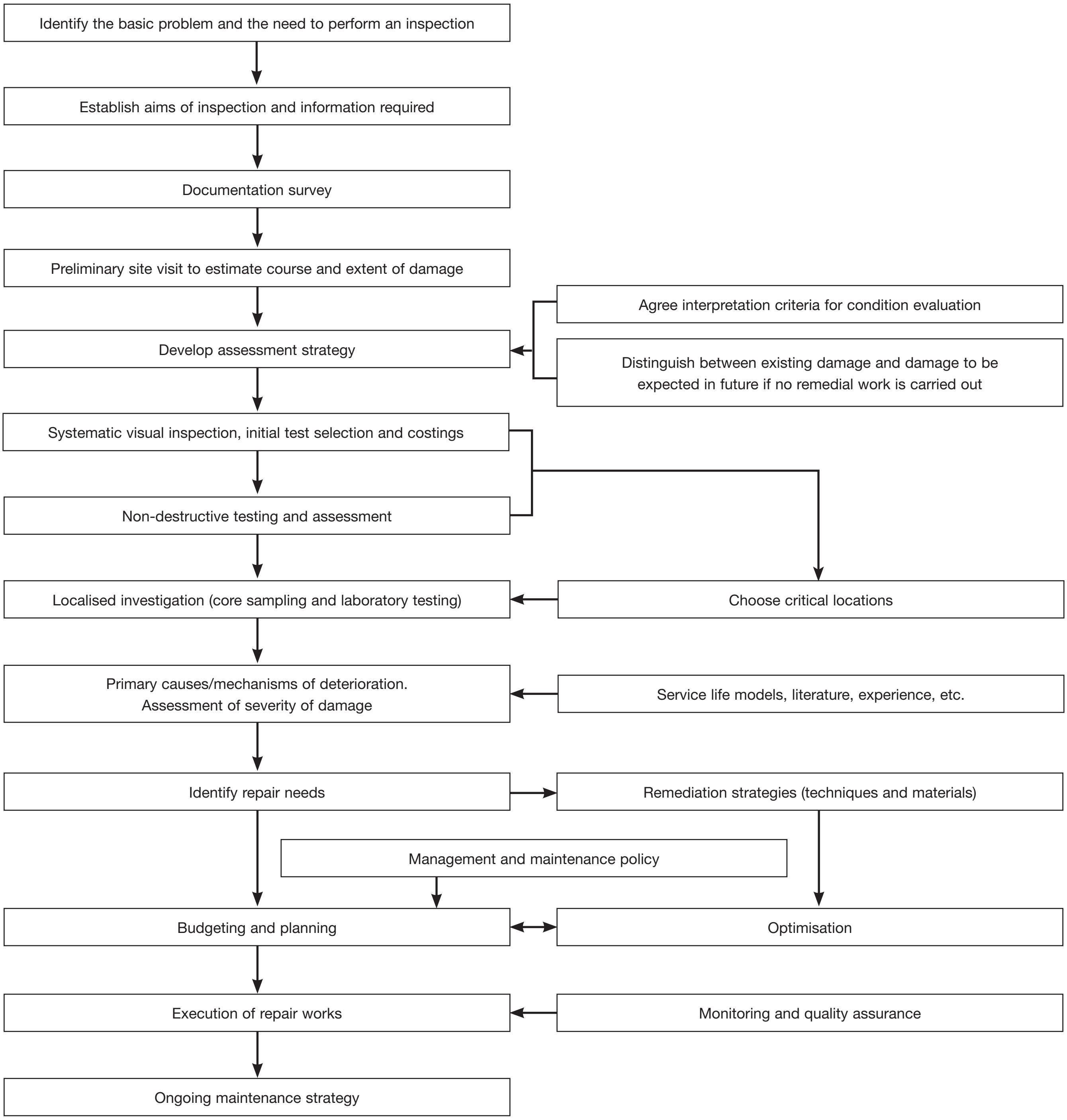


Figure 27.1: Main stages and aspects in a concrete repair project, including condition assessment and maintenance strategies

deterioration mechanisms or where design loads have been revised. Based on the assessment of material and structural conditions, as well as budget constraints, possible remedial actions and maintenance strategies can then be developed. The basic concept of assessment procedures for concrete structures in a management and maintenance system is demonstrated in Figure 27.1.

A range of different in-situ and laboratory-based assessment techniques is available. A simplified guide for the initial selection of testing procedures, based on fib bulletin 17 [27.3] is presented in Table 27.2. The following sections give an overview of the test methods most commonly applied in condition assessment of concrete structures. More detailed information on test techniques and methods, their interpretation and limitations are presented in the literature.[27.4, 27.5, 27.6, 27.7]

27.2.2 In-situ assessment

Visual assessment

The principal reasons for damage of concrete structures may be identified and defined using a systematic visual survey. Classification of visual evidence of deterioration must be done objectively, following clear guidelines that define damage in terms of appearance, location and cause. Such guidelines can, for example, be found in references 27.4, 27.6, 27.7, 27.8. Defects may be defined in terms of cracking (in most cases caused by reinforcement corrosion, restrained volume changes, structural effects, or alkali-silica reaction (ASR)), surface damage (abrasion, delamination, pop-outs, spalling), surface colourations or leakage (rust stains, lime leaching, or chemical reaction products), joint deficiencies (joint spalling, movements or distortion, seal damage), changes in member shape (curling, deflection,

Table 27.2: Overview of testing techniques for concrete structure (reproduced from fib bulletin 17 [27.3])

Information sought	Diagnostic testing or inspection technique
Strength / surface hardness.	1. *Near surface properties of concrete:* Cored samples: examination and crushing. Penetration resistance tests. Break-off tests. Internal fracture tests. Pull-off tests. Rebound hammer. 2. *Properties of body of concrete:* Cored samples: examination and crushing, etc. Ultrasonic pulse velocity.
Comparative quality / uniformity of concrete.	1. *Near surface properties of concrete:* Cored samples: examination and crushing, etc. Ultrasonic pulse velocity. Rebound hammer. 2. *Representing properties of body of concrete:* Cored samples: examination, crushing. Ultrasonic pulse velocity. Impact echo tests. Radiography.
Presence of reinforcement and depth of cover.	Covermeter. Subsurface radar. Physical exposure by excavation or cored sample.
Presence of steel tendon or duct or other metal object deeply embedded within concrete.	Subsurface radar. Pulse echo test.
Depth of carbonation.	Phenolphthalein test. Petrographic / microscopic analysis.
Presence of chlorides and chloride profiles. Presence of sulphates.	Drilled, lump or cored samples for presence of contamination by laboratory analysis; incremental sampling required for profile determination. Site chemical tests performed on drillings.
Reinforcement corrosion. Condition of embedded steel reinforcement / prestressing tendons.	Electro-potential mapping. Resistivity evaluation. Physical exposure by excavation or coring.
Corrosion rate.	Linear polarisation resistance. Galvanic current measurement.
Extent and significance of cracking and visible defects.	Visual inspection / photographic record.
Delamination.	Sounding surveys (tapping, chain drag, etc.). Examination of cored samples. Thermography. Impact echo tests. Ultrasonic pulse velocity. Subsurface radar.
Material properties: Concrete, steel, etc.	Material samples for laboratory inspection and testing for strength or other mechanical properties, physical or chemical composition, condition, durability, etc
Moisture content.	Direct (laboratory) measurement from lump sample. Direct site measurement on drilled dust sample using a chemical reagent and calorimeter. Resistance / capacitance / dew-point probes. Subsurface radar. Thermography.
Abrasion resistance.	Accelerated wearing test.
Alkali-silica reactivity.	Petrographic / microscopic analysis to identify. Laboratory tests for latent expansion properties.
Grouting of post-tensioning ducts/ corrosion or other damage to prestressing tendon within duct.	Physical exposure by excavation or coring, coupled with an air test to estimate void volume. Borescope or optical viewer inspection. Radiography.
Surface permeability / absorption.	Surface absorption test. Water and gas permeability tests. Absorption tests on intact cored samples.
Freeze-thaw.	Petrographic / microscopic analysis to identify. Laboratory tests.

settlement, deformation), and textural features (blowholes, honeycombing, sand pockets, segregation).

Visual assessment of deterioration can provide useful information when done in a rational, systematic manner. However, in the case of reinforcement corrosion, damage is often only fully manifested at the concrete surface after significant deterioration has occurred. In this case, early evidence of distress can sometimes be detected by an experienced engineer before major distress takes place, using detection methods such as rebar potential or corrosion rate measurements, as discussed in sections below.

Cover surveys

Cover surveys are routinely done to locate the position and depth of reinforcement within a concrete structure. Covermeters use an alternating magnetic field to locate steel and any other magnetic material in concrete. Cover measurements may be unreliable when rebar is at deep covers (eg greater than 80 mm), in regions with closely spaced bars, or when other magnetic material is nearby. Cover depth is of particular importance when the ingress of aggressive agents has initiated reinforcement corrosion or is suspected to do so in future. Test results should therefore be correlated with other elements of the investigation, including defect mapping, carbonation depths and chloride profiles. Though commercially available equipment is straightforward to use, considerable skill is needed to obtain reliable results.[27.3] Some modern equipment can be linked to an automatic data logger, which facilitates assessment of large areas.

Delamination surveys

Hammer survey or chain drag are simple methods of locating areas of delamination in concrete, which may for example be caused by reinforcement corrosion. Hollow sounding areas can be marked up on the concrete surface or recorded directly in a survey diagram to identify problematic areas of the structure. Delamination surveys however often under-estimate the full extent of internal damage and should not be considered as definitive. Radar and ultrasonic instruments, discussed in sections to follow, may provide a more sophisticated approach to locating areas of delamination, particularly at greater depths.

Resistivity measurements

Corrosion is an electrochemical phenomenon and therefore the electrical resistivity of the concrete affects the rate at which corrosion takes place, since an ionic current must pass from the anode to the cathode of the rebar for corrosion to occur (Figure 27.2). The higher the concrete resistivity, the lower the current flowing between anodic and cathodic areas, and therefore the lower the corrosion rates. Resistivity depends on the amount of moisture in concrete, the permeability and interconnectivity of the pore system, and on the concentration of ionic species in the pore water, such that in general poor quality, saturated concrete has low resistivity and high quality, dry concrete has high resistivity. Resistivity is also strongly affected by the quality of the concrete in terms of binder content, w/b ratio, curing, and extenders used.

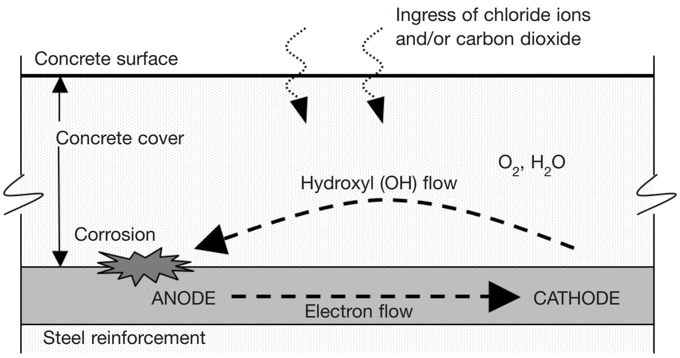


Figure 27.2: Mechanism of corrosion of steel in concrete

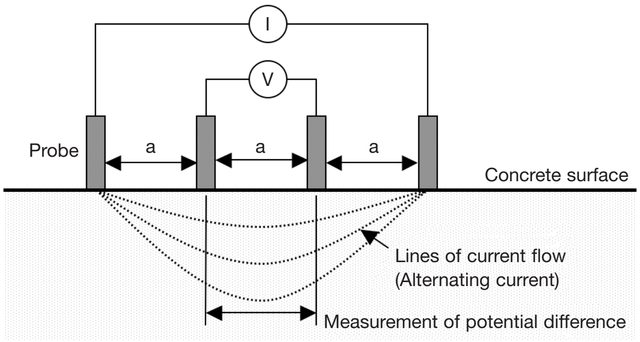


Figure 27.3: Principle of the Wenner probe for measuring concrete resistivity

Concrete resistivity is usually measured using the principles of the Wenner Probe, which consists of four equally spaced probes in contact with the concrete surface (Figure 27.3). The outer two probes send an alternating current through the concrete while the inner two probes measure the potential difference in the concrete. The measurements are useful in identifying problem areas or confirming concerns about poor quality concrete. They are further generally employed to facilitate interpretation of corrosion rate or half-cell potential measurements. Relationships between corrosion rate and resistivity have been derived and proven to be reasonably accurate.[27.9] Test interpretation and limitations of this method are presented in the literature.[27.1, 27.4, 27.6].

Rebar potential mapping

The most widely used method for assessing the likelihood of reinforcement corrosion in chloride contaminated concrete is the measurement of half-cell potentials. Chloride-induced corrosion of steel is associated with distinct anodic and cathodic areas along the rebar with consequent changes in electropotential of the steel. It is possible to measure these rebar potentials at different points and plot the results in the form of a "potential map" to indicate probable locations of corrosion activity. Measurement of rebar potentials may provide an indication of localised zones of potential corrosion activity but cannot evaluate corrosion rates or the degree of corrosion that has occurred.

Rebar potentials are normally determined in accordance with ASTM C876,[27.10] using a reference electrode (half-cell) connected to a handheld voltmeter (Figure 27.4). The half-cell is a relatively simple device made up of a metal in a saturated solution of its own ions. If this half-cell is connected to another metal there will be a potential difference between the two half-cells. When the external half-cell is connected to reinforcing steel in concrete the potential observed is the potential difference between the metal in its own solution and the steel in the concrete pore solution. The measured potential difference can be used to assess the probability of reinforcement corrosion. Note that the technique is not recommended for carbonation-induced corrosion where clearly defined anodic regions are absent. Procedures for undertaking rebar potential surveys, interpretation of test results, and limitations of this method are discussed in the literature.[27.1, 27.4, 27.6]

Corrosion rate measurements

Corrosion rate measurement is the only reliable method of measuring actual corrosion activity in reinforced concrete. A number of sophisticated corrosion monitoring systems are available, based primarily on linear polarisation resistance (LPR) principles. Measurement of polarisation resistance involves applying a small potential shift to the reinforcement and measuring the resulting current flow, which allows an estimate of the corrosion current to be made. These techniques require considerable expertise to operate reliably. Experience indicates that corrosion rates fluctuate significantly in response to environmental and material influences and single readings are generally unreliable. Test procedures and interpretation and limitations of corrosion rate measurements are discussed in the literature.[27.1, 27.4, 27.6]

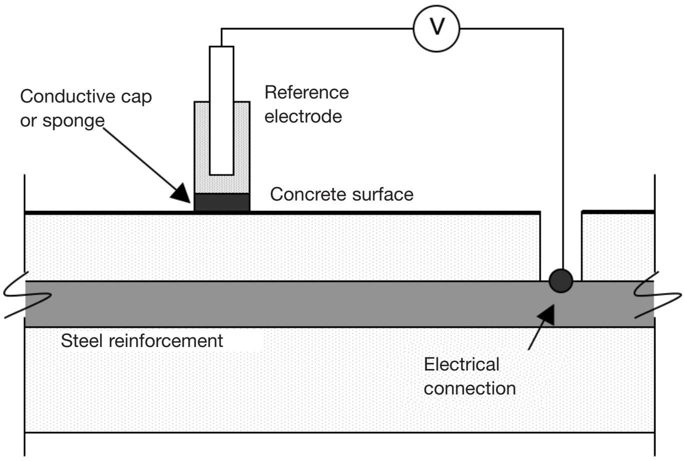


Figure 27.4: Principle of half-cell electro-potential measurements to assess chloride-induced corrosion of steel reinforcement

Acoustic methods

Information on member thickness, variations in material properties, the presence of major internal voids, delaminations or other defects can be obtained in a non-destructive manner using acoustic testing methods such as the measurement of ultrasonic pulse velocity, ultrasonic echo and impact echo. The latter two methods are used in

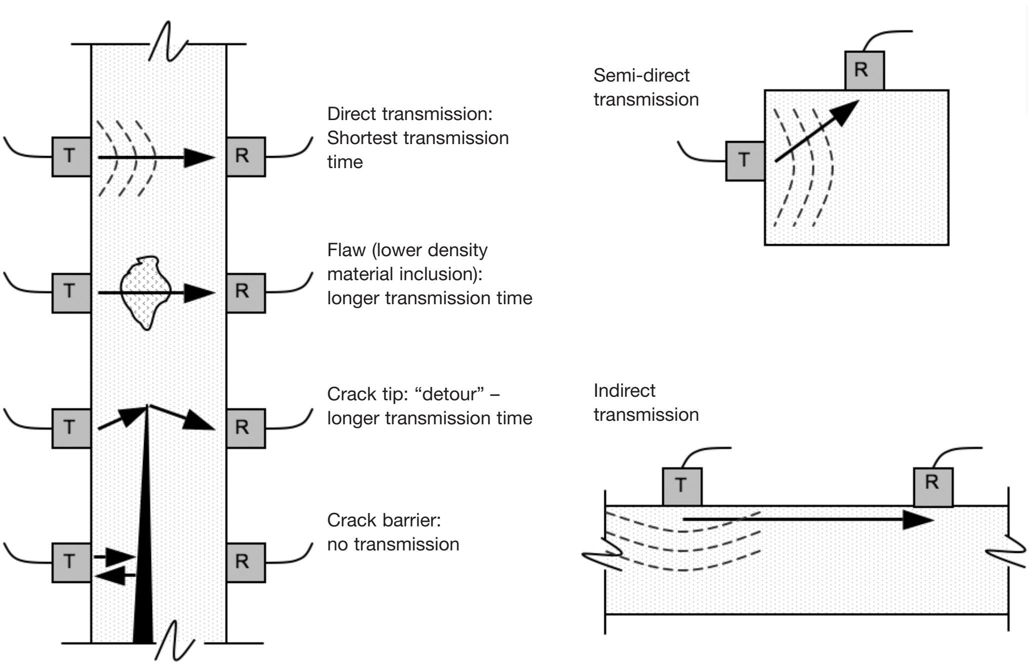


Figure 27.5: Principle of ultrasonic pulse velocity testing (T = Transmitter; R = Receiver)

echo configuration which has the advantage that access to the structure is needed from one side only.

The principle of the ultrasonic pulse velocity method comprises generating, receiving and amplifying ultrasonic pulses (frequency range 20 – 150 kHz) and measuring their velocity as they travel through the concrete (Figure 27.5). Based on the measured pulse velocity, ie the travel time of the pulse over a known path length (distance between transmitter and receiver), the relative condition of the concrete can be assessed. The measured pulse velocity allows an estimation of the dynamic elastic modulus of the concrete, which in turn can be linked to concrete strength. Since the relationship between elastic modulus and strength in concrete is non-linear and complex, measurements commonly need to be interpreted using correlation curves obtained on laboratory-tested specimens.[27.6] In the diagnostic assessment of concrete structures, ultrasonic pulse velocity testing is commonly applied in measuring variations in concrete strength and compaction, which can be used to identify defects such as flaws, cracks, compaction voids, honeycombing, etc.

The ultrasonic echo technique follows similar principles, with the difference that it measures the transit time of transmitted and reflected ultrasonic impulses generated at the concrete surface. Pulses are reflected at opposite sides of the member, or internal defects such as delaminations and cracks. The arrival of a reflected pulse is recorded by a receiver placed next to the transmitter, and the round-trip travel time recorded. Common applications of ultrasonic echo are measurement of member thickness, location of prestressing ducts, and detection of compaction faults (eg honeycombing).[27.11]

The impact echo method uses transient stress waves generated on the surface of concrete structures by an elastic (mechanical) impact of a 3 -15 mm steel sphere.[27.12] As the waves propagate through the structure, they are reflected by internal interfaces and external boundaries of the structure (Figure 27.6). In order to detect such interfaces, the emitted waves are recorded by a displacement or acceleration transducer which is placed near the impact point on the surface of the structure. The impact echo method has its strength in the ability to detect voids in structures and to measure the thickness of concrete members with considerable accuracy. Common practical limitations result from geometric effects, large reinforcement densities, or rough concrete surfaces.[27.13]

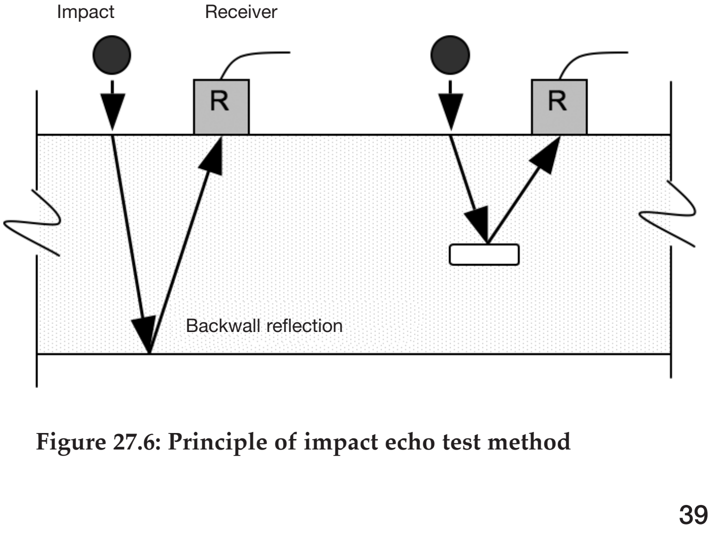


Figure 27.6: Principle of impact echo test method

Radar

Radar methods can be used for several applications, such as the location of objects in concrete members, ie reinforcement and prestressing ducts, measurement of moisture profiles, and evaluation of the thickness of the wearing course on bridges. As for acoustic methods, radar measurement techniques are based on the transit time measurement of transmitted and reflected impulses, in this case however generating electromagnetic impulses. An interface with a great change in dielectric properties (eg concrete/air in the case of a void or a back wall) or high conductivity (eg embedded reinforcement steel) produces a clear reflection. The device consists of one transmitting and one receiving antenna in a single casing and is moved over the surface with a clock wheel for positioning, which makes radar a fast testing method.[27.13] The equipment is relatively expensive and requires experienced personnel to operate it and interpret the results.[27.3]

28.2.3 Laboratory tests

In-situ concrete sampling

On-site sampling is undertaken by removing either core or powder samples from the concrete structure. Since this technique is partly destructive, ie superficial damage is inflicted on the concrete surface, it is usual to perform minor repairs to reinstate the integrity of the structure once the sampling programme is completed. Powder samples are typically used to measure chloride profiles or carbonation depths. Core samples can be used for a number of tests, the most common being the visual assessment of concrete quality (constituents, compaction, voids, etc), the measurement of physical and mechanical characteristics such as transport properties, density, strength and elastic modulus, chemical analysis to obtain carbonation depths, chloride or sulphate profiles, and petrographic investigation to obtain information on chemical constituents and damage on a microscopic level.

Chloride content

The presence of sufficient chlorides at the surface of reinforcement is able to depassivate steel and allow corrosion to occur. Chlorides exist in concrete as both bound and free ions but only free chlorides directly affect corrosion. Chlorides are most commonly determined as acid soluble or total chlorides in accordance with BS 1881.[27.14]. Chloride samples are extracted as either core or drilled powder samples. In the latter case, contamination between successive drilled samples must be avoided. Depth increments are chosen depending on the cover to steel and the likely level of chloride contamination (increments are typically between 5 and 25 mm). Chloride contents are generally expressed as a percentage by mass of cement and used to estimate the risk of reinforcement corrosion. Chloride profiles may be drawn such that chloride concentrations can be interpolated or extrapolated for any depth. Future chloride levels can be predicted using chloride diffusion theories in order to estimate remaining service life of the structure. The corrosion threshold, ie the chloride content at which corrosion is initiated, depends on several factors including binder type, concrete quality, cover depth, and saturation level of the concrete. The assessment of corrosion probability based on chloride measurements is discussed in the literature.[27.1, 27.4]

On-site estimation of chloride content of drilled powder samples can be done using commercially available kits, and is generally straightforward and quick to perform. If chloride contents are high, the tests are generally sufficiently accurate to indicate approximate chloride concentrations. However, such fast-track indications need to be confirmed by a proportion of laboratory-based determinations of chloride contents,[27.3] especially when reliable information on chloride content is needed for selection of appropriate repair methods.

Carbonation depth

Carbonation depth is measured conventionally by spraying a freshly exposed concrete sample removed from the structure, eg a core sample, with a phenolphthalein indicator solution. When removal of samples from the structure is not possible, the solution can be progressively sprayed into an incrementally drilled hole or progressively onto the powder drilled from that hole. Phenolphthalein remains clear where concrete is carbonated but turns pink/purple where concrete is still strongly alkaline (pH > 9.0). Carbonation moves through concrete as a distinct front and reduces the natural alkalinity of concrete from a pH in excess of 12,5 to approximately 8,3, which is sufficiently low to depassivate steel and initiate corrosion. In the vicinity of damage or cracks, the carbonation front progresses to greater depths, which should be considered in the interpretation of test results.

Petrographic examination

Petrographic analysis is the detailed microscopic examination of a concrete sample to determine its composition and identify deterioration mechanisms using established petrographic techniques such as polarised light microscopy. It includes identification of mineral in aggregates, aggregate-paste interface characteristics, and the assessment of the integrity of the cement matrix. Petrographic examination is a highly specialised practice requiring skilled personnel and is particularly useful in identifying the presence of sulphate attack, alkali-silica reaction, aggregate alteration, and carbonation. Standard methods are provided by ASTM.[27.15]

27.3 Repair strategies and techniques

27.3.1 General considerations

Numerous repair options are available for damaged concrete structures. New technologies and materials continue to make an impact in the field of concrete repairs. The suitability and cost effectiveness of repairs depend on the level and type of deterioration, specific conditions of the structure, and the desired remaining service life. An optimum repair strategy can only be identified based on methodical condition assessment and forensic analysis, an appropriate maintenance plan, and sound engineering judgement of the various repair options available. The final design of remedial measures needs to consider durability and constructability of the system, availability of materials and skills, and compatibility of the repair system with the existing structure and the environment. Further aspects that may need to be considered include aesthetics, safety and environmental issues, time constraints, and access restrictions.

A comprehensive overview of principles for concrete protection and repair methods can be found in the European standard EN 1504,[27.2, 27.16] which is expected to replace national European standards by 2009. EN 1504 specifies the following options to be taken into account in deciding appropriate actions:

- Do nothing
- Re-analysis of structural capacity, possibly resulting in downgrading of the function of the structure
- Prevention or reduction of further deterioration, without improvement of the structure
- Improvement, strengthening or refurbishment of parts or all of the structure
- Reconstruction of parts or all of the structure
- Demolition of parts or all of the structure

Based on the RILEM Technical Recommendation 124-SRC,[27.17] EN 1504 defines different principles for protection and repair, separately in terms of damage to the concrete and damage induced by reinforcement corrosion, as summarised in Tables 27.3 and 27.4.

27.3.2 Repair principles for structures damaged by reinforcement corrosion

General

In the highly alkaline environment of newly constructed concrete, steel reinforcement is protected by a passivating film of ferric oxides bonded to the surface of the reinforcement. When this passivating film is disrupted by a reduction in the alkalinity of the concrete (principally by carbonation) or by the presence of aggressive ions such as chlorides and sulphates, reinforcement corrosion may occur. During the corrosion process, anodic and cathodic regions form along the reinforcement while the concrete acts as an electrolyte to allow the flow of electrical current between cathode and anode. This electrical flow results in an accumulation of expansive iron corrosion products at the anode, which in its advanced state will be evident in rust stains, cracking and spalling of concrete. In this reaction, the rate of corrosion is largely determined by oxygen availability and moisture content together with resistivity of the concrete (compare Figure 27.2).

For the choice of the appropriate time to undertake repairs it is important to note that reinforcement corrosion can conceptually be divided into 3 different stages (Figure 27.7). Corrosion is activated at the end of the initiation period, usually either by carbonation or chloride attack, but negligible concrete deterioration occurs. Active

Table 27.3: Deterioration processes and remedial actions according to EN 1504 [27.16]

	Observation	Cause of defects	Principle of remedial actions (see Table 27.4)
Defects in concrete	Cracks. Spalling. Delamination. Disintegration of the matrix.	1. *Mechanical:* Impact. Overload. Movement (settlement). Explosion. Vibration. Seismic.	Concrete restoration (CR). Structural strengthening (SS).
		2. *Chemical:* Alkali-aggregate reaction. Aggressive agents (sulphates, soft water, acids, salts). Biological activities.	Protection against ingress (PI). Moisture control (MC). Increasing resistance to chemicals (RC).
		3. *Physical:* Freeze / thaw. Thermal - fire. Salt crystallisation. Shrinkage. Erosion. Wear.	Protection against ingress (PI). Moisture control (MC). Increasing physical resistance (PR). Structural strengthening (SS).
Reinforcement corrosion	Uniform corrosion. Pitting corrosion. Stress corrosion. Cracking.	Carbonation of concrete.	Preserving or restoring passivity (RP). Control of anodic areas (CA).
		Corrosive contaminants: Sodium chloride, calcium chloride, others.	Cathodic control (CC). Cathodic protection (CP). Control of anodic areas (CA). Preserving or restoring passivity (RP).
		Stray currents.	Increasing resistivity (IR).

Table 27.4: Principles and remedial actions according to EN 1504 [27.16]

	Principle (see Table 27.3)	Methods based on the principle (examples)
Surface protection	*Protection against ingress (PI):* Reducing or preventing the ingress of adverse agents, e.g. water, other liquids, vapour, gas, chemicals, and biological agents.	Surface impregnation. Surface coating. Bandaging cracks. Filling cracks. Converting cracks to joints. Erecting external panels. Applying membranes.
	Physical resistance (PR): Increasing resistance to physical or mechanical attack.	Overlays. Coatings. Impregnation.
	Resistance to chemicals (RC): Increasing resistance of the concrete surface to deteriorations by chemical attack.	
	Moisture control (MC): Adjusting and maintaining the moisture content in the concrete within a specified range of values.	Hydrophobic impregnation. Surface coating. Sheltering or overcladding.
Repair	*Concrete restoration (CR):* Restoring to the originally designed shape and function.	Hand-applied mortar. Recasting with concrete. Spraying concrete or mortar. Replacing elements.
	Cathodic control (CC): Creating conditions in which potentially cathodic areas of reinforcement are unable to drive an anodic reaction.	Reducing oxygen supply at the cathode by saturation or surface coating.
	Preserving or restoring passivity (RP): Creating chemical conditions in which the surface of the reinforcement is maintained in, or is returned to, a passive condition.	Increasing cover with additional concrete or mortar. Replacing contaminated or carbonated concrete. Electrochemical realkalisation of carbonated concrete. Realkalisation of carbonated concrete by diffusion. Electrochemical chloride extraction.
	Cathodic protection (CP)	Applying electrical potential
	Control of anodic areas (CA): Creating conditions in which potentially anodic areas of reinforcement are unable to participate in corrosion reaction.	Painting reinforcement with coatings containing active pigments (eg zinc). Painting reinforcement with barrier coatings. Applying penetrating corrosion inhibitors to the concrete surface.
Structural strengthening	*Structural strengthening (SS):* Increasing or restoring the structural load bearing capacity of an element of the concrete structure	Adding or replacing embedded or external reinforcing steel bars. Installing bonded rebars in preformed or drilled holes in the concrete. Plate bonding. Adding mortar or concrete. Injecting cracks or voids. Filling cracks or voids. Prestressing - post- tensioning.

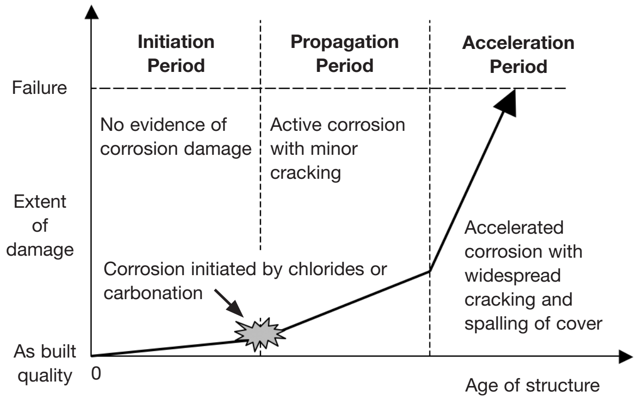


Figure 27.7: Three-phase corrosion damage model

corrosion commences in the propagation period, which often results in staining and cracking of the cover concrete due to the formation of expansive corrosion products at the steel surface. During the acceleration period of damage, corrosion increases due to easy access of oxygen and water through cracks in the cover concrete, resulting in delamination and spalling of concrete.

Unfortunately most reinforced concrete structures that exhibit cracking and spalling have gone beyond the point where simple, cost-effective repair measures can be taken to restore durability. Condition surveys are therefore an important strategy to identify and quantify the state of corrosion of a structure timeously. In order to choose the right time to begin maintenance and repair works, it is important to understand that once the propagation period has been reached, damage occurs at an increasing rate. In principle, remedial measures for corrosion damaged concrete structures will be most effective and least costly, the earlier they commence.

One of the most common reinstatement methods for cracking, delamination and spalling damage is the removal of deteriorated concrete and application of patch repair materials. However, before patch repairs are considered it is important that the distinction between chloride- and carbonation-induced corrosion is appreciated, these two mechanisms being by far the most common causes of reinforcement corrosion. As a general rule chloride-induced corrosion is far more destructive and difficult to treat than carbonation-induced corrosion. This often dictates a completely different approach to repairing damage due to the two types of corrosion.

Carbonation-induced corrosion

Carbonated concrete tends to have fairly high resistivity that discourages macro-cell formation and allows moderate corrosion rates. As a result, carbonation-induced corrosion causes multiple pitting along the reinforcement. Environmental conditions that are most favourable for carbonation (ie 50 to 65% R.H.) are usually too dry to allow rapid steel corrosion that normally requires humidity levels above 80% R.H. Structures exposed to fluctuations in moisture conditions of the cover concrete, such as may occur during rainy spells, are however vulnerable to carbonation-induced corrosion.

For reinforced concrete structures affected by carbonation-induced corrosion, the bonded overlay repair (patch repair) technique is generally successful provided all of the corroded reinforcement is treated and the overlay is designed and applied properly. Localised patch repair of those areas where reinforcement corrosion has already resulted in detectable damage is commonly the most cost-effective method for this type of deterioration. Adjacent areas, which may also have carbonated but which do not yet show signs of corrosion damage, may need to be protected by means of surface coatings or corrosion inhibitors to slow down corrosion rates sufficiently, preventing defects from occurring in these regions once the structure has been repaired.

Proper workmanship and repair performance monitoring are essential in ensuring that the repair material bonds properly to the vertical boundaries around the patch. Debonding at vertical boundaries, for example caused by repair material shrinkage, may open pathways for aggressive agents to the reinforcement, which would promote the development of new corrosion sites and render the repair unsuccessful. Cracking of the repair material under service conditions needs to be prevented for the same reasons.

Chloride-induced corrosion

Chloride-induced corrosion is characterised by pitting corrosion with distinct anodic and cathodic sites. The presence of high salt concentrations in the cover concrete means that macro-cell corrosion is possible with relatively large cathodic areas driving localised intense anodes. High corrosion rates can be sustained under such conditions resulting in severe pitting of the reinforcement and damage of the surrounding concrete. Much of the reinforcement may be exposed to corrosive conditions without showing any signs of corrosion, which is particularly noticeable when corroded structures are demolished.

Localised patch repairs of areas of corrosion damage are popular due to their low cost and temporary aesthetic relief. This form of repair has however very limited success against chloride-induced corrosion as the surrounding concrete may be chloride-contaminated and the reinforcement is therefore still susceptible to corrosion. The patched area of new repair material often causes the formation of incipient anodes adjacent to the repairs as shown in Figure 27.8. These new corrosion sites not only affect the structure but often also undermine the repair leading to accelerated patch failures in as little as two years. Consequently, it is necessary to remove all chloride-contaminated concrete from the vicinity of the reinforcement.

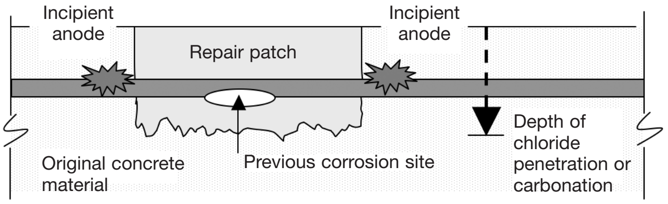


Figure 27.8: Formation of incipient anodes after patch repairs

Complete removal of chloride-contaminated concrete, where it is possible, should successfully halt corrosion by restoring passivating conditions to the reinforcement. This form of repair is most successful when treating areas of localised low cover, before significant chloride penetration has occurred. If repairs are only considered once corrosion damage is fairly widespread, it will be expensive to mechanically remove chloride-contaminated concrete from depths well beyond the reinforcement. In such cases, electrochemical techniques, such as cathodic protection, or application of penetrating corrosion inhibitors may need to be considered, as discussed in later sections.

27.3.3 Patch repairs and bonded concrete overlays

General considerations

Bonded concrete overlays are widely used for repair, rehabilitation and strengthening of concrete members. The term "patch repair" is typically used for bonded overlays of relatively small surface area. Despite their increasing use, failure of overlays is often observed in practice, manifesting itself mainly in overlay cracking and debonding. Possible reasons for such failures are numerous and include substrate surface preparation, choice and application of overlay materials, curing procedures, time-dependent material properties and environmental influences. In general it is accepted that poor workmanship and differential shrinkage between substrate and overlay are the main detrimental influences for the performance of bonded concrete overlays both in terms of cracking resistance and bond strength.

Standards and specifications for the design of bonded concrete overlays are generally deficient in scope and detail and it is left to the engineer to specify appropriate materials and application procedures. For structural applications, codes commonly define limitations for design shear resistance at the interface between substrate and overlay. If composite members are designed with adequate interface shear strength, they can be modelled following the same procedures as those developed for monolithic members.

The most common test method to measure bond strength in situ is the pull-off test. Standards and specifications for concrete repair[27.18-27.22] therefore specify overlay bond strength as tensile bond and give limiting values as acceptance criteria, commonly in the range of 1,0 – 1,5 MPa.

The bond between substrate and overlay is dependent on a large number of factors.[27.23] Figure 27.9 attempts to estimate the importance of the factors listed. The estimation is based on test data and case studies.

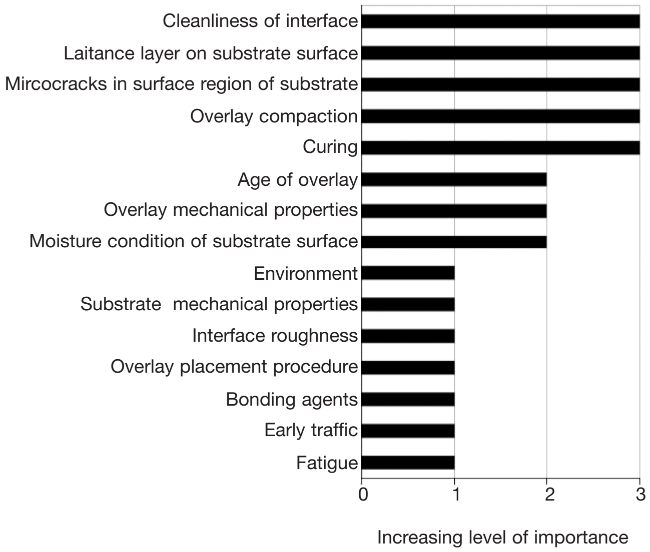


Figure 27.9: Level of importance of factors influencing bond strength between concrete substrate and concrete overlays[27.23]

An overview of factors to be considered in the design and application of bonded overlays and patch repairs is presented in Figure 27.10. The following sections relate to basic rules applying to bonded overlays and patch repairs. Additional aspects relating to structural, load-bearing, overlays, such as replacement of disintegrated reinforcement bars, provision of additional internal or external reinforcement, or techniques for the transfer of load-induced stresses into the overlay, are not considered. Information on these aspects may be found in the literature in references 27.4 and 27.8.

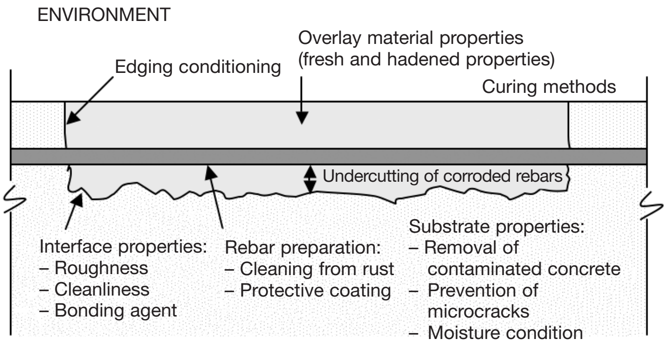


Figure 27.10: Overview of factors to be considered in the design and application of bonded overlays and patch repairs

Substrate preparation

Surface preparation and cleaning of the concrete substrate are generally considered to be the most crucial steps in a concrete patch repair. In order to achieve a good and durable repair, poor quality and deteriorated substrate concrete must be removed to the greatest possible extent. It is important that corroding reinforcement is undercut (typically by approximately 20 mm) and properly cleaned of all corrosion products. The remaining substrate should constitute a sound and strong material, which is one of the prerequisites for producing a repaired concrete structure with monolithic behaviour. A poorly prepared substrate surface will always constitute the weak link in the system, no matter how good the repair material is or how thorough its application. It is important to stress the difference between substrate surface removal and substrate surface cleaning.[27.24] Surface removal includes taking away of parts of the substrate concrete and previously applied coatings, whereas cleaning commonly refers to the removal of loose particles and contaminants. Surface preparation is important for the removal of deteriorated concrete and the creation of a good, sound interface with adequate texture. Surface cleanliness is vital, as any loose debris, dirt, grease or other surface contaminants can act as bond breakers. The above highlights the importance of good workmanship and site supervision.

The surface of the substrate concrete needs to be free from microcracking as it will otherwise constitute a zone of weakness prone to bond failure. The amount of microcracking is governed by the selected method of concrete removal. In principle, mechanical methods such as jackhammers are likely to introduce microcracking, whereas waterjetting or sandblasting techniques have been shown to produce sound substrate surfaces with sufficient roughness for bond strength provision.[27.23, 27.25-27.29] Field tests have shown that good bond strength can be achieved if mechanical removal, which in many cases is necessary to remove deteriorated concrete to sufficient depth, is followed by high-pressure waterjet cleaning.[27.30] Boundaries of repair patches should be cut straight and normal to free surfaces to avoid feather edges that may result in localised patch repair failure and unsightly appearance.

The substrate moisture condition may have a significant influence on bond strength. A dry, "thirsty" concrete surface tends to suck water from the fresh overlay material, which may result in a weak interfacial layer and low bond strength. A surface which is too wet tends to dilute the repair material at the interface and increases the water/cement ratio, hence leading to low material strength, increased shrinkage and low bond strength. Water in open pores further prevents the interlocking effect. The substrate concrete should, therefore, be saturated but surface dry.

The use of bonding agents has been intensively investigated by many researchers, resulting in a broad range of opinions. However, it is generally accepted that bonding agents cannot compensate for bad substrate surface preparation and may act as a bond breaker when used inappropriately.[27.31, 27.32] When bonding agents are used, care must be taken that the repair mortar is applied before the bonding agent has hardened, as bonding is otherwise prevented. By contrast, bonding agents may improve bond strength for certain materials, especially stiff repair mortars that cannot properly fill open pores and cavities.

Different types of bonding agents are available, mostly in the form of cement-based slurries, epoxies, and polymer-cement slurries. Cement-based slurries, which can be mixed on site, were found to result in high bond strengths if applied correctly.[27.31, 27.33, 27.34]

Proprietary patch repair materials are often specified to be used in conjunction with proprietary bonding agents from the same supplier. The decision on whether or not to use a bonding agent should be made by an experienced engineer based on consideration of interface texture, type of repair material, application methods and prevailing site conditions. One advantage of proprietary bonding agents is that they are commonly formulated to also function as a protective coating to reinforcing steel, so that rebar protection and bonding agent application can be carried out in one go. Montani[27.35] states that the reasons bonding agents have gained a bad reputation lie in poor workmanship during their application. Once again, this points out the importance of good workmanship and site supervision.

Plastic repair material properties and material application

The fresh repair material properties are important both for early-age bond strength development and bond durability. Workability and compaction of the freshly placed overlay influence the ability to fill open cavities and voids on the substrate concrete surface and determine the effective contact area between the two composites. A relatively fluid overlay further enhances capillary suction in the substrate and therefore improves physical anchorage in substrate surface pores and cavities. Horizontal repairs, for example on pavements or bridge decks, and large application areas on vertical surfaces, in the latter case using formwork, may be carried out using conventional concrete with high workability.

By contrast to the above, small repair patches are commonly made with premixed, relatively stiff mortars, which are applied with a trowel or, without using application tools, by hand. This leads to lower capillary suction of the substrate, compared to overlays of higher fluidity, thus resulting in lower bond strength. For these kinds of mortars, bonding agents might be helpful to improve adhesion, as discussed in the previous section. On large vertical, inclined or overhead repair areas, sprayed concrete (shotcrete or gunite) application may be a practical and economical option.

Sprayed concrete is applied to the concrete substrate utilising a high amount of energy. The mixture of fine aggregate and paste is pressed into the open pores and surface texture of the substrate, which, if applied correctly, results in very good bond between substrate and overlay.[27.36]

Cracks resulting from shrinkage may initiate debonding due to the formation of boundary conditions at free edges. Curing prevents rapid moisture loss to the environment and thus controls the development of overlay and interface stresses by reducing shrinkage at early ages when bond strength may be low. Common curing practices for concrete also apply to patch repairs and bonded overlays. Curing should be monitored to ensure that the repair material is constantly kept moist for at least 3 days, or, ideally, for a period of at least 7 days. Special circumstances, eg harsh environmental conditions or site access restrictions, may result in the need to apply a proprietary curing compound instead of wet hessian and plastic sheets.

Hardened repair material properties

On well-prepared and sound concrete substrates, and under ideal curing conditions, bond strength development between substrate and overlay is largely controlled by mechanical strength development of the overlay.[27.37–27.39] For the long-term serviceability of overlays, which is mainly governed by crack-resistance and bond durability, overlay material properties influencing stress development are of importance, such as elastic modulus, shrinkage, and thermal coefficient.[27.40–27.42] As a principal rule, overlays should have as little shrinkage as possible and a thermal coefficient similar to that of the substrate. Repair materials are often not required to have high values of compressive strength or elastic modulus, especially in non-structural applications. In fact, so-called "high-performance" overlays of high strength and high stiffness are generally outperformed by overlays of apparent "lower quality", which, due to a lower stiffness, have much better crack resistance.[27.43] The addition of fibre reinforcement is generally considered to further enhance crack resistance and bond durability of repair patches and overlays. [27.44, 27.45]

Typical material requirements for bonded overlays and patch repairs comprise load-bearing properties (compressive strength, elastic modulus, abrasion resistance, etc), aesthetics (colour to match existing concrete?), durability (permeability, absorption, and diffusion characteristics), and dimensional stability (shrinkage, thermal deformations). In principle, the choice of a suitable repair material should be based on considerations of compatibility between substrate and overlay. Next to similar thermal deformation characteristics, similar elastic properties and low overlay creep may need to be aimed at if substrate and overlay are subjected to load-induced deformations.

Of major concern are shrinkage characteristics of common proprietary repair materials. Emmons and Vaysburd[27.46] found that a minority of repair materials tested could be labelled as low shrinkage, despite the fact that manufacturers classify them as expansive, non-shrinking or shrinkage-compensating. Similar conclusions were drawn from test results obtained with locally available commercial repair materials.[27.47] Thorough curing and application of surface coatings may prevent extensive development of shrinkage cracks on patches. In all cases, extra care should be taken under hot or windy weather conditions. Conventional concrete mixes can be better controlled in terms of shrinkage characteristics, compared to commercial repair mortars and may in many cases offer better performance, especially on large repair areas. By contrast, application of repair mortars (instead of conventional concrete) is recommended for areas of up to approximately 1 m^2.[27.48]

Other requirements may result for example from thickness limitations or prescribed material application methods. Overlays made from conventional concrete can be custom-made to conform to such requirements. A large range of proprietary repair mortars is available, representing numerous material properties and application fields. Prior to application of such materials, data sheets and material suppliers should be consulted to determine which material best fits the requirements of the specific project. In critical cases, tests should be conducted in situ or in the laboratory to confirm desired properties and performance of chosen repair mortars.

Common types of repair mortars

Common commercial repair mortars contain sand, cement, polymers and other minor constituents and, in some cases, fibres. Locally available pre-packed mortars are usually supplied in pre-mixed powder form, packed in 25 kg bags. Due to the application of factory-based quality assurance schemes, proprietary repair materials are generally considered to give better performance and consistency than site-batched repair mixes.[27.3] However, shrinkage characteristics of such commercial materials must be considered, as discussed in the previous section. Portland cements are used for most general repair works and deep sections. Durability of such materials can be provided through the use of cement extenders or admixtures that reduce permeability. Polymer materials such as epoxies and acrylics are used for special applications requiring chemical resistance or very thin applications [27.8] When using polymer-modified repair mortars, compatibility requirements with the concrete substrate may need to be considered as they have higher thermal coefficients, compared to conventional concrete.

Another issue that may need consideration is the permeability of polymer-modified repair mortars. In certain circumstances the substrate concrete, ie the repaired structure, may need to "breathe" through the repair layer, eg to allow removal of moisture vapour that may otherwise accumulate at the interface between substrate and repair mortar.

For certain applications or requirements, epoxy resin repair mortars may provide useful alternatives to cement-based mortars. Epoxies, commonly supplied as two-component materials, can be designed with relatively high strength and good resistance to abrasion, chemical attack and environmental influences. One of the main advantages of epoxy repair materials is that they can be applied in small thicknesses of a few millimetres. Bond strength to the substrate is commonly good, provided the substrate surface is prepared according to the guidelines of the material supplier. Problems may however arise from dimensional incompatibility between the epoxy mortar and concrete substrate. The coefficient of thermal expansion of epoxy is 2-5 times as high as that of concrete.[27.8] The higher elastic modulus of the concrete substrate tends to restrain the deformations of the epoxy mortar, thereby causing severe stresses at the interface when subjected to temperature changes.[27.4] Elastic modulus and thermal properties of epoxy mortars can however be partially adjusted by the addition of fillers, such as aggregates.

27.3.4 Surface protection systems and coatings

A variety of surface coatings and penetrant systems are available that are claimed to be beneficial in corrosion repair and protection of concrete structures. An overview on common types of coatings and their applications is presented in Table 27.5. Barrier systems attempt to seal the surface thereby stifling corrosion by restricting oxygen flow to the cathode and preventing further ingress of aggressive agents. In large concrete structures, however, successful corrosion control is theoretically unlikely due to the presence of oxygen already in the system. In practice, when used on their own, barrier systems may be ineffective due to the presence of defects in the coating during application and further damage during service. Localised damage in the coating may promote the formation of differential aeration cells further exacerbating the potential for corrosion.

A hydrophobic coating (sometimes referred to as a penetrant pore-liner) may be used to reduce the moisture content of

Table 27.5: Summary of common applications of main types of surface coatings for concrete repair and protection, based on the *ACI Concrete Repair Manual* [27.4]

Reasons for application of coating		Type of coating						Comments
		Inorganic		Hybrid	Organic			
		Cementitious	Silicate-based	Penetrant poreliner: Silicone; Silane; Siloxane	Pore filling sealer	Resins, eg epoxy; Thermosetting	Thermoplastic; Synthetic rubber	
Enhance or maintain appearance	Mould, dirt staining	✓	✓	✓	✓	✓	✓	
	Uniformity after repair	✓	✓	✗	✗	✓	✓	
	Anti-grafitti	✗	✗	(✓)	(✓)	✓	(✓)	Some products only
	Colour, texture	(✓)	✓	✗	✗	✓	✓	
Prevent rebar corrosion	Carbonation	✓	✓	✗	(✓)	✓	✓	
	Chloride ingress	✓	(✓)	✓	✓	✓	✓	
Prevent deterioration (chemical attack)	Acids	✗	✗	✗	(✓)	✓	(✓)	Depends on specific chemicals
	Sulphates	✓	✗	(✓)	✓	✓	✓	
	Chemicals	(✓)	✗	(✓)	(✓)	✓	(✓)	
Prevent deterioration (physical attack)	Freeze-thaw	✓	✗	✓	✓	✓	✓	Depends on circumstances
	Salt crystallisation	✓	✗	(✓)	(✓)	✓	✓	
	Erosion, abrasion	✓	✗	✗	✓	✓	✓	
	Impact	✓	✗	✗	(✓)	✓	✓	
Limit or control ingress or contact	Waterproofing	✓	✓	✓	✓	✓	✓	
	Vapour and gas barriers	(✓)	(✓)	✗	(✓)	✓	✓	
	Liquid barriers, breathable	✓	(✓)	✓	(✓)	✗	✓	Only certain organic products
	Potable water	✓	✗	(✓)	✓	✓	✓	Approved products only
Enhance safety	Anti-slip/skid	(✓)	✗	✗	✗	✓	(✓)	Use with aggregate scatter

✓ = commonly used
(✓) = occasional use depending on circumstances
✗ = use is unlikely

concrete and thereby electrolytically stifle the corrosion reaction. The drying action works on the principle that surface capillaries become lined with a hydrophobic coating that repels water molecules during wetting but allows water vapour movement out of the concrete, to facilitate drying. Hydrophobic coatings using silanes and siloxanes are generally most effective on uncontaminated concrete, free from cracks and surface defects. The feasibility of such an approach is questionable for marine structures where high ambient humidity and presence of high salt concentrations all interfere with drying. The long-term effectiveness of hydrophobic systems applied to new construction is not known but local South African studies suggest reasonable performance over 10-15 years service. For repair applications, silane-based impregnation was found to have limited effect on preventing chloride ingress over time when used on its own.[27.49] However, when used in conjunction with penetrating corrosion inhibitors, hydrophobic surface treatments were found to substantially increase the performance of concrete repairs to marine structures in the Western Cape.[27.50]

In some applications, coatings need to have the capability to bridge cracks, which requires high elasticity of the coating material. In general, so-called crack-bridging coatings are able to bridge cracks of up to about 0,5 mm in width.[27.3]

For successful application of coating systems, attention must be directed to providing adequate surface preparation, the use of a proper application method, and, as with all repair works, the specification and control of the standard of workmanship. Information on available coating systems and materials, as well as appropriate substrate surface preparation and material application techniques, is commonly obtained from the material supplier.

27.3.5 Penetrating corrosion inhibitors

Corrosion inhibitors (CI) are defined as chemical substances that reduce the corrosion of metals without a reduction in the concentration of corrosive agents. The principle of CI is to develop a thin chemical layer on the steel surface, which reduces the rate of the anodic and/or cathodic reactions thereby suppressing the overall corrosion rate. Corrosion-inhibiting chemicals are either mixed into the fresh concrete, or, as is commonly the case in repair works, applied to the hardened concrete surface, after which they penetrate (migrate) through the concrete to react on the reinforcing steel surface. Penetrating CI are generally organic-based materials, such as amino-alcohols, that move through unsaturated concrete by vapour diffusion. Organic CI are believed to suppress corrosion primarily by being adsorbed onto the steel surface thereby displacing corrosive ions such as chlorides. The adsorbed organic layer inhibits corrosion by interfering with anodic dissolution of iron while simultaneously disrupting the reduction of oxygen at the cathode.

When assessing the suitability of repairs using penetrating CI, two important issues must first be considered. Firstly, the likely penetration of the material into the concrete needs to be determined and secondly, the severity of the corrosive environment at the reinforcement must be quantified. Penetrating CI are designed to move fairly rapidly through partially saturated concretes that allow vapour diffusion. Penetration has however been found to be poor in near-saturated concretes typically found in partially submerged marine structures.[27.51] It is critical therefore that satisfactory penetration of CI is checked on site before undertaking full-scale repairs.

It is generally recognised that admixed CI, if used correctly, are successful in suppressing the rate of reinforcement corrosion in concrete and thus "buying" time and extending the life of structures. The long-term effectiveness of surface-applied penetrating CI for concrete repair however still needs to be confirmed through further research.[27.52] Locally, organic CI, applied either before or after carbonation were found to be effective in reducing corrosion rate in carbonation-induced corrosion.[27.51, 27.53]

The performance of penetrating CI in controlling chloride-induced corrosion, however, is largely dependent on chloride levels at the reinforcement. For common CI materials used in South Africa a maximum chloride content of 1% by mass of cement at the level of the reinforcement is specified for successful application.[27.54, 27.55] Work reported by Mackechnie et al[27.51] indicates that effective inhibition is not possible at chloride levels above 2% (by mass of cement) at the reinforcement. For chloride levels between 1 and 2%, the same reference suggests additional protection in the form of a high quality coating system should be applied when CI are used as a repair option. Similar results were presented by Stanish and Alexander,[27.56, 27.57] who concluded that the application of CI after initiation of corrosion can cause corrosion to stop, provided the chloride concentration at the reinforcement is below the enhanced threshold level that the inhibitor provides, as would be determined if the inhibitor was applied prior to corrosion.

Considering plain CEM I concrete, for example, this translates into effective repair using CI at chloride levels of up to about 1% by mass of cement. For chloride levels between roughly 1 and 2%, a reduction in corrosion activity, however to a lesser degree, was also observed. To decide on the best time of CI application it must be understood that CI perform better if treatment is done earlier in the life of a structure when chloride contents are lower and the philosophy of prevention is applied rather than the philosophy of repair.

In general, when used to reduce chloride-induced corrosion of steel reinforcement, surface-applied CI tends to be most effective at lower concrete quality, ie concretes with high water binder ratios.[27.56, 27.57, 27.58] The effectiveness of migrating corrosion inhibitors appears to be enhanced when used in combination with hydrophobic coatings that reduce moisture levels in concrete. Rowen[27.50] monitored the performance of migrating CI on marine structures in the Western Cape and concluded that penetration depth is increased and inhibitor performance is improved when the inhibitor is applied in conjunction with a hydrophobic surface treatment.

In conjunction with conventional patch repairs, CI can be applied to protect repairs and adjacent unrepaired areas and thus be incorporated into a holistic repair approach that makes use of a combination of repair methods to create a project-specific repair system.

27.3.6 Temporary electrochemical techniques

Corrosion of reinforcement in concrete is an electrochemical process that occurs when embedded steel is depassivated by a reduction in concrete alkalinity or the presence of corrosive ions such as chlorides (see Chapter 9). Two repair techniques, electrochemical chloride removal (ECR) and realkalisation of carbonated concrete, attempt to restore passivating conditions by the temporary application of a strong electric field to the cover concrete region. Both methods are essentially temporary cathodic protection systems and require electrical continuity in the reinforcement and a reasonably uniform conductivity of the concrete, no short-circuits between anode and cathode, and no insulation layers on the reinforcement or concrete surface. The amount of patch repair work necessary can be minimised when electrochemical techniques are applied and can thus prove to be more cost-effective.

Realkalisation is the process of restoring the original alkalinity of carbonated concrete in a non-destructive manner. The electrochemical treatment consists of placing an anode system and sodium carbonate electrolyte on the concrete surface and applying a high current density (typically 1 A/m^2). The electrical field generates hydroxyl ions at the reinforcement and draws alkalis into the concrete, raising its pH to about 12. Alkaline conditions may be restored in the concrete in as little as one to two weeks using the system. This method must be used with caution when applied to prestressed concrete members, as hydrogen evolution at the reinforcement may damage the prestressing strands. Internationally, there has been strong growth in the use of this method in recent years, presumably because of its greater convenience and cost advantage over extensive patch repair.[27.3]

Chloride removal, also commonly termed electro-chemical chloride extraction or desalination, is induced by applying a direct current between the reinforcement and an electrode that is placed temporarily on the outside of the concrete. The impressed current creates an electric field in the concrete that causes negatively charged ions to migrate from the reinforcement to the external anode. The process usually takes about 6-8 weeks to complete. This technique is similar in operation to cathodic protection but utilises a much higher electrical current density (typically 1-2 A/m^2).

Chloride removal decreases the potential of the reinforcement, increases the hydroxyl ion concentration and decreases the chloride concentration around the steel thereby restoring passivating conditions. Figure 27.11 shows the basic principles of ECR. Electrochemical chloride removal is a time-consuming and complex technique and its suitability needs to be carefully assessed. Care needs to be exercised in relation to hydrogen evolution in the member, as with other electro-chemical methods. Effectiveness and limitations of ECR are discussed in the literature.[27.1, 27.3, 27.4]

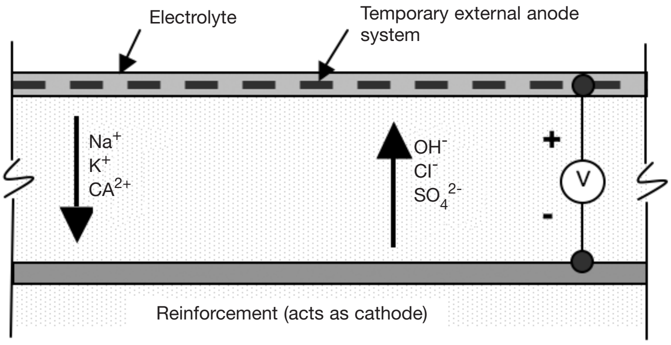


Figure 27.11: Schematic illustration of the electrochemical chloride removal technique

Due to the generation of hydroxyl ions and the ingress of metal alkalis, it needs to be ensured that alkali-reactive aggregates are not present in the concrete in order to prevent alkai-silica reaction (ASR) from occurring.

27.3.7 Cathodic protection systems

Chloride extraction and re-alkalisation are temporary repair processes. Cathodic protection is a similar technique but permanent. Cathodic protection systems (CP) have an excellent track record in long-term corrosion control of steel and reinforced concrete structures.[27.1] The principle of CP is that the electrical potential of the steel reinforcement is artificially decreased by providing an additional anode system, usually at the concrete surface, thereby converting the steel reinforcement into a large cathode. An external current is required between anode and cathode that diminishes the corrosion rate along embedded reinforcement. The current may be produced either by a sacrificial anode system or using an impressed current from an external power source. Figure 27.12 illustrates a typical cathodic protection layout. Sacrificial anode systems consist of metals higher than steel in the electrochemical series (eg zinc or titanium). The external anode corrodes preferentially to the steel and supplies electrons to the cathodic steel surface. The most widely used system consists of a surface-applied titanium

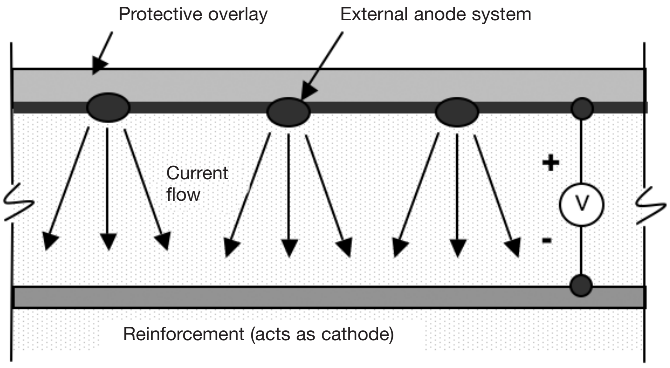


Figure 27.12: Typical cathodic protection layout

mesh anode with a cementitious overlay.[27.3] Other common systems include electroconductive tape systems, surface-applied zinc anodes, or internal anodes installed in slots or drilled holes. Sacrificial anode systems are generally most effective in submerged structures where the concrete is wet and resistivity is low, and in regions of warm temperatures (ie above 20°C).

Discrete sacrificial anodes, embedded in the substrate concrete next to patch repairs, may be used to prevent the formation of incipient anodes in the vicinity of the patch (500 – 750 mm diameter area of influence).[27.59] This may prevent the patch from failing due to ongoing corrosion but does not prevent the formation of anodes further away from the patch. Bonding agents for patch material application should be used with caution in such cases, as the bonding layer may prevent the passage of current to the reinforcement within the repaired zone.[27.48]

Impressed current systems, which are more commonly used, require an external electrical power source to supply electrons from anode to cathode. The anode is placed near the surface and is connected to the reinforcement through a transformer rectifier that supplies the impressed current. The primary anodes may consist of special metal alloys or polymer composites and are usually linked by electrically conductive material, such as conductive overlays or conductive paint systems. An installation will typically be divided into a number of zones on the structure, each with its own power supply. Experience with recent application of such a CP system in South Africa was presented by Newmark.[27.60]

Several factors need to be considered before applying CP systems. Any necessary repair works, eg patch repairs, to the concrete need to be carried out before applying CP. Electrical continuity of the reinforcement must be ensured for the system to work and the concrete should have similar conductivity throughout the structure. For concrete structures with old types of prestressing steel, the risk of hydrogen embrittlement needs to be analysed when considering implementation of impressed current CP systems.[27.3] Anode systems are usually designed for a minimum service life of 20 years. Special aspects to consider for the application of CP systems are discussed in the literature.[27.1, 27.3, 27.4] Before an electro-chemical system is installed, the potential for ASR needs to be assessed, as mentioned in Section 27.3.6.

27.3.8 Crack repair

The majority of concrete structures show a certain degree of surface cracking, which is often of no great concern. Cracks that do not have structural implications, are not unsightly, and are unlikely to be subjected to the ingress of moisture or other contaminants are generally best left alone. However, cracks may be problematic if they are aesthetically unacceptable, result in non-watertight structures, affect the durability of the structure, or are structurally significant.[27.7] The two repair options available are application of crack-bridging coatings, or filling of cracks. With crack-bridging coatings, it is possible to bridge cracks up to 5 mm in width.[27.3] However, if cracks are expected to have a significant impact on the performance of the structure, crack-filling techniques generally offer a more reliable repair option. The principal purposes of crack-filling are either minimisation of the ingress or passage of fluids, or the strengthening and/or stiffening of structural members.

If crack repair is necessary, it must first be established whether a crack is active ("live"), or inactive ("dormant"). Active cracks change in width when subjected to structural load, changes in environmental conditions, or ongoing deterioration. The material used to repair such cracks should be flexible to allow for movement, or otherwise the movement should be prevented. Flexible crack-filling material will generally not aid much in terms of strength and stiffness of the structure. Various flexible materials are available, including polyurethanes, polysulphides, as well as some polymer modified cementitious grouts.[27.3] Resins can generally not accommodate movement in active cracks caused by loading or temperature cycles.[27.7]

Inactive cracks, which are not subjected to major movement, can be repaired using a rigid material. Common techniques include gravity-feed crack sealing, as well as vacuum or pressure impregnation to induce penetration of low viscosity adhesives, such as epoxy, polyester or acrylic resins, or cementitious grouts. This method usually strengthens and stiffens the structure to a certain degree and can also be used for very fine cracks of 0,05 mm.[27.8] Vacuum and pressure impregnation are difficult operations that need to be carried out by qualified and experienced operators. Inactive cracks that are structurally insignificant are commonly repaired by routing and sealing, which is considered an easy and relatively fool-proof repair method.[27.7] The routing operation consists of chasing the crack along its length to open the gap sufficiently for filling with suitable materials.

27.3.9 Structural strengthening

Structural strengthening and retrofitting of concrete structures has become increasingly important in recent years, mainly due to structural degradation, or revised design requirements. Methods for structural strengthening generally include application of external load bearing elements such as reinforced overlays, bonded steel plates, or fibre reinforced composite (FRC) sheets or plates. More specialised methods include external post-tensioning with or without overlay systems.

Load jacking of columns and beams may be required when strengthening elements are intended to participate in bearing the dead load of the structure. For reinforced overlay systems, the principles discussed in Section 27.3.3 need to be considered in order to achieve sufficient interface bond strength for composite action. A newly developed overlay material gaining acceptance in Europe and North America is so-called Ultra-High-Performance Fibre Reinforced Concrete (UHPFRC).[27.60] Still in its development state, UHPFRC holds considerable promise especially for application as wearing surfaces of bridge decks.

Steel plate bonding has traditionally been used to strengthen concrete structures, particularly when additional flexural capacity is required.[27.59] It is mostly done on large bridge decks and beams with smooth surfaces that facilitate using bolted and welded connections. Mechanical anchorage of external reinforcement is of particular importance in view of long-term properties (strength and stiffness) of the system under the influence of creep and shrinkage. Additional shear bond strength for full composite action is commonly supplied using structural adhesives such as epoxy-based materials. In contrast to rigid steel plates, flexible FRC sheets can be bonded to uneven and rough surfaces and are relatively easily wrapped around columns. FRC materials consist of a large number of small fibres bundled in a resin matrix. The fibres are the principal stress bearing constituents while the resin transfers stress among fibres, protects them, and is commonly used to bond the system to the concrete surface. High strength and comparatively low weight of the material often gives it preference over conventional steel plate bonding techniques. Detailed information and guidance on design and application of FRP systems is presented in the literature.[27.62, 27.63]

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