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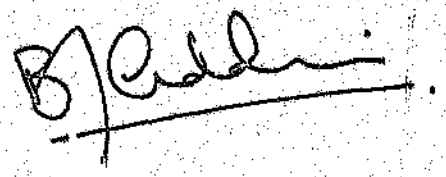
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PROPERTIES OF HIGH-STRENGTH CONCRETE MADE WITH SOUTH AFRICAN MATERIALS

BRIAN JEFFERY ADDIS

DECLARATION

I declare that this thesis is my own unaided work. It is being submitted for the degree of Doctor of Philosophy in the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination in any other university.



November 1991

ABSTRACT

This thesis concerns the properties of high–strength concrete. To make the work relevant to local economic conditions, South African materials, proportioned to produce mixes that could be compacted by conventional means, are the basis for all tests described. Properties of concrete are examined and the mechanisms responsible for them are explained where possible. Recommendations are made for the selection of materials and the proportioning of mixes. Economic implications of using high–strength concrete are investigated.

The workability of fresh concrete is dealt with by examining the factors which play a role in determining the workability of cement paste and the effect of paste workability on the concrete. It is shown that superplasticizing admixtures are an essential ingredient in high–strength concrete and the performance of various types is compared.

It is shown that high–strength concrete cannot be proportioned by methods used for conventional concrete. A method for proportioning mixes for high–strength concretes is developed and tested.

Chemical and volumetric factors which influence the hydration of cement are examined. The concept of cement–saturation is introduced and further chapters show that cement–undersaturated and cement–oversaturated mixes exhibit fundamentally different properties in the mature state. The possibility of self desiccation of cement paste in high–strength concrete is discussed.

The effect of stone particle size and the relative importance of aggregate and hardened cement paste on strength are investigated.

Various cements and various aggregate types are compared with regard to compressive strength. Recommendations for the selection of materials obtainable in South Africa are given.

Measured compressive strength of a given concrete is influenced by aspect ratio of the specimen. A study to investigate whether this influence is dependent on strength is described. An investigation into the strength of in situ concrete is reported from the literature. Within-batch variation of concrete strength of laboratory concretes is analysed. All of the above work indicated that methods of testing concrete need not be modified for high-strength concrete.

The movement-related properties: modulus of elasticity, creep and shrinkage, are discussed. Test results support the view that modulus of elasticity and creep of cement-oversaturated mixes differ from those of cement-undersaturated mixes.

Durability of high-strength concrete is discussed. It is concluded that such concretes are potentially very durable in most conditions.

Economic implications of the use of high strength concrete are dealt with by examples from the literature and by designing and costing a hypothetical column. The effect of column size on usable floor space is taken into account by introducing the concept of cost-area. Calculations show that high-strength concrete made from readily available local materials is economically advantageous for use in columns.

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1. INTRODUCTION

High–strength concrete is concrete of which the strength at 28 days is higher than that of concrete used for conventional structural work. The strength level which separates high–strength concrete from conventional concrete is taken in this thesis to be about 60 MPa. Most conventional concrete on the other hand falls within the strength range of 20 to 40 MPa.

Originally, high–strength concrete was developed and used for specialist, relatively low–volume applications such as precast concrete and prestressed concrete. Although high–strength concrete is still used in these ways, modern high–strength concrete was developed, and has been used, mainly for the columns of very tall buildings.

High–strength concrete differs, in respect of ingredients and mix proportions, from conventional concrete by:

- including a superplasticizing admixture;
- being made of selected aggregates;
- being made of selected cementitious materials;
- having a relatively high cement content;
- having a relatively low water content.

Because of these differences, the properties of high–strength concrete and typical conventional concrete differ significantly.

The object of this thesis is to:

- examine the properties of high–strength concrete;
- explain, where possible, the mechanisms responsible for these properties;
- investigate the use of South African materials for high–strength concrete;
- make recommendations for the selection of materials, especially those available in South Africa, for high–strength concrete;

- make recommendations for the proportioning of mixes for high-strength concrete;
- examine the economic implications of using high-strength concrete in South Africa.

The thesis begins with a survey of the technical literature dealing with high-strength concrete. Although a few older references are included where they serve to provide background information, the emphasis is on recently published work.

The aims of work undertaken are then discussed in the light of information provided by the literature survey.

A schedule of experimental work is included.

The main body of the thesis is divided into chapters dealing with the following aspects of high-strength concrete:

- workability
- mix proportioning
- the effect of high cement:water ratios on hydration
- strength
- movement properties
- durability
- economic implications.

Each aspect was investigated by examining existing information available in the technical literature and from other sources. Where appropriate, tests and/or calculations were carried out, and the results analysed. These investigations are described and, where possible, recommendations based on the investigation are given.

The emphasis of the work generally is on material properties. Although a section on the design of reinforced-concrete members is included in the literature survey, this

aspect is not pursued further in the body of the thesis except where specific material properties have a direct influence on structural design.

Although the aspects of high-strength concrete dealt with are universal, all experimental work was done with aggregates, cementitious materials and admixtures obtainable through normal commercial channels in South Africa.

Conventional mixing equipment and practice were used for making mixes. In addition, concrete workability was such that site compaction using conventional equipment would be possible. Concretes made with exotic aggregates and/or to a consistence requiring excessive compactive effort are therefore outside the scope of the work.

2. LITERATURE SURVEY

2.1. The development of high-strength concrete

2.1.1 History and applications

The dividing line between normal-strength and high-strength concrete is arbitrary and has changed over the years. The history of the development of high- strength concrete can therefore not be seen in isolation but as part of the history of the development of the strength of concrete in general.

Malinowski et al[2.1] report that ancient Roman concretes found in Caesarea and Tiberius had strengths of between 6 and 25 MPa.

Draffin[2.2] reports that samples of mortar made with natural cement (made between 1818 and 1940 in the United States of America of calcined argilliferous limestone) and taken from a building foundation had an average compressive strength of 80 MPa at an age of 35 years. Similar samples made with a different cement had a strength of 33 MPa at an age of 60 years.

It is doubtful whether the early users of concrete had more than a qualitative notion of the strength of concrete. As the technology developed, strength became a measurable parameter and designers could apply rational design methods which related expected stresses to the strength of the concrete to be used.

The Concrete Society[2.3] (United Kingdom) in 1987 published a summary of permissible design stresses in various codes of practice between 1910 and 1972 as shown in Table 2.1.

Table 2.1: Changes in permissible stress in codes of practice[2.3]

Date	Code	Allowed stress for design purposes for beams
1910	Manual of reinforced concrete and block construction	600 psi (4 MPa) or 0,25 of test cube strength
1913	Cassel's reinforced concrete	600 psi (4 MPa) or 0,33 of test cube strength
1930/39	London County Council	900 psi (6 MPa)
1948	CP114	1 000 psi (7 MPa)
1957	CP114	1 000 psi (7 MPa)
1969	CP114	0,37 x specified cube strength eg 1 110 psi for 3 000 psi (7,7 for 21 MPa
1972	CP110	0,4 x characteristic cube strength eg 1 200 psi for 3 000 psi (8,3 for 21 MPa

Increases in the strengths of concretes used for structures apparently encouraged the manufacturers of cement to improve the strength efficiency of their product.

Gonnerman and Lerch[2.4] report that the strength of portland cement made in the United States of America increased between 1920 and 1940. The 28-day strength of cylinders made of concrete batched to fixed proportions and with a cement:water ratio of 2,4 was 21 MPa in 1920 and 42 MPa in 1940.

Draffin[2.5] reports that the strength of British cements, as reflected by 28-day strengths of vibrated mortar cubes, made with a cement:aggregate ratio of 1:3 and a cement:water ratio of 2,5, increased from 51 MPa for the period 1943–1950 to 56 MPa for the period 1978–1979.

Russell[2.6, 2.7] states that the development and applications of high-strength concrete have occurred over many years. As development has occurred, the definition of high-strength concrete has changed. In the 1950s, concrete with a compressive strength

of 35 MPa cylinder strength was considered high strength. In 1960s, commercial usage of 40 and 50 MPa was achieved. In the early 1970s, 60-MPa concrete was being used. During the mid 1980s, design strengths of 75 to 95 MPa were being considered for commercial application in buildings.

Pioneering work in the development and use of high-strength concrete for structures was done in Chicago during the 1960s and 1970s. A document published by the Chicago Committee on high-rise buildings[2.8] describes the reasons for the development of high-strength concrete in that city.

A combination of high property values and limited space in the city encouraged the construction of tall buildings. To make these buildings practically possible and economically viable, columns at the lower levels had to be as small as possible in cross-sectional area. Strengths used at various times are summarized in Table 2.2.

Table 2.2: Concrete used in the Chicago area - highest strength

Year	Highest strength* (MPa)	Project where first used
Up to 1962	35 [41]	
1962	42 [49]	Lake Shore Plaza
1965	53 [62]	Lake Point Tower 1130 South Michigan Avenue
1972	63 [74]	Midcontinental Plaza Building

* ASTM cylinders. Approximate cube strengths (assuming a cube strength:cylinder strength ratio of 1/0,85) are given in brackets.

Parrott[2.9] reports strengths of concrete used for some major projects in the United States between 1972 and 1987 as shown in Table 2.3.

Table 2.3: Strengths of concrete used in various projects between 1979 and 1988[2.9]

Year	Highest strength* (MPa)	Project where first used
1972	66 [78]	Union Square Building, Chicago
1984	70 [82]	Century Square, Seattle
1987	131 [154]	Two Union Square, Seattle

* ASTM cylinders. Approximate cube strengths (assuming a cube strength:cylinder strength ratio of 1/0,85) are given in brackets.

Russell[2.10] states that the largest use of high-strength concrete has been in columns of buildings. But high-strength concrete was being used (in the mid 1980s) increasingly in bridge structures. Examples quoted in the reference are a prestressed concrete girder in the Tower Road Bridge in Washington State and a box girder in the Shubenacadie Bridge in Nova Scotia, Canada. High-strength concrete has also been used in cable-stayed bridges.

Russell[2.10] also mentions the use of high-strength concrete in off-shore oil exploration and production structures for the US and Canadian Arctic. More recently, high-strength concrete has been used for floors in warehouses, foundries, and parking garages; bridge deck overlays; dam spillways; and for heavy-duty industrial floors. In these applications it is used to provide improved resistance to chemical attack, better abrasion resistance, improved freeze-thaw durability, and reduced permeability.

High-strength concrete has also been used extensively for off-shore structures in the North Sea.

Jakobsen et al[2.11] report that between 1971 and 1987, 18 large concrete off-shore structures were built or ordered for the North Seal oil field for water depths ranging from 70 to 216 m. The platforms consist in total of about 2×10^6 m^3 of concrete. Specified characteristic cube strength increased from 45 MPa in 1974 to 70 MPa in 1976.

Holand[2.12] states that specified cube strength for North Sea oil structures increased from 45 MPa in 1974 to 85 MPa in 1986.

Other applications of high-strength concrete are reported and further innovative uses proposed in the literature.

Glaever et al[2.13] report that the largest segmental cantilever concrete box girder in Norway (1987) had a main span of 230 m and a maximum section height of 11,5 m. Alternative designs using various strengths of concrete showed that construction costs decreased as concrete strength was increased in steps from 45 to 65 to 95 MPa.

Glaever et al[2.13] also state that the development of high-strength concrete has made concrete truss girders more attractive because member cross sections may be reduced. (It is not clear whether they use the word attractive in the economic or aesthetic sense; presumably both would apply.)

Other applications of high-strength concrete proposed by Glaever et al[2.14] include "sandwich" structures in which layers of concrete of different strengths form a composite; foundation piles; bridge railings; rehabilitation of bridges; and wearing surfaces.

2.1.2 Achieving high strengths : some early proposals and attempts

Increased concrete strengths have probably long been a challenge to concrete technologists. Some older published work on high-strength concrete is, in the light of current knowledge and practice, interesting and sometimes quaint.

An article in the ACI Journal of January 1967[2.15] is an example. Titled "Methods of achieving high strength concrete", it proposes techniques which include compression of the fresh cement paste, with or without vibration; very-high-frequency vibration; grading of cement particle sizes; the use of portland cement clinker as an aggregate; autoclaving a silica-lime mixture; and synthetic organic resins as binders.

Perhaps the most significant feature of the article is that superplasticizers are not mentioned.

In 1969 Parrott[2.16] reported the results of an attempt to make high-strength concrete in the laboratory. Using a combination of low water:cement ratio (0,28) and stiff mixes (Vebe times of between 12 and 20 seconds), he achieved 28-day cube strengths ranging from 83 to 100 MPa. No admixtures were used and cement contents are not reported.

A document titled "High-strength concrete" published by the Portland Cement Association in 1971[2.17] gives recommendations for proportioning high-strength concrete. Some interesting aspects of the document are:

- The highest 28-day cylinder strength given is about 50 MPa. Assuming a conversion factor of 1/0,85 this is equivalent to a cube strength of 59 MPa.

- The most efficient mix given requires a cement content of 500 kg/m^3 to achieve a 28-day cylinder strength of 50 MPa. Assuming this is equivalent to a cube strength of 59 MPa, cement efficiency* is 11,8 MPa/100 kg cement/m^3.

- No mention is made of superplasticizing admixtures.

The proposals given in references 2.15, 2.16 and 2.17 are generally impractical; results fall far short of those achieved currently. The reason is that the authors of these references failed to appreciate that superplasticizers are an important, if not essential, ingredient of high-strength concrete.[2.18, 2.23, 2.26] (This notwithstanding the fact that superplasticizers were commercially available in 1964 – see section 2.2.4.2 below.)

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* Cement efficiency =

$$\frac{\text{cube strength (MPa)}}{\text{mass of cement per cubic metre of concrete (kg)} \times 100}$$

2.2 Selection of materials for high-strength concrete

2.2.1 Microstructure

Mehta and Aitcin[2.18] address the problem of selection of materials for high-strength concrete from a microstructural standpoint. In the reference they discuss the principles governing the strength of brittle solids. Fundamental principles governing the behaviour of brittle solids such as ceramics are applied to the microstructure and properties of concrete. In the reference, Mehta and Aitcin make the following points:

- Porosity, grain size and presence of inhomogeneities are factors which control the tensile strength of ceramics.
- The compressive strength of brittle materials is much greater than the tensile strength because in tension the material can fail by rapid propagation of a single flaw or crack.
- In general, compressive strength decreases with increasing pore size and increases with decreasing grain size.
- Strength and fracture depend critically on microstructural extremes rather than averages.

The strength of non-metallic, brittle, crystalline solids can therefore be improved by paying attention to the following principles:

- Porosity: A large number of big pores (ie > 50 nm), especially when concentrated in one location, are detrimental to strength.
- Grain size: In general, the strength of a crystalline phase increases with decreasing grain size.
- Inhomogeneities: In multi-phase materials, microstructural inhomogeneities are a source of strength loss.

Mehta and Aitcin then apply the above principles to concrete:

The hydrated cement paste consists of products of hydration, unhydrated paste, and pores. Some of the hydration products form outside the boundaries of the original cement grains and some form inside these boundaries. There is a distinct difference between these products, which can be referred to as the outer and inner products respectively. The outer products, which are more crystalline and less compact than the inner products, are the weaker. This explains why the path of fracture tends to go through the outer rather than the inner products. The concept of outer and inner products in cement hydration is useful in understanding the role of low water:cement ratio, superplasticizers, and finely divided mineral additions. The composite material, concrete, is weaker than either of its two components, namely aggregate and hardened cement paste. The reason for this apparent anomaly is the interface between aggregate and paste, is, compared with the bulk paste, reduced by the presence of large pores and large crystalline products. The difference between interface and bulk paste properties is especially pronounced around coarse aggregate particles. Because concrete failure occurs mainly in tension, a chain model can be used to illustrate the strength of concrete. In this model, the transition zone is the weakest link and therefore where failure is initiated and where most failure occurs. Cracking of the transition zone may also be caused prior to loading by shrinkage of the cement paste.

The primary requirements for high concrete strength are low porosity, hydration products which are poorly crystalline, and a homogeneous microstructure. The selection and proportioning of materials for high-strength concrete are based on these requirements.

2.2.2 Selection of aggregates

According to Mehta and Aitcin,[2.18] aggregates should be selected for the lowest possible water requirement. Grading, maximum size and shape are properties which are relevant. With regard to maximum size of coarse aggregate, Mehta and Aitcin state that a change in size can result in two conflicting effects. Larger coarse aggregate tends to reduce water requirement but makes the mix dimensionally more

heterogeneous and particles are more likely to include flaws. The first of these effects tends to increase strength while the second and third tend to reduce strength. They state that it is difficult to produce high–strength concrete with a maximum aggregate size of more than 25 mm. With most natural aggregate the optimum size is probably 10 to 12 mm.

A document titled "High–strength concrete" published by the Portland Cement Association in 1971[2.17] recommends smaller coarse aggregate for higher–strength concretes. According to the document, the aggregate sizes giving the most efficient mixes are 38 mm, 25 mm, 15 mm and 10 mm for cylinder strengths of 28 MPa, 35 MPa, 42 MPa and 50 MPa respectively.

Aggregates should be selected also for high intrinsic strength and freedom from flaws. Mehta and Aitcin [2.16] made concretes with granite, gravel, limestone and diabase. The granite concrete was the weakest and limestone and diabase gave the highest strengths. It was concluded that the granite aggregate was weak.

2.2.3 Selection of cement and cementitious materials

Regarding the selection of cement type, Mehta and Aitcin[2.16] quote results of tests on flow rates of pastes made with cements of different C_3A content. Rapid loss of flow was generally associated with high C_3A content. Cements of high C_3A content are therefore to be avoided for the manufacture of high–strength concrete.

Djellouli et al[2.19] propose the substitution of materials such as ground granulated blastfurnace slag, fly ash and condensed silica fume for part of the portland cement in order to retard the rate of slump loss. They also contend that such substitutions can reduce the required superplasticizer dosage in cases where the portland cement has a high C_3A content.

According to Mehta and Aitcin,[2.16] finely divided mineral additives can be used to improve the microstructure of concrete and so enhance strength and durability. Condensed silica fume (CSF) has proved to be especially effective. CSF particles,

which are much finer than cement, improve cohesiveness and workability. By segmenting the channels of water flow, which normally exist between cement and aggregate particles, CSF reduces bleeding and segregation and so improves homogeneity, especially at the aggregate–paste interface. In addition, well dispersed fine particles of a material which does not decompose as quickly as cement in water form numerous nucleation sites for the precipitation of cement hydration products. The result is a more homogeneous hardened cement paste.

The efficiency of CSF has been studied by various workers.

Yogendran et al[2.20] investigated the strength efficiency of CSF in cement paste at two different water:cement ratios (where cement is the total of portland cement and CSF): 0,28 and 0,48. They found that CSF was considerably more efficient in the leaner mixes. At W/C = 0,48, for example, CSF used at 15 % of the cementitious material increased 28–day strength by about 50 % compared with zero CSF. At W/C = 0,28, an increase in CSF content from zero to 25 % generally reduced strengths.

Sellevold and Nilsen[2.21] report that various investigators found that 2 to 5 kg of cement in concrete could be replaced with 1 kg of condensed silica fume (CSF) without reducing the compressive strength of the concrete. Some investigators found that CSF efficiency increased with increasing richness of the mix while others found the opposite to be true.

2.2.4 Admixtures

The importance of admixtures which improve workability and/or reduce water requirements of the concrete is generally recognized in the recent literature.

2.2.4.1 Reasons for use and best types

Mehta and Aitcin.[2.18] state that chemical admixtures are used to reduce water requirement while maintaining the desired workability. A water–cement system tends to form flocs in which large volumes of water are trapped. The entrapped water does not contribute to workability and its presence leads to local areas of high porosity in

the hydrated cement paste. Water–reducing surfactants, which prevent flocs from forming should therefore result in high strength if the water content is reduced for a given cement content and slump. In fact the reduction in water:cement ratio achieved by the use of such an admixture produces a greater strength increase than that produced by the reduction of water:cement ratio by increasing the cement content.

Normal water–reducing admixtures, derived from salts of sulphonated lignin, hydroxy acids or hydroxylated polymers, have a limited effect on water requirement. Higher–than–recommended dosage of such admixtures causes undesirable side effects such as excessive retardation and reduced early strengths.

High–range water reducers or plasticizers are capable of reducing water requirement significantly without the above side effects. Derived from sulphonated melamine formaldehyde or sulphonated naphthalene formaldehyde, superplasticizers are high molecular mass, anionic surfactants with linear molecules that do not form flocs. Dosed at up to 2 % solid by mass of cement they make it possible to achieve high slumps at very low water:cement ratios.

El–Zanaty[2.22] discusses the relative effectiveness of plasticizers and superplasticizers. Lignosulphonate–type plasticizers are not suitable for the manufacture of high–strength concrete: at normal dosages water reduction is insufficient while high dosages have a severe and unacceptable retarding effect. When using the same dosage, based on mass of cement, sulphonated naphthalene formaldehyde superplasticizers are more effective than sulphonated melamine formaldehyde superplasticizers. Both types have been used successfully to produce high–strength concrete but at different dosages.

2.2.4.2 History of the use of superplasticizers

According to Mielenz,[2.23] sulphonated melamine formaldehyde condensates were first produced and tested in Germany during the 1960s. They were offered for sale in 1964 and their use increased rapidly thereafter. They were used primarily to produce very high fluidity of the fresh concrete while avoiding unacceptable segregation and

bleeding. A secondary object was the achieving of high strength, as in precast structural members.

Products based on sulphonated naphthalene formaldehyde condensates were developed in Japan during the early 1960s and were introduced for sale in 1964. They were widely accepted for production of very-high-strength precast structural members.

The use of superplasticizers (or high-range water reducers) spread rapidly on an international basis during the early 1970s, including the United Kingdom, the United States, Canada, and elsewhere.

In 1980, ASTM 494[2.24] was modified to include superplasticizers. No compositional requirements are stipulated; specifications are of a performance nature: the required minimum reduction of water requirement is 12 %.

2.2.4.3 Mechanism of superplasticizers

According to El-Zanaty,[2.25] superplasticizers exert their action by:

- decreasing the surface tension of water;
- equidirectional charging of cement particles;
- producing a lubricating film at particle surfaces (slip action).

Superplasticizers based on sulphonated melamine formaldehyde condensate act mainly by forming a lubricating film at the particle surfaces. Superplasticizers based on sulphonated naphthalene formaldehyde condensates act mainly by charging the surfaces of cement particles equidirectionally to form a lubricating film at the particle surfaces; they decrease the surface tension of water to only a minor degree.

The mode of action of superplasticizers is described as follows:

> "These admixtures are thought to be adsorbed onto cement particles, causing them to become mutually

repulsive as a result of the anionic nature of superplasticizers, which causes the cement particles to become negatively charged. In principle, this adsorption and dispersing effect is similar to that found for normal anionic plasticizers."

Hattori[2.26] reports on the measurement of zeta–potential of cement particles in water. Before the addition of superplasticizer (sulphonate naphthalene formaldehyde) zeta–potential was +9,7 mV; with addition of a small amount of superplasticizer it decreased rapidly to about –15 mV. As the equilibrium concentration approached 0,8 %, zeta–potential reached a lowest value of –34 mV.

2.2.4.4 Dosage of superplasticizers

El–Zanaty[2.27] recommends minimum dosages of superplasticizers (by mass of cement): sulphonated melamine formaldehyde type: 2 %; sulphonated naphthalene formaldehyde type: 1 %.

2.2.4.5 Selection of superplasticizer

Penttala[2.28] used the Vicat apparatus to compare the consistence of combinations of paste and superplasticizer in order to find the optimum combination for high–strength concrete.

This approach seems at first glance to be a way of optimizing the selection of materials by using very small quantities. But examination of Penttala's results shows that the method could be used only for very rich pastes: W/C from 0,20 to 0,28. Such pastes tend to be impractically stiff and are seldom required for the proportioning of high–strength concrete.

2.2.5 Proportioning of mixes

Mehta and Aitcin[2.29] state that the ACI Committee 211[2.30] recommendations, which are applicable to concrete strengths between 13 and 40 MPa, have been successfully extrapolated to determine proportions for concrete strengths up to 60 MPa. The ACI

recommendations are not applicable to strengths above 60 MPa (ie high-strength concretes) because such concretes:

- Contain a substantial amount of unhydrated cement.
- Have a compact microstructure and a strong aggregate-paste interface zone.
- Have strength characteristics which are highly sensitive to aggregate type and small changes in water content of the concrete mix.
- Do not show any direct relationship between water:cement ratio (or water:cementitious material ratio) and strength.

A different approach is therefore needed for determining mix proportions. The process of arriving at the best combination of cement, water, aggregate and admixtures involves balancing various conflicting requirements.

Mehta and Aitcin[2.29] have developed a method of proportioning the trial mixes used for the development of mixes for site use. They use a total cement paste content of 25 % of the concrete by volume for concretes in the range of 65 to 120 MPa. Water content depends on the strength required and not on aggregate characteristics; water contents for various strength ranges are proposed. Consistence is controlled by adjusting the dosage of superplasticizer. Strength is controlled by controlling the quality of the cement paste, and not by increasing the ratio of cement paste to aggregate. As a first approximation, the ratio of fine aggregate to coarse aggregate is assumed to be 0,67 by volume.

The approach proposed in reference 2.29 appears to be rather arbitrary and does not take cognizance of specific aggregate properties.

Danielsen[2.31] states that cement content of high-strength concrete should be limited to 400 to 450 kg/m^3 in order to keep temperature increases resulting from heat of

hydration within acceptable limits. He also states that a cube strength of 90 MPa requires a water:cement ratio of 0,34 to 0,36. This means that total water content should be about 150 ℓ/m^3. Such low water contents are possible only if effective admixtures are used and aggregate properties are optimized.

Attempts have been made to quantify the relation between strength and cement:water ratio for high-strength concretes.

Hattori,[2.32] using a sulphonated naphthalene formaldehyde superplasticizer established the following relation between compressive strength of concrete, containing between 400 and 600 kg/m^3 of cement, and cement:water ratio:

$$f_{28} = 25{,}2\ C/W - 5{,}6\ (MPa) \quad \text{.....(2.1)}$$

where f_{28} is the cylinder compressive strength at 28 days, and C and W are the cement and water content respectively in kg/m^3 of concrete.

Suzuki[2.33] made a series of mixes using 20-mm stone and different cement types of cement in order to investigate the relation between strength and cement:water ratio. For a slump between 80 and 180 mm, he found the following relations:

Cement type	Plasticizer type	Relation*
Ordinary portland	Naphthalane	$F_{28} = 13{,}5 + 16{,}9\ C/W$ .....(2.2)
	Lignosulphonate	$F_{28} = 9{,}3 + 18{,}5\ C/W$ .....(2.3)
High early strength	Naphthalene	$F_{28} = \ldots 15{,}2\ C/W$ .....(2.4)

* F_{28} is 28-day compressive strength of concrete; C/W is cement:water ratio

It is interesting to note that, according to equation 2.4, concrete made with rapid-hardening (ie high early strength) cement and a naphthalene type superplasticizer would have a 28-day strength of about 75 MPa at a cement:water ratio of 3,0.

The linear form of the strength C/W function in references 2.32 and 2.33 is surprising and appears to be somewhat simplistic, especially in view of the fact that high-strength concrete contains a significant amount of unhydrated cement.[2.42]

Strength efficiency

In order to achieve the greatest possible dimensional stability in the hardened state, concrete should be made with the highest possible aggregate content and the lowest possible paste content. Paste content is a function of cement content and water content. Therefore in order to achieve low paste contents and maximum economy in high-strength concrete, cements and other materials should be selected and mixes proportioned so that high strengths can be achieved with the use of the minimum amount of cement or cement plus cement extender. Strength efficiency is a parameter that can be used to assess the efficiency of the combination of cementitious and other materials and of the mix proportions.

In the following, mix proportions and strength results reported in the literature are analysed in terms of cement-efficiency factor (CEF) which is calculated by dividing the equivalent cube strength at 28 days by the content of portland cement and cement extender per cubic metre of concrete and multiplying the result by 100. Units of CEF are therefore MPa/100 kg cementitious material/m^3 of concrete.

Mukherjee et al[2.34] made concrete with high percentages of fly ash and superplasticizers. With cement and fly ash contents of 390 and 230 kg/m^3 respectively, a water content of 174 ℓ/m^3 and a high dosage of superplasticizer, he achieved 28-day compressive strengths of up to 53,8 MPa cylinder strength. This strength is equivalent to about 63 MPa, assuming a strength ratio of cube to cylinder of 1/0,85. The best cement-efficiency factor, based on cube strength, was therefore 10,2 MPa/100 kg cementitious material/m^3.

Concrete mixes used for two high-rise buildings in Chicago are given in a task force report[2.35] prepared for the Chicago Committee on High-rise buildings. The buildings, Water Tower Place and River Plaza, were built in 1975/76. Both mixes contained

about 500 kg cement and 60 kg fly ash per m^3. Slump was 210 mm for both mixes. 28–day cylinder strengths were 66 MPa; assuming a conversion factor of 1/0,85, this is equivalent to a cube strength of 77 MPa. Cement–efficiency factor was therefore 13,8 MPa/100 kg cementitious material/m^3.

The relatively low efficiency is probably the result of relatively high water contents: 177 and 195 ℓ/m^3 for Water Tower Place and River Plaza respectively.

Haug and Jakobsen[2.54] reported mix data and strength results of concrete used for three offshore platforms. Maximum size of coarse aggregate was 20 mm. Cementitious content was either 400 kg/m^3 of portland cement only or 408 kg/m^3 of portland cement and condensed silica fume. Average cube strengths at 28 days ranged between 67 and 76 MPa and cementitious efficiency ranged between 16,7 and 19,8 MPa/100 kg cementitious material/m^3.

Gopalan and Haque[2.37] reported strengths of concrete mixes made with portland cement and different additions of fly ash. Superplasticizer was used to reduce water requirement. Slumps were between 50 and 95 mm. Maximum size of coarse aggregate was 20 mm. Total cementitious content for the series of mixes ranged between 399 and 738 kg/m^3. Water contents ranged between 171 and 201 ℓ/m^3. Strengths of cylinders after 28 days curing in 95 % relative humidity ranged between 68 and 82 MPa. Using a conversion factor of 1/0,85, these strengths are equivalent to cube strengths ranging between about 80 and 96 Mpa. Cementitious efficiency factor therefore ranged between 11,8 and 20,9 MPa/100 kg cementitious material/m^3.

Aitcin et al[2.38] used a combination of selected aggregates (dolomite), portland cement, condensed silica fume (CSF), superplasticizer and retarder to make high–strength concrete. Contents of cement and CSF were 550 and 55 kg/m^3 respectively. Superplasticizer and retarder dosages were 2,8 % and 0,14 % of cement mass respectively. Slump was 105 mm. Cylinder strength at 28 days was 108 MPa; assuming a conversion factor of 1/0,85, this is equivalent to a cube strength of

127 MPa. Cement efficiency factor, based on cube strength was therefore 21 MPa/100 kg cementitious material/m^3.

Ryell and Bickley[2.39] describe the concrete mixes used for the construction of Scotia Plaza, a 68-storey building built in 1987 in Canada. Mixes included: 10-mm limestone coarse aggregate; three-way blends of portland cement, ground granulated blastfurnace slag and condensed silica fume; and superplasticizer. Slumps were about 200 mm. Total cementitious contents were between 467 and 472 kg/m^3. At 28 days, cylinder strengths were about 88 MPa; this, assuming a conversion factor of 1/0,85, is equivalent to a cube strength of 103 MPa. Average cement-efficiency factor based on portland cement plus slag plus condensed silica fume was therefore 22 MPa/100 kg cementitious material/m^3.

Colepardi and Corradi[2.40] made concrete mixes with three different cement types and two commercial brands of sulphonated naphthalene formaldehyde superplasticizer available in Italy. Cement types were ASTM V, I and III. Water:cement ratios ranged from 0,26 to 0,35 and slumps were all about 200 mm. With cement contents between 413 and 503 kg/m^3 they obtained 28-day cube compressive strengths between 83 and 104 MPa. Cement efficiency factors at 28 days ranged from 16,5 to 24,2 MPa/100 kg cement/m^3.

The best overall cement efficiency was obtained with the type III cement, followed by type I and type V in that order. It is worth noting that the concrete mixes with the lower cement contents gave the best cement efficiencies for all cement types.

De Larrard et al[2.41] reported mix characteristics and strengths as follows:

Concrete description	Cement content (kg/m^3)	CSF content (kg/m^3)	Total cementitious (kg/m^3)	Water content (ℓ/m^3)	Superplasticizer content (kg/m^3)	Slump (mm)	Cylinder strength* (MPa)
High strength	425	0	425	145	6,75	200	76
Very high strength	421	42	463	112	7,6	200	105

* 160 mm diameter x 320 mm high cylinders; strengths at 28 days.

If a value of 0,85 is assumed for the ratio of cylinder strength to cube strength, cement efficiency factors, based on cube strength, are 21,0 and 26,7 MPa/100 kg cementitious material/m^3 for "high-strength" and "very-high-strength" concretes respectively.

In summary, this part of the literature survey:

- Shows the importance of microstructural aspects in the selection of materials and proportioning of mixes.
- Gives vague guidance on the optimum size of coarse aggregate.
- Suggests that cements with high C_3A content should be avoided.
- Recommends the use of CSF but reports gives contradictory results for its efficiency.
- Explains why water-reducing agents are necessary and why they should be of the superplasticizer type.
- Gives a history of the use of superplasticizers and describes their mechanisms in fresh concrete.
- Gives some approaches to mix proportioning.
- Reports on attempts to relate strength to cement:water ratio.
- Reports on cement-efficiency factors ranging from 10,2 to 26,7 MPa/100 kg cementitious material/m^3.

2.3 Microstructure and microcracking

The properties of hardened concrete are the result mainly of its microstructure. The literature on high-strength concrete includes reports of various investigations into microstructure and microcracking under different loading conditions.

2.3.1 Microstructure and pore size

Aitcin et al[2.42] used a scanning electron microscope (SEM) to study the microstructure of hardened concrete which achieved a cylinder strength of 135 MPa at six months. Condensed silica fume (CSF) had been used at a dosage of between 6 and 8 % of the cement mass. Superplasticizer and retarder dosages were 2,8 % and 0,14 % of cement mass respectively.

The SEM study showed that:

1. The cement paste was very dense

2. Some unreacted CSF grains were still present.

3. Some unreacted C_3S was still present.

4. The aggregate-paste interface was free of the weak calcium hydroxide zone which is found in normal-strength concretes.

Aitcin et al also used a mercury porosimeter to study pore-size distribution. They found that pores between 0,035 µm and 2,7 µm were present but were completely closed.

Sarkar and Aitcin[2.43] used mercury porosimetry and scanning electron microscopy-energy dispersive X-ray analysis (SEM-EDXA) to study the microstructure of normal-strength concrete (water:cement ratio 0,56, 28-day cylinder strength 32 MPa) and high-strength concretes (water:cement ratio 0,24 to 0,21, 28-day cylinder strengths 84 to 103 MPa). Some of the high-strength concretes contained condensed silica fume.

They found that pore-size distribution of normal-strength and high-strength concretes differed significantly. The total volume of intruded mercury at the highest pressure (150 MPa which corresponded to a pore entrance radius of about 0,005 μm) was about twice as great in the normal-strength concrete as in the high-strength concrete. On the other hand the various high-strength concretes had similar pore-size distributions.

They also found differences in the calcium hydroxide content and the nature of the aggregate-paste interface. High-strength concretes contain a lower proportion of calcium hydroxide than normal-strength concrete. The aggregate-paste interface in high-strength concretes is stronger and is characterized by direct contact between the calcium silicate hydrate and the aggregate. In normal-strength concretes this interface is relatively weak; in some places it takes the form of a microfissure which contains calcium hydroxide crystals.

2.3.2 Microcracking of high-strength concrete subject to short-term loading

Carrasquillo et al[2.44] studied the microcracking of concrete under short-term loading. Specimens were loaded in a compression machine to predetermined percentages (50, 70 and 90 %) of strain at maximum load. They were then removed from the compression machine, returned to the curing room, and within 24 hours of loading, a horizontal slice about 4 mm thick cut from mid height. Slices were X-rayed and the cracks mapped on the X-ray film.

They found it appropriate to classify cracks as "simple" cracks (ie at the coarse aggregate-mortar interface) and "combined" cracks which develop when cracks through the mortar link "simple" cracks. As load on the concrete is increased, simple cracks develop first, followed by combined cracks which then form complex combined cracks leading to disruptive failure. Simple cracks are generally stable: they will not grow into combined cracks and lead to failure unless the load is increased. But combined cracks are generally unstable and may propagate and lead to failure under constant load.

Three strength levels: normal-, medium- and high-strength (about 30, 55 and 75 MPa), were used for the investigation. Limestone aggregate was used in the concrete.

It was found that for normal-strength concrete (ie about 30-MPa compressive strength), a continuous increase in length and number of combined cracks occurred as the strain increased to 70 and 90 % of strain at failure. For medium-strength concrete at 90 % strain, an increase in length of combined cracks was observed but no additional combined cracks were developed. For high-strength concrete, the amount of combined cracks formed, even at 90 % strain, was insignificant.

The observed behaviour is explained by better compatibility of strength and elastic properties between coarse aggregate and mortar and higher tensile bond strength in concretes with higher strength.

The study indicated that high-strength concrete can be loaded to a higher stress-strength ratio without initiating a self-propagating mechanism leading to disruptive failure: the sustained-load strength is a higher fraction of the short-term strength.

Nilson[P.45] states that the shape of the compressive stress-strain curve can be related to, and explained by, progressive internal cracking with increasing or sustained load. Microcracks start on the interface between coarse-aggregate inclusions and mortar. Normal and shearing stresses cause slip and some separation. As load is increased these bond cracks branch into the mortar.

The interconnected network of bond and mortar cracks results in material discontinuity and further load increase will produce failure.

There are significant differences between low- and high-strength concrete. For low-strength concrete, microcracking starts at about 30 % of ultimate strength, and at 90 % of ultimate strength, interconnected cracking is well established. For high-strength concrete, there is very little microcracking at loads below 65 % of ultimate; even at

90 %, cracking is mostly in the form of isolated bond cracks. These differences are explained by the closer match of strength and stiffness of stone and mortar, and the higher interface bond strength associated with cement-rich mixes.

Unfortunately, Nilson does not define the terms "low strength" and "high strength" used in the reference. It appears however that strengths of about 85 MPa are regarded as "high" while strengths of about 20 to 30 MPa are considered to be "low".

2.3.3 Microcracking and time-dependent strains

Ngab et al[2.46] studied internal microcracking of concrete specimens subjected to drying shrinkage, short-term loading and sustained loading. Two compressive-strength levels were used: 33 to 40 MPa ("normal") and 62 to 70 MPa ("high") cylinder strengths. Cracks were marked on photographs of 4-mm-thick slices of the specimens.
They found that:

- The amount of microcracking in high-strength concrete associated with shrinkage, short-term loading, and sustained loading is significantly less than in normal-strength concrete.

- Time-dependent increase in cracking occurs in normal- and high-strength concrete subject to shrinkage conditions or sustained loading; but to a far lesser extent in high-strength concrete. They believe that this explains, in part, the smaller creep of high-strength concrete.

- The amount of microcracking for both normal and high-strength concrete appears to be approximately linearly related to strain, regardless of whether the strain is caused by short-term or sustained load, or by shrinkage.

- High-strength concrete exhibits a discontinuity stress (the beginning of an unstable form of mortar cracking which is self-propagating and leads to failure under sustained stress) at a higher fraction of its short-term compressive strength than does normal-strength concrete. The sustained-load strength of high-strength concrete is therefore relatively higher.

- The stress–creep relationship for high–strength concrete is linear to a higher percentage of the compressive strength than that of normal–strength concrete because of the smaller amount of microcracking of the former at high stress levels.

Nilson[2.47] investigated the behaviour of concrete under sustained loading by carrying out two types of tests: creep tests at sustained loads from 40 to 85 % of ultimate strength, and time dependent failure tests at sustained loads from 75 to 90 % of short–term strength. Nilson states that three stages of behaviour exist: linear creep, for which creep strain is proportional to stress and elastic strain, thus a constant creep coefficient is applicable; nonlinear creep, for which creep strains become disproportionately larger than creep coefficient times elastic strain; and failure under high sustained load which is less than short–term strength. Differences between microcracking of low– and high–strength concrete are summarized below:

Stage	Stress as percentage of short–term strength		Microcracking
	Low–strength concrete	High–strength concrete	
Linear creep	to 45 %	to 65 %	Some increase in bond cracks for low–strength; negligible for high
Nonlinear creep	to 75 %	to 85 %	Bond cracking increases for both
Failure	above 75 %	above 85 %	Bond cracks, mortar cracks, and combined cracks increase sharply

In summary, this part of the literature survey:

- describes the microstructure of high–strength concrete as dense with strong contact at the aggregate–paste interface;

- compares the microcracking of concrete of different strengths under short–term loading and explains why the sustained–load strength, expressed as a fraction

of short-term strength, of high-strength concrete is higher than that of normal-strength concrete;

- compares the microcracking of concrete of different strengths under sustained loading.

2.4 Engineering properties

In this section, the following engineering properties are dealt with:

Compressive strength
Tensile strength
Compressive stress-strain curve
Modulus of elasticity
Creep and shrinkage.

2.4.1 Compressive strength

2.4.1.1 Failure mode in uniaxial compression

Carrasquillo et al[2.48] carried out strength tests on concretes at three strength levels and studied the mode of failure in each case. These levels were designated "normal", "medium" and "high" and corresponded to cylinder strengths of about 30, 55 and 75 MPa.

They concluded that:

1. A tensile or tensile-shear failure mechanism is the most relevant crack mechanism controlling failure of concrete in uniaxial compression. Failure of concrete in uniaxial compression occurs in a direction parallel to the direction of the applied load for concrete at all strength levels.

2. Normal-strength concretes develop highly irregular failure surfaces including a large amount of bond failure.

3. Medium–strength concretes develop a mechanism similar to normal–strength concretes but at a higher strain.

4. The failure mode of high–strength concretes is typical of that of a nearly homogeneous material: failure occurs suddenly in a vertical nearly flat plane passing through coarse aggregate and mortar.

2.4.1.2 Sustained–load strength

The strength of concrete under sustained loading is less than under short–term loading. Nilson[2.49] loaded concrete specimens of various strengths for 60 days. For low–strength concrete (≈ 25 MPa), stresses above about 75 % of compressive strength produced failure within the 60–day period. For high–strength concrete (≈ 80 MPa), stresses of up to 85 % of compressive strength could be sustained indefinitely.

2.4.1.3 Strength gain with age

Carrasquillo et al[2.50] made concrete cylinders at "normal", "medium" and "high" strength levels to investigate the rate of strength development. Cement was ASTM Type I for all mixes. Water:cement ratios were 0,70, 0,47 and 0,32 and strengths were about 30, 55 and 75 MPa for the three strength levels respectively.

They found that the high–strength concrete showed the highest rate of strength gain at early ages, followed by medium–strength and low–strength concrete in that order. At later ages, differences were not significant. Typical ratios of 7–day to 95–day strength were 0,60, 0,65 and 0,73 for normal–, medium–, and high–strength concrete respectively.

2.4.1.4 Specimen size effect

Carrasquillo et al[2.51] made concrete cylinders of two different sizes to investigate the effect of specimen size on strength at different strength levels. Cylinder sizes were 152 mm diameter x 305 mm long and 102 mm diameter x 203 mm long. Strength levels were about 30, 55 and 80 MPa for the smaller specimens. They found that,

regardless of strength level or test age, the average ratio of compressive strength of large cylinder to small cylinder was close to 0,90.

2.4.1.5 Variation of concrete strength

Hester[2.52] investigated the within–batch variation of concrete and found that there is little or no correlation between strength and within–batch standard deviation. He states that the overall standard deviation for high–strength concretes is no different than for low– or moderate–strength concretes, and that the within–test variation "should also be no different" and that "if high strength concretes are reported to be more variable than low or moderate strength concretes, this is probably due to use of poor or inappropriate testing practices".

An examination of the results shown in Figures 1 and 2 of the reference shows however that Hester's argument is based on so few data points that no meaningful conclusion should be drawn from them.

2.4.1.6 Effect of drying on strength

Carrasquillo et al[2.53] investigated the effects of drying conditions on the strength of concretes of different strengths. Two strength levels were used: about 30 and 75 MPa ("normal–" and "high"–strength respectively). Two different drying conditions were studied: moist curing for 7 days followed by drying at 50 % relative humidity until testing at 28 days, and moist curing for 28 days followed by drying at 50 % relative humidity until testing at 95 days.

They found that the higher–strength concrete showed a larger reduction in strength than the lower strength concrete when allowed to dry before completion of curing. Strength reductions were as follows:

Curing	Test age (days)	Strength reduction relative to moist cure entire time until tested	
		Normal strength	High strength
Moist 7 days; dry day 8 – day 28	28	4 %	10 %
Moist 28 days; dry day 29 – day 95	95	Negligible	4 %

No explanation for the difference is given in the reference. A possible reason is that high–strength concretes tend to self–desiccate and stop hydrating more readily because of their low water:cement ratios.

2.4.1.7 Strength of in situ concrete

Haug and Jakobsen[2.54] tested about 1 000 cores taken from three offshore platforms and compared core strengths with strengths of cubes used for monitoring strength. They found that the ratio of core strength to cube strength at the same age ranged between 0,74 and 0,85 for non–slipformed concrete and between 0,79 and 1,00 for slipformed concrete. They concluded that the superior performance of slipformed concrete was the result of better compaction.

2.4.2 Tensile strength

Nilson[2.55] investigated the tensile strength of high–strength concrete by determining modulus of rupture and split cylinder strength. He found that, for moist–cured concrete, tensile strength in both cases exceeded values predicted by the ACI code.[2.55] He cautions however that tensile strength of in situ concrete could be considerably lower than that of laboratory concrete because of poorer curing.

Marro[2.57] carried out flexural and tensile–splitting tests in order to characterize concrete used in an investigation into beam behaviour. He found that flexural strengths generally exceeded those predicted by the ACI Committee 363 formulae.[2.56] Splitting–tensile strengths were generally slightly lower than those predicted by CEB[2.56] and

ACI[2.56] formulae. Concrete cube strengths used for the tests varied from 35 MPa to 145 MPa.

Carrasquillo et al[2.59] carried out flexural strength (modulus of rupture) tests on "normal–", "medium–" and "high–strength" concretes made with limestone aggregate. Compressive strength levels were about 30, 55 and 75 MPa (cylinder strength) respectively. Testing was done on 102 x 102 x 356–mm beams loaded at third points. They found that the amount of aggregate fracture along the fracture plane was substantially larger for high–strength concrete than for normal–strength concrete.

They recommend the following equation for predicting modulus of rupture:

$$f_r' = 0{,}94 \sqrt{f_c'} \text{ MPa} \quad \text{.....(2.5)}$$

for 21 MPa < f_c' < 83 MPa

where f_r' = modulus of rupture

f_c' = cylinder compressive strength.

Carrasquillo et al[2.59] also carried out tensile splitting strength tests on normal– and high–strength concretes described above using 102 x 203–mm cylinders.

They recommended the following equation for predicting tensile splitting strength:

$$f_{sp}' = 0{,}54 \sqrt{f_c'} \text{ MPa} \quad \text{.....(2.6)}$$

for 21 MPa < f_c' < 83 MPa

where f_{sp}' = splitting tensile strength

and f_c' = compressive strength (152 x 305–mm cylinder).

2.4.3 Compressive stress–strain curve

Nilson[2.60] used 100– x 200–mm cylinders to investigate compressive stress–strain curves of concretes with different strengths. Results are shown in Figure 2.1.

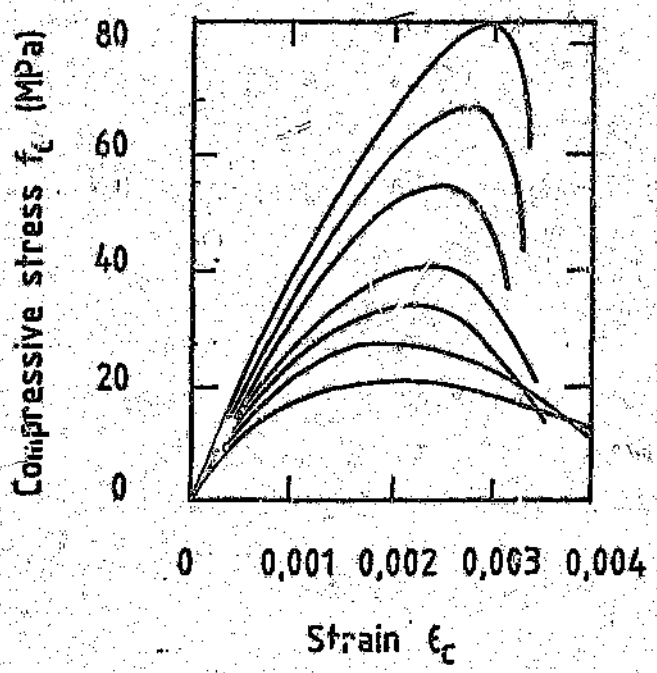


FIGURE 2.1: Typical compressive stress–strain curves[2.60]

The curves in the figure indicate higher elastic modulus as concrete strength increases up to about 80 MPa. The range of approximately linear behaviour is extended to 80 to 90 % of maximum stress as strength increases to about 80 MPa. Strain at peak stress is about 0,002 for low–strength concrete, but increases to about 0,003 for high–strength concrete. Maximum strain is less for high–strength concrete than for low–strength concrete. For high–strength concrete there is a rapid drop of stress after peak stress is reached.

The long descending branch displayed for lower–strength concrete, corresponding to the gradual development and spread of microcracking, becomes steeper as concrete strength increases and is very steep in high–strength concrete.

2.4.4 Modulus of elasticity

Marro[2.61] found that static modulus of elasticity for concretes of cube strength from about 35 MPa to 145 MPa were generally slightly lower than predicted by the CEB formulae[2.62] and slightly higher than predicted by the ACI Committee 363 formulae.[2.63] Differences were however not significant. Comparisons were based on a ratio of 0,83 for cylinder strength to cube strength.

Brooks et al[2.63] made concretes with rapid–hardening portland cement with and without superplasticizer and fly ash. They found that static modulus of elasticity (E) was only marginally influenced by the combination of the above materials. Values for E were higher than predicted by Teychenne et al[2.64] and by ACI Committee 209[2.65] but lower than predicted by CEB–FIP formulae[2.66] Comparisons were based on a ratio of 0,80 for cylinder strength to cube strength.

Carrasquillo et al[2.67] measured the static modulus of elasticity of concretes of cylinder strengths ranging from 30 MPa to 75 MPa. They found that existing equations for predicting modulus of elasticity on the basis of strength were unreliable for the concretes tested. The ACI 318–77[2.68] equation predicts higher moduli than measured for concretes with cylinder strengths greater than about 40 MPa.

They propose the following equation for normal–density concretes:

$$E = 3\ 320\ \sqrt{f_c'} + 6\ 900\ \text{MPa} \quad \text{.....(2.7)}$$

for 21 MPa < f_c' < 83 MPa

where E = modulus of elasticity

f_c' = compressive strength (152 x 305–mm cylinder).

Nilson[2.69] found that the ACI Code equation[2.69] for modulus of elasticity (E_c) [E_c = 4 700 $\sqrt{f_c'}$ MPa, where f_c' is compressive strength in MPa] overestimates E_c for strengths in excess of about 30 MPa; at a strength of 85 MPa overestimation is about 20 %. The reason is that concrete strength is controlled mainly by mortar strength;

concrete stiffness is influenced by the mortar and the stone. Consequently an increase in mortar strength will increase concrete strength more than stiffness. Nilson proposes the following equation for E_c for high-strength concrete:

$$E_c = 3\,300\sqrt{f_c'} + 6\,900 \text{ MPa} \quad \text{.....(2.8)}$$

(Equation 2.8 is virtually identical to equation 2.7 above.)

2.4.5 Creep and shrinkage

Ngab et al[2.70] investigated time-dependent deformation of high-strength concrete. High-strength concrete was compared with conventional concrete. They found that:

- The creep coefficient and specific creep of concrete are smaller for concretes with higher compressive strength. This behaviour is more significant for drying conditions (which usually prevail in practice). Drying high-strength concrete has a specific creep of 20 to 25 % of that of drying normal-strength concrete. Under nondrying conditions, this percentage is 30 to 35 %. The creep coefficient of high-strength concrete under drying conditions is about 50 to 75 % of that of normal strength. Under nondrying conditions the percentage is 75 to 90 %.

- The stress-creep relationship of high-strength concrete is linear to about 70 % of maximum stress; this is significantly higher than for normal-strength concrete.

- The most important parameters which seem to affect that part of creep not associated with microcracking are the compressive strength at loading and the amount of water in the system prior to and while the specimen is under load.

- Shrinkage of high-strength concrete may be greater than for normal strength, but not significantly so.

Penttala and Rautanen[2.71] measured mass losses of concrete specimens used to determine creep and shrinkage.

They found that:

- Creep and shrinkage deformations of high–strength concrete take place more rapidly than those of normal–strength concrete.

- Curves of mass loss versus time were similar in shape to curves of creep and shrinkage versus time. The degree of correlation between mass loss and creep and shrinkage deformations was high (0,90 to 0,99) except for slag concretes (0,63 and 0,60).

They conclude that the early creep and shrinkage deformations are governed by the evaporation of water from the concrete.

Penttala[2.72] compared the creep rate of high–strength concrete with the rate predicted by the CEB formulae.[2.73] He found that high–strength concretes tend to creep more rapidly during the first forty days and more slowly thereafter. After 180 days, high–strength concretes showed little or no increase in creep strain.

Penttala[2.74] found that the creep of high–strength concretes was significantly lower than that given by the CEB formulae.[2.73] Creep, after one year, of high–strength concrete made with condensed silica fume, fly ash, and ground granulated blastfurnace slag was 80 %, 60 %, and 40 % respectively of the CEB values.

Nilson[2.75] found that high–strength concrete had significantly lower creep coefficient than low–strength concrete. (Creep coefficient is the ratio of additional time–dependent strain in uniaxial compression to initial elastic strain.) Creep coefficient of concrete with a compressive strength of 69 MPa was roughly half of that of concrete with a compressive strength of 21 MPa.

Penttala[2.76] found that shrinkage deformations of high-strength concretes were similar to those predicted by CEB formulae[2.77] when the concretes were made with condensed silica fume and fly ash. Shrinkage deformations were significantly higher when the concrete contained ground granulated blastfurnace slag.

2.5 Design of reinforced-concrete members

2.5.1 Design approach

Nilson[2.78] states that the use of HSC has preceded full knowledge of the material properties, or the behaviour of members made of the new material.

"In the US and elsewhere, many equations used for material relationships and for design of members are based on tests in which concrete strength was not more than about 40 MPa. Extrapolation to strengths that may be more than twice as great is not justified and may be unsafe."

2.5.2 Modifying existing codes to cater for high-strength concrete

Zia[2.79] discusses the approach to the modification of codes, which were developed for concrete with a relatively low compressive strength, in order to make them applicable to high-strength concrete. He states that the aspects that are most difficult to address are those which incorporate many empirically determined constants. Zia believes that many provisions would be conservative if applied to high-strength concrete but some may impose limitations on the effective use of high-strength concrete. He suggests that design values for elastic modulus, modulus of rupture and minimum flexural reinforcement should be revised. Revisions are also required in the areas of sheer and torsion.

2.5.3 Axially loaded columns

The design of axially loaded columns is discussed by ACI Committee 363.[2.80]

Design practice for conventional concrete is to assume a direct addition law summing the strength of the concrete and that of the steel. The justification for this is shown in Figure 2.2 which superimposes typical stress-strain curves in compression for three

concretes with that for reinforcing steel having 415 MPa yield strength. The assumption is that steel and concrete strains are identical at any load stage.

For lower–strength concrete, when the concrete in a loaded reinforced column reaches the range of significant nonlinearity (about 1 000 microstrain) the steel is still in the elastic range and consequently starts to pick up a larger share of the load. When the strain is close to 2 000 microstrain, the slope of the concrete curve is nearly zero and it can be thought of as deforming plastically, with little or no increase in stress. The steel reaches its yield point at about the same strain in this case; ie when the concrete is at its maximum stress.

The strength of the column is therefore predicted by

$$P = 0{,}85\, f_c' A_c + f_y A_s \quad \text{.....(2.9)}$$

where f_c' = cylinder compressive strength of concrete
f_y = yield strength of steel
A_c = area of concrete section
A_s = area of steel.

The factor of 0,85 is used to account for the observed difference in strength of concrete in columns compared with concrete of the same mix in standar compression–test (ASTM C42[2.81]) cylinders.

A similar analysis holds for high–strength concrete columns, except that the steel will yield before the concrete reaches its peak strength. But the steel will continue to yield until the concrete is fully stressed. Prediction of strength may therefore still be based on equation 2.9.

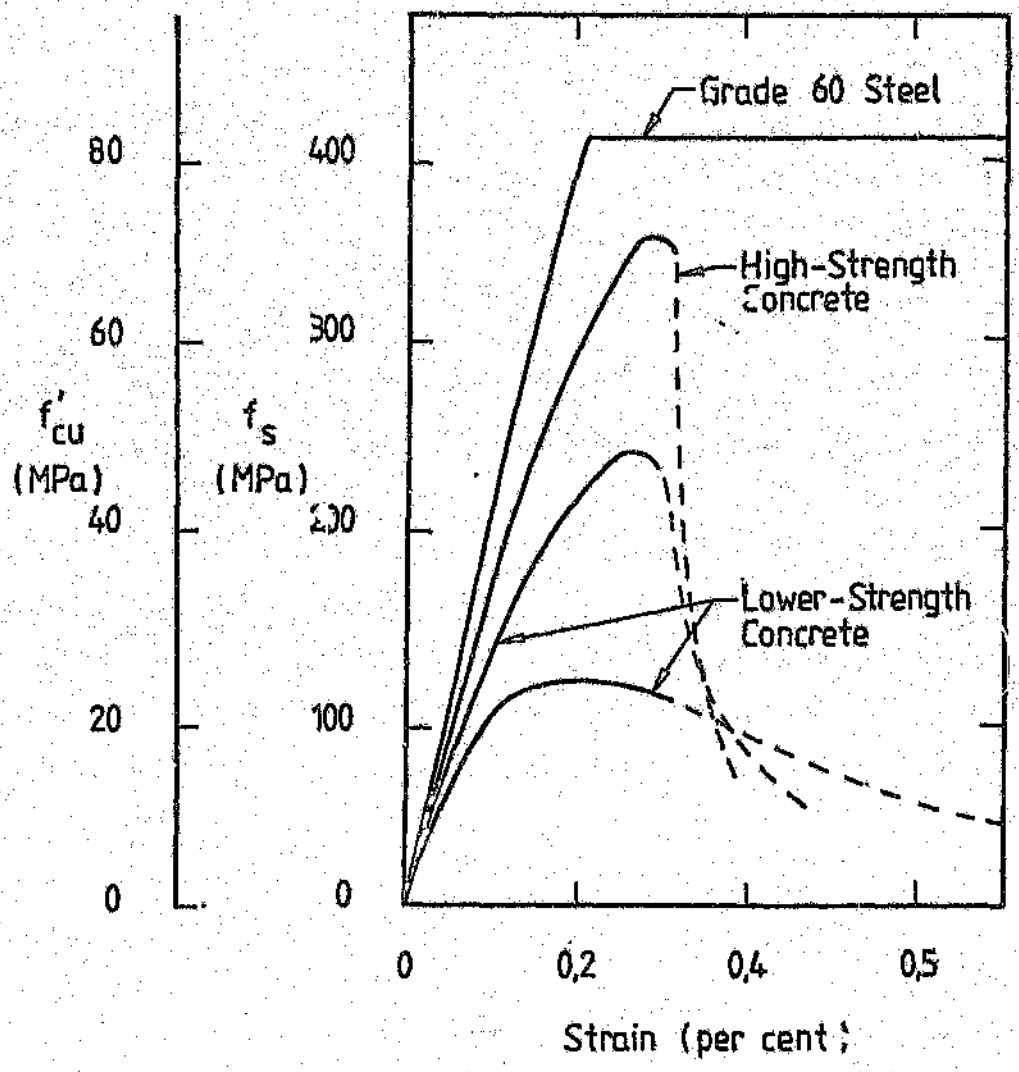


FIGURE 2.2: Stress–strain curves of concrete and steel in compression; the curve for steel is drawn to a different vertical scale[2.80]

Nilson[2.82] tested 94 columns to investigate the ductility of axially loaded columns in an overloaded state. In spite of the brittle nature of high-strength concrete, results showed that the strength gain resulting from the confinement effect of spiral reinforcement is accurately predicted by conventional equations. But the gain in ductility usually associated with increasing confinement stress is not obtained with high-strength concrete; there is a rapid drop of resistance after the peak stress is reached, even for large confinement pressures.

On the other hand, work by Mugurama and Watanabe [2.83] showed that a high degree of ductility could be achieved by using high-yield-strength lateral confining reinforcement in columns of high-strength concrete. They found that the effectiveness of confining reinforcement decreases with increasing concrete strength and that the use of high-yield-strength steel for lateral confining reinforcement is indispensable.

2.5.4 Beams and slabs

2.5.4.1 Compressive stress distribution

According to ACI Committee 363,[2.80] stress distribution in the compression zone of a member in bending is generally parabolic for concrete of normal strength. Stress distribution tends to be more linear for high-strength concrete; a combination of rectangle and triangle is proposed for the diagram. The Committee believes that, for under-reinforced beams, the ACI 318[2.84] method can be used without change, at least for concrete strengths up to 83 MPa (cylinder strength). For over-reinforced beams, which are not allowed by ACI 318, or for members combining compression and bending, important differences may occur.

2.5.4.2 Flexural-critical beams

The constant value of strain at extreme concrete compression fibre of 0,003 prescribed by ACI 318[2.84] is seen by ACI Committee 363[2.80] to represent satisfactorily the experimental results for high-strength as well as lower-strength concrete, although it is not as conservative for high-strength concrete.

Because of the difference in shape of the compressive stress-strain curve, and the more brittle nature of high-strength concrete, Nilson[2.85] investigated the applicability of flexural design procedures to beams of high-strength concrete. He tested ten eccentrically loaded compression specimens, to simulate the compression zone of beams, and twelve full-scale beams.

It was found that the flexural strain limit of 0,003 assumed in the ACI Code [2.84] was always attained, although strain at failure was less for high-strength concrete. Modelling of the compression zone by the rectangular stress-strain block gave good

results. Deflections at service load were predicted closely by the ACI Code procedure which uses a weighted average of gross section and cracked section moments of inertia.

The ACI Code[2.84] imposes a minimum steel ratio for beams in order to avoid rupture of the steel immediately upon tensile cracking of the concrete. Nilson[2.85] believes that, because of the higher flexural tensile strength of high-strength concrete, this ratio should increase with compressive strength. He proposes that

$$\rho_{min} = \frac{3\sqrt{f_c'}}{f_y} \quad \text{.....(2.10)}$$

where ρ_{min} is the minimum ratio of cross sectional area of steel to concrete,

f_c' is the compressive strength of the concrete,

and f_y is the yield stress of the steel.

2.5.4.3 Influence of confinement steel and compression steel

ACI Committee 363[2.80] reports results of tests done on beams of concrete of different strengths. Closer spacing of stirrups tended to improve ductility of beams of higher-strength concrete (59-MPa cylinder strength). The effect of increasing the amount of compression steel showed no clear trend.

2.5.4.4 Minimum tensile steel ratio

ACI Committee 363[2.80] recommends that the minimum tensile steel ratio should be adjusted to take into account the fact that modulus of rupture of high-strength concrete is greater than that of lower-strength concrete.

2.5.4.5 Shear and diagonal tension

ACI Committee 363[2.80] describes the failure surface of concrete of different strengths: lower-strength concrete tends to form a rugged surface while high-strength concrete tends to fail along a smooth surface that is nearly plane. Diagonal tension cracks in high-strength concrete can therefore be expected to be deficient in aggregate interlock.

The Committee does not make any recommendations for web reinforcement for beams of high–strength concrete.

Nilson[2.86] states that failure surfaces in high–strength concrete tend to be smoother and closer to a plane than those of low strength concrete. Along a diagonal tension crack in a beam, such a smooth surface would be deficient in providing aggregate interlock, one of the mechanisms by which a reinforced–concrete beam transmits shear. Nilson[2.86] tested nineteen reinforced–concrete beams which had been designed to fail in shear. He found that empirically based design equations in the ACI code to predict the concrete shear contribution were unsafe for high–strength concrete. He recommends that an upper limit for concrete compressive strength of about 55 MPa should be used for calculating shear strength of beams.

Mphonde[2.87] carried out tests on reinforced–concrete beams with and without web reinforcement; some beams were made with preformed diagonal cracks in positions which correspond with those of shear cracks. From the results obtained, he concluded that at low concrete compressive strengths the aggregate interlock mechanism operating in diagonal cracks induced by shear failure makes a significant contribution to the total shear carried by the beam. At higher concrete compressive strengths, crack faces are smoother and aggregate interlock contributes less to total shear capacity. On the other hand, dowel action of the longitudinal flexural steel plays a more predominant role at very high strengths.

Clarke[2.88] reviewed test data on the behaviour of high–strength concrete beams in shear and compared them with BS 8110[2.89] predictions. In BS 8110 the equation for design concrete shear stress is based on the characteristic concrete cube strength which may not exceed 40 MPa. This limitation reflects the lack of experimental data and the possible risks of extrapolation.

Clarke states that, on the basis of limited data available from shear tests on beams made with concrete having a cube strength between 70 and 120 MPa, the equation given in BS 8110 can be extended to take account of the higher strengths. This would

lead to an increase of 20 to 45 % in the shear stress for the concretes considered. Clarke recommends however that the minimum amount of shear reinforcement in beams should also be increased.

2.5.4.6 Bond, anchorage, and development

ACI Committee 363[2.60] states that insufficient work has been done on high-strength concrete to permit recommendations.

2.5.4.7 Cracking

According to ACI Committee 363,[2.60] modulus of rupture for high-strength concretes is higher than that predicted by the ACI 318[2.84] value of 0,62 $\sqrt{f_c'}$ (where f_c' is the cylinder compressive strength in MPa). But this applies to continuously moist-cured specimens. Concrete subjected to a limited period of moist curing would have a lower modulus of rupture and the ACI 318 value is therefore probably realistic for high-strength concrete in structural elements.

2.5.4.8 Elastic deflections

ACI Committee 363[2.60] states that the main uncertainties in predicting elastic deformations of reinforced concrete beams are elastic modulus; modulus of rupture; and effective moment of inertia, which depends on the extent of cracking of the beam.

The formula given in ACI 318[2.84] can be used for calculating beam deflection if actual values of elastic modulus and modulus of rupture used. If these are not known, modulus of elasticity should be based on the equation

$$E_c = 3\ 320 \sqrt{f_c'} + 6\ 900 \text{ MPa} \quad \text{.....(2.11)}$$

where E_c = modulus of elasticity
and f_c' = cylinder compressive strength;
and modulus of rupture is taken to be 0,62 $\sqrt{f_c'}$.

2.5.4.9 Time-dependent deflections

According to ACI 318,[2.84] time-dependent deflections of beams due to creep and shrinkage are calculated by applying multipliers to computed elastic deflections. ACI Committee 363[2.80] states that these multipliers should be lower for high-strength concrete than for normal-strength concrete.

Compression steel tends to reduce time-dependent deflections but increased concrete strength makes compression steel less effective in this regard.

Nilson[2.90] loaded nine beams and after the loads had been sustained for more than three years, was able to confirm that deflections of high-strength concrete beams under sustained load were less than those of low-strength concrete. The difference was accounted for by the lower creep coefficient of high-strength concrete.

2.6 Durability

High-strength concrete is a relatively new material and structures built of high-strength concrete are generally not old enough to indicate the degree of durability under field conditions. There is therefore little information on actual structures in the literature; most references report on laboratory studies.

2.6.1 Potential durability

According to studies by Aitc' , et al[2.42] and Aitcin,[2.43] the microstructure of high-strength is characterized by a high degree of compactness and impermeability. This indicates that high-strength concrete is potentially a highly durable form of concrete when exposed to aggressive conditions in which the mechanism of attack involves the ingress of gases or liquids into the concrete.

2.6.2 Off-shore structures

Jakobsen et al[2.91] report that the eleven North Sea concrete platforms inspected in 1982 (construction of the first started in 1971) were performing satisfactorily and could be expected to last their intended life withou deterioration, maintenance or repair.

2.6.3 Chloride penetration

Sarkar and Aitcin[2.43] measured chloride ion permeability of an air–entrained normal–strength concrete (water:cement ratio 0,50) and a high–strength concrete (water:cement ratio 0,24). The two concretes were made with the same cement and aggregate; the normal–strength concrete did not contain condensed silica fume while the high–strength concrete did.

Chloride permeability was measured according to AASHTO standard T277-?31[2.92] "Rapid determination of the chloride permeability of concrete" which measures an electrical charge. Results were 4 600 and 150 coulomb which indicate "high" and "negligible" permeability for normal– and high–strength concretes respectively.

2.6.4 Freezing and thawing conditions

Hammer and Sellevold[2.93] used two test methods to investigate the frost resistance of high–strength concrete made with and without air–entraining agents. These methods were a volume deterioration method (ASTM C666[2.94]) and a salt–scaling method (similar to ASTM C672[2.95]). They also used low–temperature calorimetry to measure ice formation in concretes after a drying–resaturation treatment.

The volume deterioration tests showed relatively severe damage to concretes with water–binder ratios down to 0,28. (No air–entrained concrete was tested in this way.) These results were in conflict with the calorimetry tests which showed only very minor ice formation at temperatures down to –20 °C. This anomaly suggests that damage may not be related to ice formation but to differences in thermal expansion coefficients of aggregates and paste.

Salt–scaling tests showed that high–strength concrete with water:binder ratios of less than about 0,37 exhibits acceptable resistance to salt–scaling, even without air entrainment.

Malhotra et al[2.96] investigated the resistance to freezing and thawing of high–strength concretes. Water:cementitious material ratio ranged from 0,25 to 0,36. Some mixes

included entrained air. Condensed silica fume (CSF) was used to replace part of the cement at rates of 0, 10 and 20 % by mass. Any loss of slump due to the use of condensed silica fume was compensated for by the use of a superplasticizer. Specimens were subjected to a freezing and thawing regime according to ASTM C666, Procedure A.[2.84] Resistance to repeated cycles of freezing and thawing was estimated by determining mass, length, resonant frequency, and pulse velocity of specimens before and after cycling and comparing these values with those of reference specimens.

They found that, regardless of water:cementitious material ratio, non–air–entrained concrete performed very poorly and failed after only 50 cycles. All air–entrained concretes without CSF were in excellent condition after 300 cycles; those with CSF performed less well.

It is worth noting that the test procedure does not permit drying of the specimens before the start of freeze–thaw cycling. It is possible that dried concrete would perform differently in the test.

2.6.5 Abrasion resistance

Gjørv et al[2.87] investigated the abrasion resistance of concrete of different strengths. Concrete strengths from 50 MPa to 150 MPa were used to make panels which were tested by subjecting the surface to the passage of heavy–duty tyres fitted with metal studs. Wheel loads of up to 2,5 tonnes were applied and the speed varied between 56 and 70 km/hr. They found that the wear of 100–MPa concrete was roughly 50 % of the wear of 50–MPa concrete. The wear of 150–MPa concrete was similar to that of massive granite elements. Tests were done on wet and dry concrete. At the 50–MPa strength level the wet abrasion was about twice the dry abrasion; at 150 MPa the difference was negligible.

2.6.6 Fire resistance

Shirley et al[2.98] investigated the fire endurance of high–strength concrete slabs. They found that the performance of high–strength concrete was not measurably different from that of conventional concrete.

2.7 Economic implications

There is relatively little information in the literature on the economic implications of using high–strength concrete. This is probably because costs are highly specific to place and time. Nevertheless trends could be expected to be universal.

According to the Chicago Committee on high–rise buildings,[2.99] economic considerations were behind the development of high–strength concrete for use in buildings in Chicago: A combination of high property values and limited space in the city encouraged the construction of tall buildings. To make these buildings practically possible and economically viable, columns at the lower levels had to be as small as possible in cross–sectional area. A decrease in size was achieved by an increase in concrete strength.

In 1975 Schmidt and Hoffman[2.100] compiled charts indicating that the cost of supporting 445 kN of service load in a structure in Chicago was as follows:

Cylinder strength of concrete (MPa)	41	52	62
Cost per storey ($)	5,02	4,21	3,65

The authors explain that the reason for the cost savings associated with higher strengths is that, although the concrete itself is more costly than lower–strer concrete, the cost differential is offset by a reduction in member size. This capability is particularly attractive for use in columns.

ACI Committee 363 reports on a pricing study done during 1983 in the United States of America to demonstrate the cost advantage of replacing percentages of steel with high–strength concrete in short tied columns.[2.101] This study was made for a column

supporting a design load (1.4D + 1.7L) of 4 450 kN and based on the following in–place prices:

Reinforcing steel	\$760/ton	(\$0,84/kg)
7 000 psi (48 MPa) concrete	\$80/yd^3	(\$104,43/m^3)
9 000 psi (62 MPa) concrete	\$85/yd^3	(\$110,97/m^3)
11 000 psi (76 MPa) concrete	\$104/yd^3	(\$135,77/m^3)
14 000 psi (97 MPa) concrete	\$129/yd^3	(\$168,41/m^3)
Formwork[a]	\$280/yd^3	(\$365,33/m^3)

As Figure 2.3 shows, using high–strength concrete with a minimum amount of steel is the most economical solution.

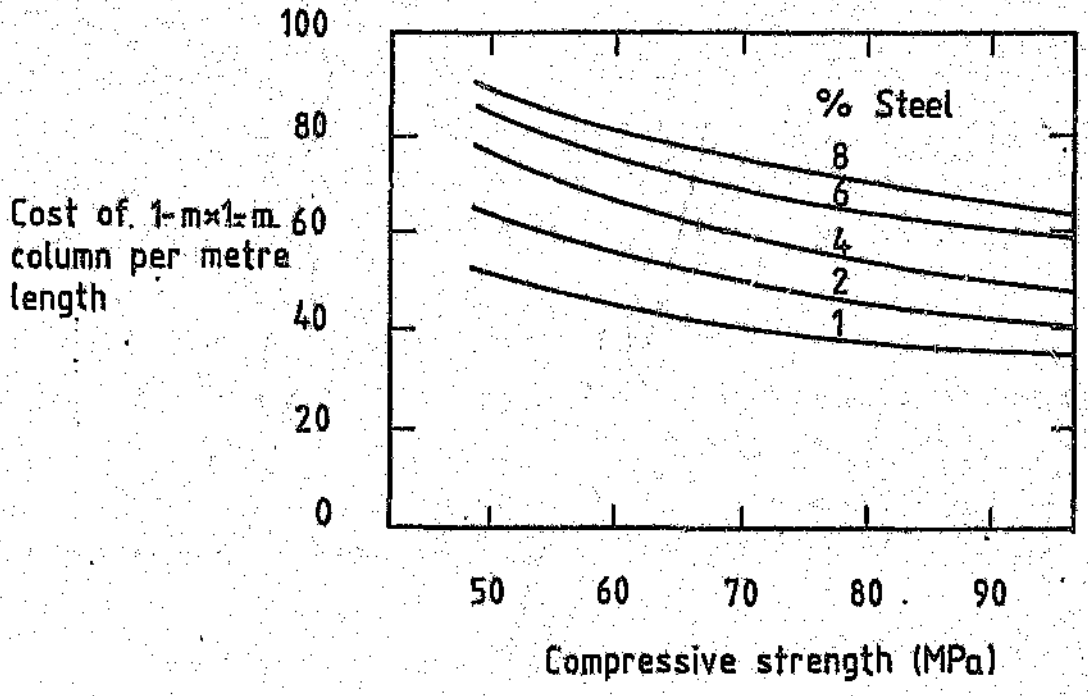


FIGURE 2.3: Cost of column versus compressive strength of concrete and for different percentages of steel[2.101]

[a] The rate is presumably per cubic yard of concrete

ACI Committee 363[2.102] states that there is no economic justification, under normal circumstances, for the use of a premium material such as high-strength concrete for slabs or beams.

Glaver et al[2.13] report on a cost comparison of various strengths of concrete for the construction of a large cantilever box girder in Norway: construction costs decreased as concrete strength was increased from 45 to 95 MPa.

2.8 Research needs

ACI Committee 363 believes that, despite the amount of research that has been performed on high-strength concrete, there are still research needs in many areas. These needs are summarized in reference 2.103.

Air entrainment requirements

Inadequate information is available to clarify the need for air entrainment with the low water:cement ratios used in high-strength concrete. Entrained air and high-strength concrete are somewhat incompatible. Consequently, new satisfactory entrained-air levels for resistance to freezing and thawing of high-strength concrete must be defined.

High-strength concrete for corrosion protection

Investigations are needed to determine the value of high-strength concrete in protecting reinforcing steel against corrosion. Reduction in chloride penetration rates and retention of high pH levels in the area surrounding the reinforcement through the use of high strength concrete should be studied and better identified.

Properties of high-strength concrete in actual applications

The properties of high-strength concretes are frequently determined based on specimens cured in an ideal laboratory environment. Information on the effect of curing procedures used in the field is needed to develop greater confidence in the use of the material. Field conditions would simulate the environmental changes as well as the heat of hydration effects produced by the concrete mass.

Evaluations should include the use of different chemical and mineral admixtures to produce the high-strength concretes and the effects of these admixtures on mechanical properties and durability over a long period of time. The role of mineral admixtures, such as condensed silica fume, for a high-strength concrete should be carefully examined.

Effects of aggregate characteristics

The properties of high-strength concrete are affected by the aggregate mineral properties and particle shape. Research work is needed to investigate the effects of various aggregate minerals and particle shapes on mechanical properties and durability.

Compaction of high-strength concrete

Current compaction procedures and equipment are based on experience with lower-strength concretes. Since high-strength concretes sometimes result in stickier mixtures, are new compaction procedures needed with the higher-strength concretes? Consideration should be given to more powerful vibrators, different frequency vibrators, and even ultrasonic vibration.

Development of compressive strengths in excess of 138 MPa

Research is needed to identify economical methods of achieving concrete compressive strengths in excess of 20 000 psi (138 MPa). Techniques are currently available, but they generally are too expensive for normal applications. Methods of obtaining ultra-high strengths without the use of exotic materials need to be developed.

Structural properties of high-strength concrete

Current design equations for development length of reinforcing bars and prestressing strands are based on lower-strength concretes. Verification is required that these design equations are applicable to higher-strength concretes.

Some data are available concerning the creep and shrinkage properties of concrete over a long time period. Additional information is needed to help design engineers and

to verify that specific creep of high-strength concrete continues to be lower than specific creep for lower-strength concretes. Research is needed to determine minimum amounts of flexural and shear reinforcement in beams and slabs made with high-strength concrete.

Currently, very little information is available about the fatigue strength of high-strength concrete. Additional data in this area are needed.

High-strength concrete in high seismic zones

Most high-strength concrete construction has taken place in zones of low seismicity. Research is needed to allow greater use of high-strength concrete in high seismic zones. Factors to be investigated include ductility, confinement reinforcement, and longitudinal reinforcement. Emphasis should be placed on achieving ductile behaviour of members under seismic conditions.

Long-term deflections of beams and slabs

Current design requirements for long-term deflections are based on a multiple of the short-term deflection. These multipliers are based on tests of lower-strength concretes. Preliminary information has indicated that deflections of beams of high-strength concretes are significantly less than those predicted from the deflection of lower-strength concrete beams. Data are needed to establish a basis for a modified equation for long-term deflections.

Testing requirements

Current ASTM procedures for testing concrete compressive strength are based on lower-strength concretes. A review of the procedures is necessary to determine if they are applicable to high-strength concretes.

Use of high-strength concrete

Construction with high-strength concrete has been somewhat limited by a lack of well-documented information to illustrate its proper use. A study is needed to show the

appropriate applications of high–strength concrete, including cost–benefit analyses for actual jobs.

High–strength fibre–reinforced concrete

The addition of fibre to high–strength concrete can improve its mechanical properties. Information is required for high–strength fibre–reinforced concrete regarding fibre amounts that can be efficiently placed and vibrated.

Efficient mixing of high–strength concrete

Some research has indicated that higher speed mixers are more efficient for mixing high–strength concretes. Research to identify mixer and mixing requirements for high–strength concrete is needed.

Abrasion resistance

High–strength concrete has been specified and used in applications where increased abrasion resistance is desired. More research is needed not only in regard to abrasion resistance but also on the practical aspects of placing and finishing high–strength concrete in flatwork. The question of whether high–strength concrete can be used in special abrasion applications such as superflat floors should be addressed.

Fire and high temperature effects

Little information is available on the fire resistance of high–strength concrete, particularly for those concretes that derive their high strength from a mineral admixture that increases the density of the paste. The question of temperature effects is also critically important, especially for long–term high temperatures such as those found in power plants, chemical plants, radioactive waste storage structures, etc. A recent report revealed the risk of explosion of high–strength concrete cylinders with very dense paste when heated in an oven. Research is needed in all these areas concerning short–term and long–term effects of fire and high temperature.

Proportioning and evaluating high-strength concrete

The requirements specified for proportioning and evaluating concrete strength in Chapter 4 of ACI 318 are based on statistical data from testing lower-strength concrete. Research is needed to develop and evaluate more strength data to determine whether the code requirements are realistic and adequate for high-strength concrete at levels considerably higher than 5 000 psi (34 MPa).

3. AIMS

The subject of high-strength concrete may conveniently be dealt with under the following headings:

Workability of fresh concrete
Proportioning of mixes
The effect of high cement:water ratios on the hydration of cement
Strength
Movement properties
Durability
Structural design
Economic implications.

In this chapter, the significance of each of these aspects is discussed. Important points found in the technical literature are reviewed. Aims of further work are proposed in the light of what is required to deal comprehensively with each aspect and what is provided by the literature survey.

3.1 Workability of fresh concrete

Although concrete is in the fresh state for only a few hours – a very short time compared with service life – its properties in the fresh state influence significantly its properties in the hardened state. In particular, workability is a property which determines what methods can be used, and what degree of success can be achieved, in handling, placing and compacting the concrete. The workability of conventional concrete is recognized by concrete technologists as being of major importance; the workability of high-strength concrete is probably as important and perhaps more critical.

In the literature survey, Mehta and Aitcin[2.16] give reasons for using admixtures to reduce water requirement. Without such admixtures, cement particles tend to form flocs in which water is trapped; this water does not contribute to workability. Water-reducing surfactants prevent flocs from forming. References 2.18 and 2.22 state that

the most effective admixtures are derived from sulphonated melamine formaldehyde and sulphonated naphthalene formaldehyde; such admixtures are known as superplasticizers. The mechanism of superplasticizers is described by El-Zanaty[2.25] and Hattori.[2.26] El-Zanaty states that they exert their action by a combination of decreasing the surface tension of water, charging cement particles equidirectionally, and producing a lubricating film at particle surfaces. Hattori reports on the effect of a superplasticizer on the zeta-potential of cement particles in water.

The above references give no more than a few basic facts on the role and mechanism of water-reducing admixtures. What is needed is an understanding of concrete workability and of the role of the ingredients of concrete in influencing workability.

A considerable body of empirically based knowledge on the workability of conventional concrete exists. It is necessary to establish whether this knowledge can be applied to high-strength concrete.

The aims of this section of the work are to:

- Describe the factors which influence the workability of cement paste.
- Investigate the effect of paste workability on the workability of concrete.
- Investigate the workability of rich mixes made without admixtures and the effect that admixtures have on workability of such mixes.
- Compare the workability of conventional concrete with that of high-strength concrete.

3.2. Proportioning of mixes

The ingredients of concrete must be proportioned to produce a mix that has the required properties in the fresh and hardened states.

To minimise waste of materials, time and other resources, it is necessary to use methods which make it possible to proportion suitable mixes quickly and efficiently.

In the literature survey, Mehta and Aitcin[2.29] give reasons why proportioning recommendations developed for conventional concrete cannot be used for high-strength concrete. The reasons are that high-strength concretes contain a substantial amount of unhydrated cement; they have a compact microstructure and a strong aggregate-interface zone; their strengths are highly sensitive to aggregate type and small changes in water content of the mix; and they do not show a direct relationship between water:cement ratio and strength.

Mehta and Aitcins' approach to mix proportioning is to use water content as a starting point; water content is determined by the concrete strength required. Total paste content is fixed at 25 % by volume of the concrete and consistence is controlled by adjusting the dosage of the superplasticizer.

Danielsen[2.31] limits cement content to 450 kg/m^3.

Various authors[2.32, 2.33] attempt to describe the relationship between 28-day compressive strength and cement:water ratio by means of linear equations.

While Mehta and Aitcins' reasons why conventional proportioning methods cannot be used for high-strength concrete are reasonable, their approach to proportioning high-strength concrete can be criticised:

- Assigning water contents to strength levels is arbitrary and does not take cognizance of the effect of aggregate or cement characteristics on the properties of fresh concrete;

- Fixing the paste content at 25 % is arbitrary;

- There is no means of establishing the optimum volume ratio of coarse aggregate to mortar.

A general linear relationship between strength and cement:water ratio seems to be unlikely; the expressions given in the literature are probably applicable to limited ranges of cement:water ratio and to very specific cases.

What is needed is a rational method of mix proportioning that is based on the characteristics of the aggregates and cementitious materials used in the mix.

The aim of this section of the work is to:

- Establish to what extent existing methods of mix proportioning are suitable for high–strength concrete.

- Establish whether such existing methods can be modified for high–strength concrete.

- Develop a rational method of proportioning mixes for high–strength concrete.

3.3 The effect of high cement:water ratios on the hydration of cement

High–strength concretes are characterized by high cement:water ratios. According to classic work done by Powers and his colleagues,[3.3, 3.4] the hydration of cement is constrained by lack of space when cement:water ratio exceeds a critical value.

In the literature survey, Mehta and Aitcin[2.29] remark on the substantial amount of unhydrated cement in high–strength concretes. The effect that this unhydrated cement has on strength and other properties of concrete is not dealt with in the literature.

The aim of this section of the work is to:

- Study the chemical reactions involved in the hydration of cement.

- Study the volumetric aspects of cement hydration.

- Explain the effect of unhydrated cement on the strength of concrete.

- Investigate the effect of unhydrated cement on the strength–efficiency of cement.

- Study the self–desiccation of pastes with high cement:water ratios.

3.4 Strength

Strength is the property of concrete which is specified and measured more often than any other property. This thesis is about a form of concrete which is distinctive by being stronger than normal or conventional concrete. It is appropriate therefore that a major part of the work should be concerned with strength.

The literature survey shows how concrete strengths, design stresses, and cement strengths have increased during the 20th century.[2.1, 2.2, 2.3, 2.4, 2.5] Over the years, the definition of high–strength concrete has changed.[2.6, 2.7] Indeed, a timeless definition is perhaps concrete with a strength higher than that of concrete normally manufactured for construction.

Some early proposals for techniques for making what is today understood to be high–strength concrete are given.[2.15, 2.16, 2.17] Most of these proposals are impractical, ineffective or inefficient; they do not include the use of superplasticizers to reduce water requirement.

The importance of selecting aggregates and cementitious materials is dealt within the literature:

- Mehta and Aitcin[2.18] state that aggregates should be selected for lowest possible water requirement.

- The nominal size of coarse aggregate is discussed by Mehta and Aitcin[2.18] and the Portland Cement Association.[2.17] There is not complete agreement on the optimum size: reference 2.18 states that size should be between 10 and 25 mm and that the probable best size is 10 to 12 mm; reference 2.17 recommends 10 mm but this is for a cylinder strength of only 50 MPa.

- Aggregates should have high intrinsic strength and be free of flaws.[2.18] Mehta and Aitcin[2.18] found that limestone and diabase aggregates gave the highest concrete strengths.

- Mehta and Aitcin[2.18] caution against the use of portland cement with a high C_3A content. Djellouli et al[2.19] suggest the substitution of materials such as ground granulated blastfurnace slag, fly ash and condensed silica fume for part of the portland cement. Mehta and Aitcin[2.18] advocate the use of condensed silica fume to improve the microstructure of concrete and enhance strength and durability. Various authors[2.20, 2.21] report on studies to compare the strength efficiency of condensed silica fume with that of cement.

The role of water–reducing admixtures in enhancing strength is dealt with by Mehta and Aitcin.[2.18] By preventing the formation of cement flocs, the presence of large volumes of trapped water is eliminated. Because these volumes of water lead to local areas of high porosity and consequent reduction in strength, water–reducing admixtures tend to increase strength for a given cement:water ratio.

Reports of mix proportions and consequent concrete strengths can be found in the literature. The literature survey includes an analysis of a number of these[2.34, 2.35, 2.36, 2.37, 2.38, 2.39, 2.40, 2.41] in terms of cement efficiency factor at 28 days which is expressed as MPa/100 kg cementitious material/m^3. Cube strength is used as a basis of comparison. Cement–efficiency factors ranged from 10,2 to 26,7 MPa/100 kg cementitious material/m^3.

The mode of failure in uniaxial compression is discussed by Carrasquillo et al.[2.48] Normal- and high-strength concretes both fail mainly by a tensile or tensile-shear failure mechanism. But normal-strength concretes develop highly irregular failure surfaces including a large amount of bond failure; high-strength concretes develop nearly flat failure surfaces passing through coarse aggregate and mortar.

Sustained-load strength is discussed by Nilson[2.28] who states that low-strength concrete (≈ 25 MPa) fails when sustained load is about 75 % of compressive strength; high-strength concrete can sustain stresses of 85 % of compressive strength indefinitely.

Strength gain with age is reported by Carrasquillo et al.[2.50] High-strength concretes exhibit the highest rate of strength gain at early ages followed by medium- and low-strength concrete.

Carrasquillo et al[2.51] report that the effect of specimen size on strength – the larger the specimen the lower the strength – is not affected by strength level.

Hester[2.52] investigated within-batch variation of concrete strength and states that standard deviation should be the same for all strength levels.

Carasquillo et al[2.53] investigated the effects of drying conditions on the strength of concretes. They found that when concretes were moist cured for only a short time and then allowed to dry out, higher-strength concrete showed the larger reduction in strength relative to concrete which had been moist cured the entire time.

Haug and Jakcobsen[2.54] tested strength of in situ concrete in offshore platforms and found that slipformed concrete was superior to non-slipformed concrete.

Studies of tensile strength of high-strength concrete is reported by various authors.[2.55, 2.57, 2.59] Formulae for predicting modulus of rupture and tensile-splitting strength of high-strength concrete are proposed.

Comments on the above points taken from the literature are as follows:

- The recommendations for the selection of aggregates are not necessarily of universal relevance. There can be little doubt that aggregates should have the lowest possible water requirement. But references 2.17 and 2.18 are based on North American experience; in reference 2.17 the highest concrete strength is only 50 MPa. An optimum coarse aggregate size of 10 to 12 mm is possibly not valid for crushed South African aggregates and higher concrete strengths. The best aggregate types can be found only by testing specific materials and comparing cement efficiency factors.

- Whether the substitution of materials such as ground granulated blastfurnace slag and fly ash will permit the achievement of acceptable strengths and strength efficiencies with South African materials can be found only by testing.

- The mode of failure of high-strength concrete as described by Carrasquillo et al[2.48] is probably of universal relevance.

- Rate of strength gain (reference 2.50) may be specific to cement type and the values given in the reference are possibly not applicable to local materials.

- Hester's work[2.52] on within-batch variation is of doubtful value because of the small number of results analysed.

- The relationship between tensile strength and compressive strength as reported in references 2.55, 2.57 and 2.59 could be influenced by aggregate type and is therefore not necessarily universally valid.

The aims of this section of the work are to:

- Investigate the relative importance of aggregate and hardened cement paste with regard to strength.

- Determine optimum size of coarse aggregate for high–strength concrete made with South African materials.

- Investigate the strength efficiencies of cement extenders.

- Compare strength efficiencies of various South African portland cements.

- Compare the performance in concrete of various South African aggregate types and relate performance to aggregate properties.

- Investigate the tensile strength of high–strength concrete relative to compressive strength and examine tensile failure surfaces.

3.5 Movement properties

Like strength, the movement properties of concrete are fundamental in determining the behaviour of concrete in structures. The engineer needs a knowledge and understanding of movement properties to be able to design a concrete structure and to detail the connections between the structure and other materials.

References in the literature are concerned mainly with comparing movement properties of high–strength concrete with those of conventional concrete.

Nilson[2.60] reports on a study of stress–strain curve of concretes of various strengths.

Modulus of elasticity has been studied by various authors.[2.61, 2.63, 2.67, 2.69] The consensus is that, because modulus of elasticity is influenced by the stiffness of coarse aggregate and mortar while strength depends mainly on the strength of the mortar, an increase in strength will not result in a comparable increase in concrete stiffness. An equation for the prediction of modulus of elasticity of high–strength concrete is proposed.[2.67, 2.69]

Creep and shrinkage have been studied by various authors.[2.70, 2.71, 2.72, 2.74, 2.75, 2.76] Their work indicates that creep coefficient and specific creep are smaller for concretes with

higher compressive strength. Shrinkages of high–strength concretes appear to be similar or slightly greater than those of conventional concrete.

In common with most of the references available in the literature, the above conclusions are based on work done in North America and Europe. It is desirable to study the behaviour of concrete made with South African materials and if possible, gain an understanding of why the concrete behaves in the way it does.

The aims of this part of the work are to:

- Study the effect of significant amounts of unhydrated cement on the movement properties of concrete.

- Measure modulus of elasticity of concretes made with local cements and aggregates and compare modulus of elasticity with strength.

- Determine creep and shrinkage characteristics of concretes made with local cements and aggregates.

3.6 Durability

In very general terms, the strength of concrete is an indication of durability. High–strength concrete has been developed primarily because of the advantages that result from superior strength; any effect on durability is mostly incidental.

The literature of high–strength concrete contains relatively little material on the subject of durability – possibly because of a perception that high–strength automatically ensures good durability. The fact that high–strength concrete is a relatively new material means that records of the performance of concrete in service are usually too short to show significant trends.

Work by Aitcin and others[2.42, 2.43] on the microstructure of concrete indicates that high-strength concrete is highly impermeable and therefore potentially durable. Sarkar and Aitcin[2.43] found that high-strength concrete had negligible chloride ion permeability.

The durability of high-strength concrete under freezing and thawing conditions, and whether air entrainment is necessary, appear to be contentious points. Hammer and Sellevold,[2.93] for example, obtained conflicting results with two different test methods.

ACI Committee 363[2.102] states that inadequate information is available to clarify the need for air entrainment in concretes with low water:cement ratios and that further research is necessary.

The aim of this part of the work is to reach an understanding of the fundamental factors that determine the durability of concrete and to see whether this understanding can be confirmed by case studies.

3.7 Structural design

As explained in chapter 1, the aspect of structural design is not taken further than the literature survey.

3.8 Economic implications

The construction industry's outlook is pragmatic; the driving force behind the development of stronger concrete has been the desire to make money spent on construction go further. This development has been so rapid that the use of high-strength concrete has preceded full knowledge of the properties of the material.[2.76]

The literature has but a few examples[2.100, 2.101] of cost studies that compare high-strength concrete with conventional concrete.

The aim of this part of the work is to study the cost effectiveness of high-strength concrete made with South African materials.

4. SCHEDULE OF EXPERIMENTAL WORK

The following experimental work was carried out in the laboratory in pursuance of the aims set out in chapter 3. The number of tests given for each investigation (or demonstration) is the number carried out for the purpose of that investigation only, unless stated otherwise. Net numbers of tests are shown in Table 4.1

4.1 The effect of a superplasticizer on the workability of a rich cement–water paste was qualitatively demonstrated.

4.2 The effect of cement:water ratio on workability of concrete made without admixtures was investigated by making mixes using various stone sizes and cement:water ratios. Twenty four different mixes were used.

4.3 The effect of three types of superplasticizer on the water requirement of concrete at a cement:water ratio of 2,0 was investigated and compared with water requirement of concrete with no admixture. Four different mixes were used. (One of these mixes was also used in 4.2 above.)

4.4 The effect of two types of superplasticizer on water requirement of concrete at various cement:water ratios was investigated. Fourteen different mixes were used. (Three of these mixes were also used in 4.3 above.)

4.5 Basic parameters for a method of proportioning high–strength concrete were investigated by making concrete mixes with four different stone sizes and various cement:water ratios. Twelve different mixes were used.

4.6 The relationship between mortar hydraulic radius and mortar solid content was investigated by making mixes with the same stone size but

different admixture dosages. Four final mixes and an unrecorded number of preliminary mixes were used.

4.7 The effect of water content on compressive strength of concretes made to the same cement:water ratio was investigated by making three different mixes and crushing cubes at 7 and 28 days.

4.8 Strength development with age was investigated by making concrete mixes at five different cement:water ratios and crushing cubes at 7, 28 and 56 days. Five mixes were used.

4.9 Grading analyses and relative and bulk density determinations were carried out on eighteen samples of stone and nine samples of crusher sand.

4.10 Maximum bulk density of a blend of two different sizes of stone was determined by carrying out compacted bulk density tests at four different blend ratios.

4.11 Optimum stone size (for highest strength and cement efficiency) was determined by making mixes with four different stone sizes and the same sand and cement. Mixes were made at five different cement water ratios. Water requirement and compressive strength at 7 and 28 days were determined on a total of twenty different mixes.

4.12 The performances of blends of rapid-hardening portland cement and condensed silica fume, fly ash, and ground granulated blastfurnace slag were compared with the performance of the cement on its own. Concretes were made with five different cementitious blends and five different cement:water ratios: a total of twenty five mixes. Water requirement, and compressive strength at 7, 28 and 56 days, were determined.

4.13 The performances of six different South African rapid–hardening portland cements were compared. Concretes were made at five different cement:water ratios. A total of twenty five different mixes was used. (Five of these mixes were also used in 4.12 above.) Water requirement, and compressive strength at 7, 28 and 56 days, were determined.

4.14 The performances of six different aggregate types and the effect of washed and unwashed sand were compared. Concrete mixes were made with seven different aggregate/aggregate combinations and at five different cement:water ratios: a total of thirty five mixes. Water requirement, and strength at 7, 28 and 56 days, were determined.

4.15 Particle–size distributions of the fine material in seven different sands were determined using the hydrometer method.

4.16 Aggregate crushing values of twelve different samples of stone were determined.

4.17 Elastic modulus of two different rock types was determined by carrying out a total of thirteen tests.

4.18 Maximum bulk densities of blends of two stone sizes were determined by measuring compacted bulk density of a total of forty two different blends.

4.19 Compressive strength, splitting–tensile strength and flexural strength of concretes made with three different aggregate types and at various cement:water ratios were determined. Thirteen mixes were made.

4.20 The effect of specimen aspect ratio on the measured compressive strength of concrete and the influence of high strengths on this effect was investigated. In the first series, two concrete mixes at different strength

levels were made and twenty prisms of different lengths were cast from each mix and crushed. In the second series, nine mixes at different cement:water ratios were made and twelve cubes cast from each mix; cubes were crushed singly or in stacks of two.

4.21 Modulus of elasticity was investigated by testing prisms made from concretes made with different aggregate types and cement:water ratios. Eighteen mixes were made.

4.22 Modulus of elasticity and strength of mixes made with one aggregate type at different cement:water ratios were investigated. Cube strengths were determined at 28 days; moduli of elasticity were determined at 28 and 56 days. Ten mixes were made.

4.23 Modulus of elasticity of cement clinker was measured on a small prism cut from a lump of clinker.

4.24 Modulus of elasticity of hydrated cement paste was estimated by determining modulus of elasticity of prisms made from crushed clinker as aggregate, cement and water.

4.25 The effect of aggregate type on creep and shrinkage was investigated by casting six prisms from each of nine mixes (three different aggregates each at three cement:water ratios). Three prisms were used for creep tests and three for shrinkage.

4.26 The effect of cement:water ratio on creep and shrinkage was investigated by casting six prisms from each of ten mixes. Three prisms were used for creep tests and three for shrinkage.

In total, about 240 concrete mixes were made and 1 400 specimens were cast for the various investigations.

Table 4.1: Numbers of tests carried out

Investigation number	Aggregate						Mix [a]	Hardened concrete [b]					
	Grading analysis	Relative density	Bulk density	Particle size distribution of sand fines	Aggregate crushing value of stone	Elastic modulus of rock [c]		Compressive strength	Splitting-tensile strength	Flexural strength	Elastic modulus	Drying shrinkage	Creep
4.2							24						
4.3							3						
4.4							11						
4.5							12						
4.6							4 [h]						
4.7							3	6					
4.8							5	15					
4.9	27	27	27										
4.10			4										
4.11							20	40					
4.12							25	74					
4.13							20	59					
4.14							35	104					
4.15				7									
4.16													
4.17						13							
4.18		42											
4.19							13	13	13	13			

Table 4.1: Numbers of tests carried out

Investigation number	Aggregate						Mix [a]	Hardened concrete [b]					
	Grading analysis	Relative density	Bulk density	Particle size distribution of sand fines	Aggregate crushing value of stone	Elastic modulus of rock [c]		Compressive strength	Splitting-tensile strength	Flexural strength	Elastic modulus	Drying shrinkage	Creep
4.20							11	10 [d] + 9 + 9 [e]					
4.21							18				18		
4.22							10	10			20		
4.23						1 [f]							
4.24							1 [g]				1		
4.25							9					9	9
4.26							10					10	10

a. Make concrete mix including at least one slump test; assess workability.

b. 3 specimens tested unless stated otherwise.

c. Individual specimens.

d. 4 specimens tested.

e. 3 sets of 2 cubes stacked one on top of the other.

f. Cement clinker.

g. Mortar made with crushed clinker as aggregate; no slump test.

h. Plus an unrecorded number of preliminary mixes.

5. WORKABILITY OF FRESH CONCRETE

5.1 Introduction

Concrete is in the fresh state for (normally) no more than a few hours. This is a very short time compared with the expected service life of most concrete elements and structures but it is a critical period. While it is in the fresh state, the concrete must be thoroughly mixed, discharged from the mixer, transported, placed in a form or mould, compacted and possibly finished. The workability of fresh concrete determines how these procedures should be carried out and how successful they are in producing concrete that can develop the properties required in the hardened state.

Workability is therefore an important property and this chapter deals with the workability of high-strength concrete.

In the following, various terms related to workability are used. Because these terms are not necessarily used in an exact sense, what is meant by them is given as follows:

- **Workability**

  That property of fresh concrete which determines the ease and homogeneity with which it can be mixed, placed, compacted, and finished.[5.1]

- **Consistence**

  The relative mobility or ability of freshly mixed concrete to flow; usually measured by means of the slump test.[5.1]

- **Plasticity**

  That property of fresh concrete that permits it to undergo the process of moulding without rupture.[5.2]

- **Thixotropy**

  The property of a material that enables it to stiffen in a short period while at rest, but to acquire a lower viscosity when mechanically agitated, the process being reversible.[5.1]

- **Viscosity**

  Viscosity is the property of a material that resists change in the shape or arrangement of its elements during flow.[5.1]

In the investigations described below, workability is assessed by measuring the slump according to the standard method[5.3] and qualitatively assessing properties such as plasticity, thixotropy, and viscosity.

The consistences of different mixes may be compared directly or indirectly. The direct approach is to compare slump measurements. The indirect approach is to make the mixes to the same slump by adjusting water content; inherently more workable mixes require less water per unit volume than inherently less workable mixes. Such comparisons are meaningful only if the mixes have similar workability at a given consistence.

5.2 The effect of cement:water ratio on workability

Cement:water ratio (C/W) has a significant influence on the strength of concrete. And high cement:water ratios are associated with high-strength concrete.[2.18, 2.43] It is necessary therefore to consider the effect of cement:water ratio on workability.

First consider concretes with low to normal strength (ie "conventional" concrete) – say 15 to 45 MPa. Measurements of water requirements for a given slump of concretes made with various aggregates and to different cement:water ratios over a range of cement:water ratio from 1,3 to 2,1 (an increase of about 62 %) showed that:[5.14]

- The average difference between highest and lowest water requirement for a given concrete was only 9 ℓ or about 5 % of average water requirement.

- The cement:water ratio at which water requirement was lowest was different for different aggregates.

- The curve for water requirement versus cement:water ratio was specific to each combination of aggregates but the general shape was as shown in Figure 5.1.

These observations show that water requirement does not simply increase as cement:water ratio increases in the range used for conventional concrete. We may deduce that the workability of conventional concrete is dominated not by the cement–water paste but by the aggregates.

But Perrie[5.4] also shows that for cement:water ratios above about 2,3 all mixes show an increase in water requirement as cement:water ratio is increased. The rate of increase in water requirement also increases as cement:water ratio increases. Once again specific mixes give specific curves. Shapes of typical curves are shown in Figure 5.2. The workability of the cement paste therefore becomes increasingly important at high cement:water ratios in influencing the workability of concrete.

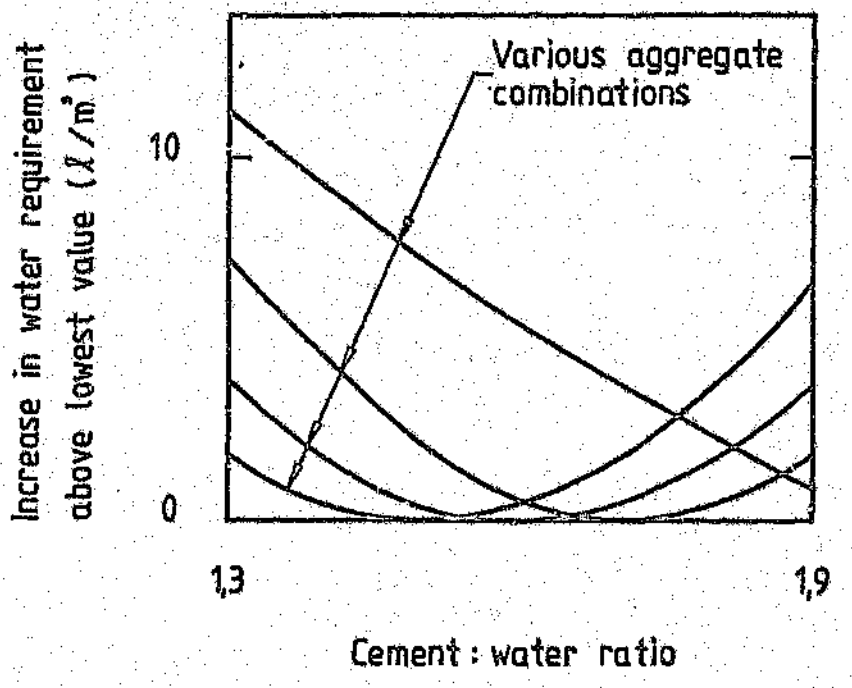


FIGURE 5.1: Increase in water requirement versus cement:water ratio for various aggregate combinations

The effect of cement:water ratio on workability of concretes with C/W ≥ 2 was investigated in the laboratory as follows:

Concrete mixes, without admixtures, were made using various stone sizes and cement:water ratios. Cements were rapid-hardening portland cement types "L" and "J". Aggregates were dolomite from Olifantsfontein. Stone content for each mix was determined according to a method which is described in section 6.4. Water requirement at a slump of 75 mm was determined for each mix.

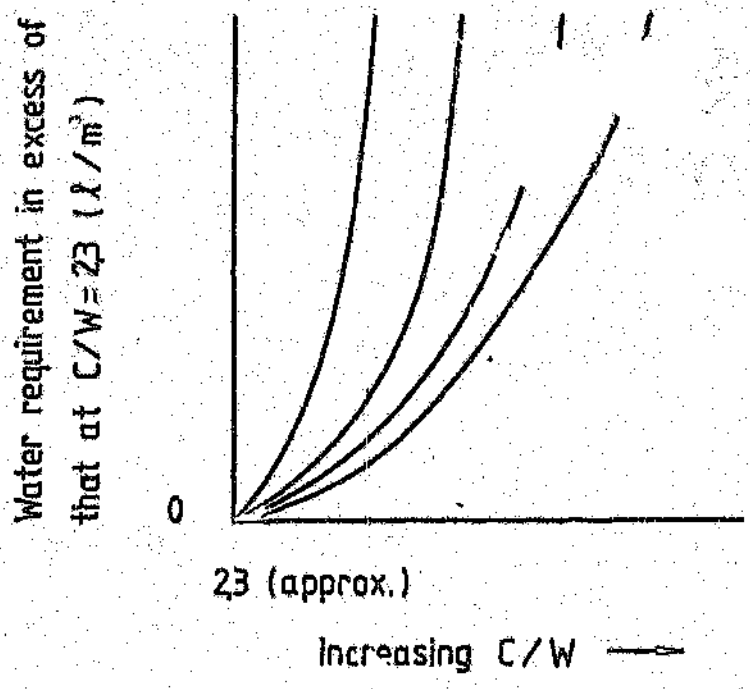


FIGURE 5.2: Shapes of typical curves for water requirement versus cement:water ratio for C/W more than about 2.3

Mix proportions are given in Appendix A5.1 and water requirements shown graphically in Figure 5.3.

The following trends emerged from the results:

- Water requirement generally increased as C/W was increased above a value of 2,0. The rate of increase increased with increasing C/W.

- Differences between mixes made with different stone sizes were greatest at C/W ≈ 2,0 and tended to become insignificant at C/W ≈ 3.

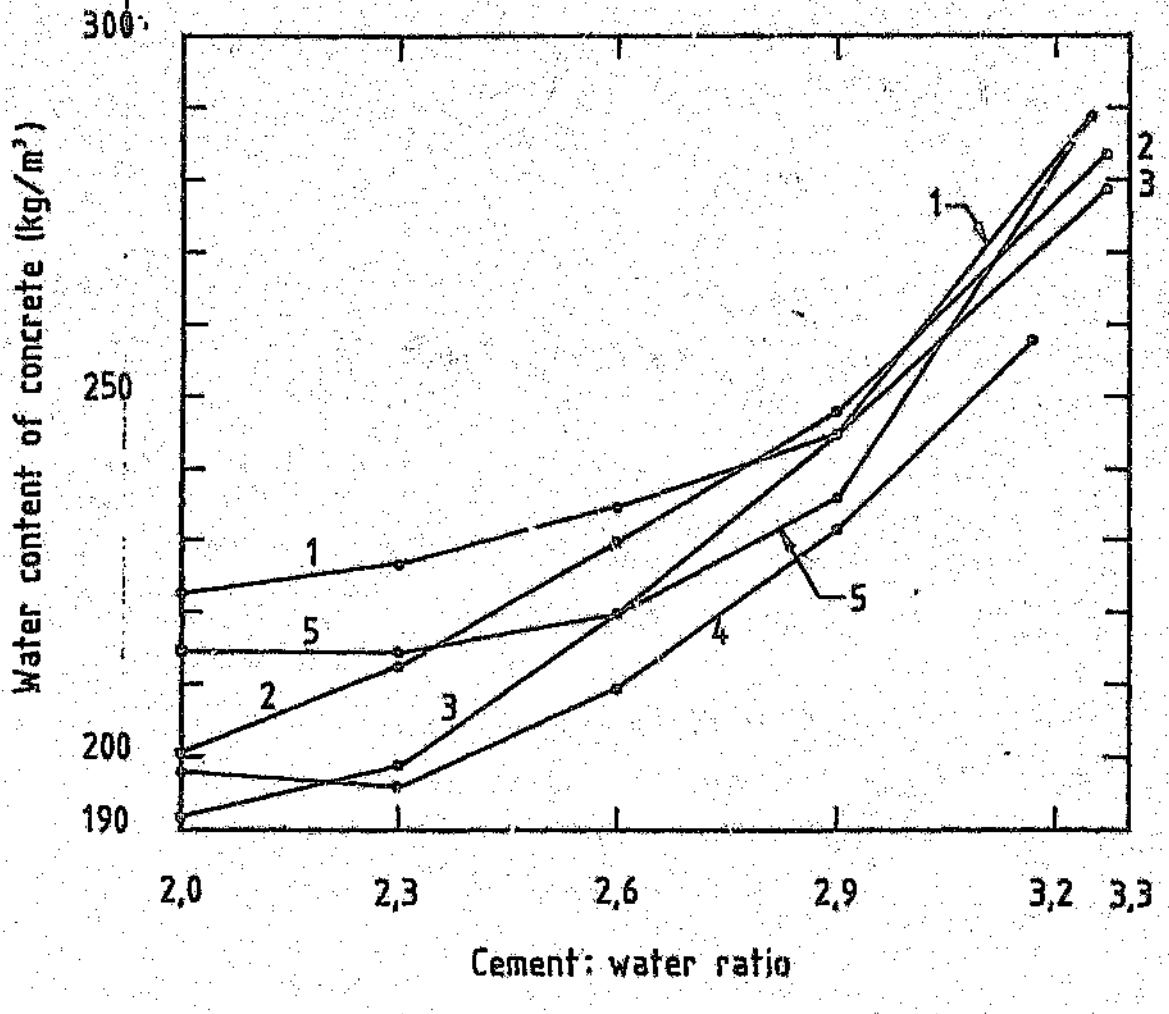


FIGURE 5.3: Water requirement versus cement:water ratio for various mixes made without admixtures; slump approximately 75 mm

Mix 1 13,2-mm stone; RHPC type L
Mix 2 19-mm stone; RHPC type L
Mix 3 26,5-mm stone; RHPC type L
Mix 4 75/25 blend of 26,5-mm and 6,7-mm stone; RHPC type L
Mix 5 13,2-mm stone; RHPC type J

- A value of C/W was reached for each mix where sand content became zero.

- Sand content for all stone sizes became zero at remarkably similar values of C/W. These values were close to 3,25. (If C/W were to be increased above 3,25, slump would decrease markedly. Slump would also decrease if some of the stone were to be replaced by the same mass of sand.)

- The water content, at between 280 and 290 ℓ/m^3, for the limiting upper value of C/W was also very similar for various mixes.

- Water requirement was influenced by cement type. Compare mixes 1 and 5.

It should be borne in mind that the above investigation was done on concrete made without admixtures. The conclusions reached do therefore not necessarily apply to concrete made with admixtures.

(The way in which sand is "squeezed out" of the mix by water, cement, and possibly stone is shown in Figure 5.4. A more detailed discussion is given in section 6.4.)

While making the above mixes, the following observations were made:

When the first tests were started, the standard mixing procedure, using a conventional pan mixer, in the Portland Cement Institute laboratory was:

mix for 2 minutes;
allow to stand in mixer for ½ minute;
mix for 1 minute.

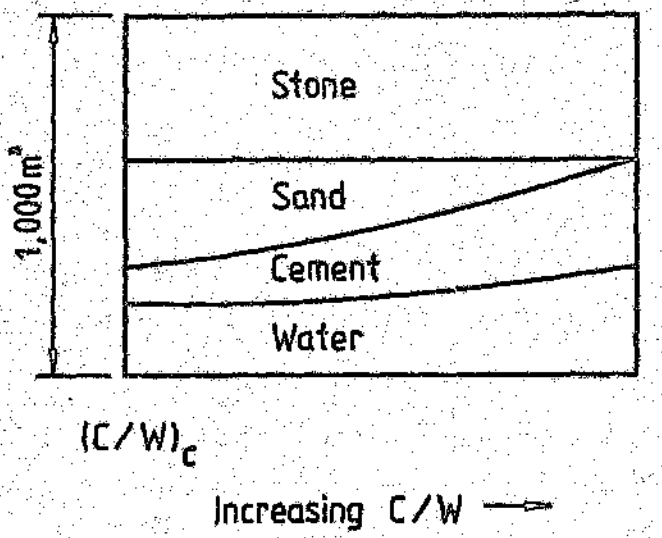


FIGURE 5.4: Solid volumes of phases of concrete versus cement:water ratio; $(C/W)_c$ is the cement:water ratio for "conventional" concrete

Although this procedure is adequate for conventional mixes, it became apparent that it was not satisfactory for very rich mixes. Observations of anomalous behaviour of the fresh concrete included:

- little or no slump response to the addition of an increment of dry aggregates to the mix;
- in some cases an increase in slump with the addition of dry aggregates;
- concrete made with given proportions showed a higher slump when these proportions were achieved by the incremental addition of aggregates than when mixed directly;
- tamping of the concrete in the slump mould failed to close holes made in the concrete by tamping the previous layers.

It was thought that this behaviour was caused possibly by what Powers terms "brief–mix–set"[5.5]* In an attempt to make the concrete more manageable, the mixing procedure was changed to:

mix for 1 minute;
wait for 3 minutes;
mix for 1 minute.

Although this effected an improvement on the leaner mixes, it was found that the richer mixes were still difficult to work with. Examples of slump behaviour using this regime were:

	MIX A	MIX B
Cement:water ratio	3,25	3,27
Stone size	13 mm	26,5 mm
Slump immediately after completion of mixing cycle (slump test takes approximately 2 minutes)	75 mm	60 mm
Slump after subsequent re–mixing*	90 mm	80 mm

* Slump appeared to stabilize after further remixing

Consequently water requirement for the target slump as calculated could be higher than actual.

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* A phenomenon attributed to the grains becoming stuck together by gel in the process of forming on grain surfaces. According to Powers, brief–mix–set occurs in conventional mixes if mixing is not continued for long enough; it may be prevented by mixing for more than about a minute. The occurrence of brief–mix–set after mixing for two minutes was probably a result of the very rich pastes being used: gel–forming was more intense and mixing was less effective because of the sticky nature of the concrete.

When it was realized that some of the slump readings were unreliable, it was decided to repeat all the work done up to that stage with a mixing procedure of:

mix 1 minute;
wait 3 minutes;
mix 1 minute;
wait 2 minutes;
mix 1 minute.

Although this procedure included the same total mixing time as the original procedure it made the mix more manageable, with the slump stable for a longer time. This was probably because of the longer waiting time between mixings which allowed initial gel growth to proceed further and made it possible to use mixing time more effectively.

Conclusions

As cement:water ratio is increased from a value of 2, water requirement increases. The rate of increase of water requirement increases with increasing cement:water ratio. At C/W = 3,2 the mix no longer has any space for sand and consists only of stone and cement paste; water requirement is extremely high.

Concretes made with high cement:water ratios and without superplasticizing admixtures are difficult to mix and handle.

This series of tests demonstrated that it is impracticable to make high-strength concrete without the inclusion of a superplasticizer – a conclusion confirmed by references 2.18, 2.23 and 2.29.

5.3 Performance of various superplasticizers

It was shown in section 5.2 that superplasticizers are an essential ingredient in high-strength concretes if they are to be mixed and placed by conventional means. In this section the performance of different types of superplasticizer is examined.

Superplasticizers may be grouped into four categories:[5,6]

- sulphonated melamine formaldehyde (SMF)
- sulphonated naphthalene formaldehyde (SNF)
- modified lignosulphonates
- others.

The so-called "modified lignosulphonate" category of superplasticizer are mixtures of lignosulphonate and either SMF or SNF. They are cheaper than, and inferior to, SMF and SNF.[5,7]

To investigate the performance of the first, second and fourth of the above categories, samples of three types of superplasticizer were obtained from local suppliers:

- **Sulphonated melamine formaldehyde** (SMF) type "M" available in powder or solution form. When used in a 20 % solution, the recommended dosage is 3 ℓ per 100 kg cement.

- **Sulphonated naphthalene formaldehyde** (SNF) type "C" available in powder or solution form. The solution is 46 % SNF. The recommended dosage of the solution is 2 ℓ per 100 kg cement.

- **Gluconate** (belonging to the "others" category in the above list) type "P" available in solution form. The recommended dosage is 0,3 ℓ per 100 kg of cement.

Because of its inferiority to SMF and SNF, the performance of the modified lignosulphonate category was not investigated.

5.3.1 Preliminary trial mixes

Concrete mixes were made using 13,2–mm stone and a rapid–hardening portland cement (RHPC) type "J" and including one of the admixtures. Cement:water ratio was 2,0. Slump at various water contents was measured in order to determine the water requirement for a slump of 75 mm. Results were as follows:

Admixture	Water requirement for 75–mm slump (ℓ/m^3)
No admixture	215
SMF "M"	175
SNF "C"	175
Gluconate "P"	205 (195 at double dose)

As the Gluconate was relatively ineffective (an ordinary water–reducing admixture is required, by specification, to reduce water requirement by 5 %) it was not used for further testing.

5.3.2 Effect of superplasticizers on water requirement at various values of cement:water ratio

Using 13,2–mm stone and the same RHPC as for the preliminary mixes described above, mixes with either SMF or SNF were made to determine the water requirement for a slump of approximately 75 mm. To do this, aggregates were added in increments equivalent to a water reduction of 10 ℓ/m^3 and the slump measured at each increment. Water requirement was then determined, using interpolation if necessary, to the nearest 5 ℓ/m^3. Both admixtures were in solution form and the water contained in the admixture (91 % and 70 % by volume in the 20 % SMF and SNF respectively) was taken into consideration in calculating total water content of the concrete.

Results were as follows:

Admixture	Water requirement (ℓ/m^3)				
	Cement:water ratio				
	2,0	2,4	2,8	3,2	3,6
No admixture	215	215	230	285	NP
SMF "M"	175	180	180	190	205
SNF "C"	175	165	155	155	150

NP : not possible

These results are shown graphically in Figure 5.5.

As can be seen, the SMF reduced water requirement by about 18 % in the range of C/W = 2,0 to 2,5. At higher values of C/W, water reduction increased with increasing C/W, and at C/W = 3,2 it was about 33 %. Nevertheless, water requirement of concrete made with SMF increased with increasing C/W.

SNF showed the same water reduction as SMF at C/W = 2,0 but with increasing C/W, SNF became increasingly more effective than SMF. Concrete made with SNF, and the specific cement, had the remarkable property of having water requirement reduce as C/W increased. It appears that the increase in the amount of admixture more than compensated for the increase in the quantity of cement as C/W increased.

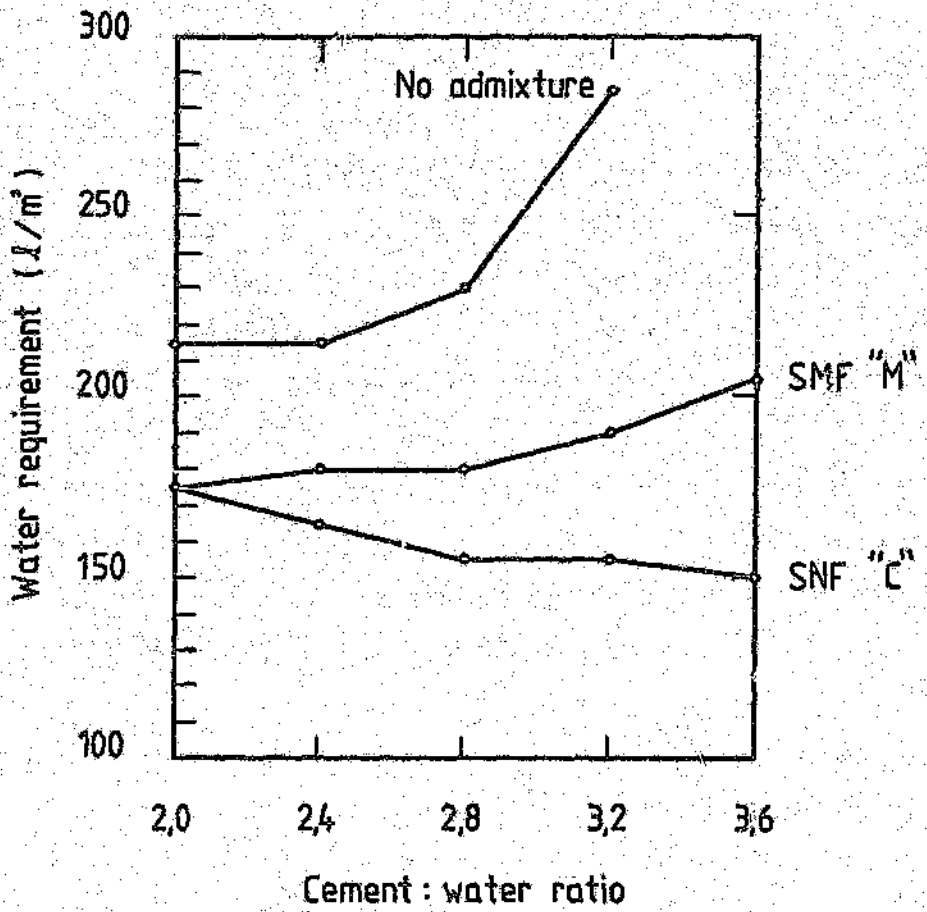


FIGURE 5.5: Water requirement of concrete versus cement:water ratio -- the effect of two types of superplasticizer

SMF: sulphonated melamine formaldehyde
SNF: sulphonated naphthalene formaldehyde

It should be noted that the above study compared the performance of commercially available superplasticizers dosed according to the manufacturer's recommendations.

El-Zanaty[2.22] states that when using the same dosage, based on the mass of cement, SNF superplasticizers are more effective than SMF superplasticizers. El-Zanaty[2.27] also recommends minimum dosages of superplasticizers (by mass of cement): 1 % for SNF and 2 % for SMF.

According to solution concentrations and dosages recommended by the manufacturers, the actual dosages (by mass of cement) used in the study were 0,92 % for SNF and 0,6 % for SMF. Both admixture dosages were therefore less than the minimum recommended by El-Zanaty: SNF marginally less and SMF significantly less. This difference could perhaps have contributed to the relatively poor performance of the SMF tested. The dispersing properties of SNF do however appear to be regarded as superior to those of SMF. SNF is, for example, widely used in the agricultural industry as a dispersant for wettable powders such as insecticides and herbicides.[5.7]**

5.3.3 Effect of superplasticizers on other properties of fresh concrete

A reduction in water content for a given slump is not the only effect a good superplasticizer has on the properties of fresh concrete. Three interrelated properties are affected: cohesiveness, viscosity and thixotropy.

In carrying out the above investigation, it was found that the admixture reduced cohesiveness. When doing the slump test, for example, superplasticized concrete with a "slump" of say 150 mm, assumed, on removal of the mould, its stable shape by the outward flow of individual, mortar-coated stones. This behaviour is very different from the slump of a conventional concrete which, if properly proportioned, settles as a cohesive, plastic mass.

Superplasticized concrete was found to be more viscous than conventional concrete: at a given slump more effort was required to mix or mould superplasticized concrete than conventional concrete without superplasticizer.

** The dramatic effect that SNF has on workability of cement paste can be demonstrated as follows:

Using a spatula, mix together in a bowl 1 kg of portland cement and 250 mℓ water. The resulting paste will be extremely stiff and difficult to mix. Now add 10 g of SNF powder and mix. The consistence changes to that of a pourable fluid and air bubbles in the paste move spontaneously upwards to break through the surface.

It was also noticed that the partially thixotropic nature typical of conventional concrete was absent in superplasticized concrete. Instead the concrete tended to behave more like a thick fluid. The fluidity of superplasticized concrete was evident in the way in which the "high-slump" mixes, if not agitated, flowed spontaneously into a more compact state if placed in a container. At the same time, air bubbles rose and escaped through the surface. At high local concentrations of stone, the mortar phase flowed downwards between the stones, presumably after they had attained a stable configuration. The lower levels of the concrete eventually reached a state of partial, if not complete, compaction.

To summarize, the properties of concrete made with an effective superplasticizer were found to be as follows:

- Cohesiveness was low.

- If kept in an agitated state the concrete behaved like a granular material in a viscous matrix.

- If left to stand, a degree of self-compaction occured. The degree depended on the fluidity of the mix.

These observations are confirmed by the findings summarized in the FIP/CEB State-of-the-art report on high-strength concrete:[5.8]

Workability obtained by the use of high dosages of plasticizers is not directly comparable with workability which results from increased water content. Plasticizers mainly reduce the yield stress in the fresh mix without seriously affecting the plastic viscosity. The result is a material with an energy consuming, "tough" behaviour. Helland,[5.9] reported that plastic viscosity of rich mixes could be reduced by entraining air in the mix. This would, however, reduce the strength of the hardened concrete.

Because the workability of superplasticized concrete with a high cement content is different from that of conventional concrete, the conventional means of handling fresh concrete may have to be modified for superplasticized concrete. Such modifications could include the more frequent agitation of the fresh concrete while being transported in order to prevent the formation of an unmanageable mass by the process of self compaction. ACI Committee 363[2.103] recommends that research should be done to establish whether the methods of compacting concrete should be modified for high-strength concrete.

In addition to the effect a superplasticizer has on rheological properties of fresh concrete, it was also noticed that the slump of superplasticized mixes was extremely sensitive to changes in water content. This means that in practice, water content would have to be very closely controlled if uniform consistence is to be maintained.

6. PROPORTIONING HIGH-STRENGTH CONCRETE MIXES

In the course of making laboratory mixes of high-strength concrete, it became clear that conventional methods of mix proportioning did not work for this type of concrete and that a method of proportioning specifically for high-strength concrete was needed.

This chapter includes: a summary of the method used widely in South Africa for proportioning conventional concrete; a description of its limitations for high-strength concrete; the development of a method for proportioning high-strength concrete; and a discussion on the hydraulic radius of cement paste.

6.1 The need for a method of mix proportioning

High-strength and conventional concrete can be proportioned by following a trial-and-error process. Such a process is usually time consuming and can result in the wastage of large quantities of materials and other resources. A method which allows the operator to arrive, first time or with the minimum of trials, at mix proportions which produce concrete that is satisfactory in the plastic and hardened states is therefore essential if proportioning is to be done quickly and efficiently.

6.2 Existing methods

References to existing methods of proportioning high-strength concrete are extremely sparse in the literature. Reference 6.1, for example, gives a table, taken from References 6.2 and 6.3, which shows the volume of coarse aggregate depending on the fineness of the sand and the size of the coarse aggregate. But this table fails to address factors such as very low water:cement ratios, which are relevant specifically to high-strength concretes. References 6.4 and 6.5 discuss proportioning but do not offer guidance on the stone content of mixes.

6.3 Requirements for a mix-proportioning method

Ideally, a mix-proportioning method should be:

- adaptable to a wide range of ingredient properties;
- based on easily determined properties of ingredients;
- easy to use and consist of as few simple steps as possible to obviate errors.

6.4 The development of a method of proportioning high-strength concrete

6.4.1 A starting point

The starting point for developing the mix-proportioning method described below was the Portland Cement Institute (PCI) method.[6.6] This method is an empirical development of the ACI method[6.7] published in 1954.

South Africa is poorly endowed with natural aggregates for concrete. Crushed rock is almost exclusively used for stone and for much of the sand; natural deposits are exploited for the remainder of the sand. Crushed-rock aggregates, which are made from many different rock types, differ very widely in their concrete-making properties. Natural sands, which include river, dune and pit sands, also differ widely in their properties.

In spite of this diversity of aggregates, the PCI method has been used successfully in all parts of South Africa for several decades for proportioning conventional concrete.

The PCI method consists essentially of the following steps.

1. Determine the relative density (RD) and fineness modulus (FM) of the sand.

2. Determine the RD and the nominal size of the stone (South African concrete stone is nominally single sized by specification).

3. Determine the bulk density of the stone in a compacted state (CBD).

4. Calculate the stone content of the concrete according to the formula:[6.8]

$St = CBD\ (K - 0{,}1\ FM_{sa})$ .....(6.1)

where St = stone content of concrete in kg/m^3

CBD = compacted bulk density of stone in kg/m^3

FM_{sa} = fineness modulus of sand

and K is a constant which depends on the stone size,[6.8, 6.9] as shown in the following table.*

Stone*	Nominal size (mm)	13,2	19	26,5
	Equivalent FM	6,35 – 6,75	6,75 – 7,15	7,45 – 8,0
K**		0,9	1,0	1,06

* Grading limits for nominal stone sizes are specified in Reference 6.10. The equivalent FMs are calculated from these limits but with an anomalous overlap eliminated from the limits for 19-mm stone.

** For a slump range of 25 to 100 mm, which makes the concrete suitable for compacting with moderate vibration. Values of K for higher slumps are given in Reference 6.8.

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* Equation 6.1 with values for K taken from the table is shown graphically in Figure 6.1. St/CBD vs FM of sand is a straight line for each stone size with K the intercept on the St/CBD axis. A serious flaw in the equation is apparent when St/CBD at very low values of FM is considered. For 19-mm stone, for example, St/CBD = 1,0 if FM is zero. This is clearly impossible if the mix is to be workable because relative movement of stone particles could occur only if the overall volume of the mix is distended. For 26,5-mm stone, St/CBD > 1,0 at FM = 0 which is impossible. Equation 6.1 therefore needs to be qualified by imposing a maximum value of St/CBD. This value, and whether it should be different for each stone size, is investigated later in this section.

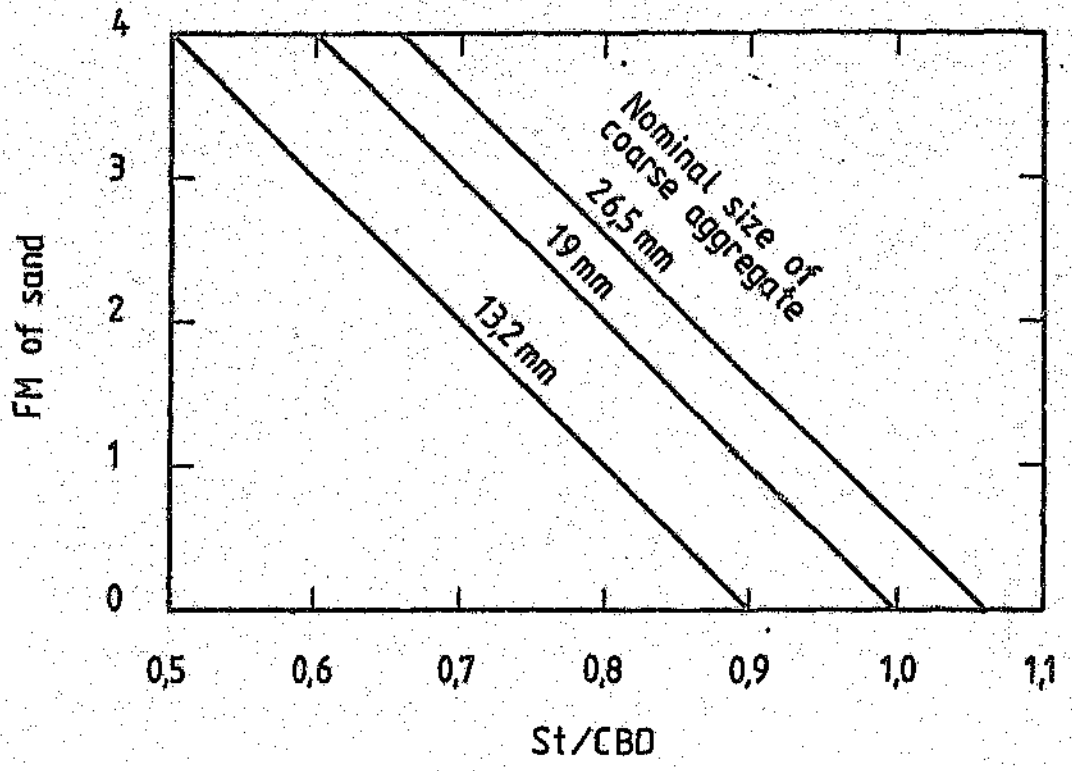


FIGURE 6.1 Stone content as a function of fineness modulus of sand and nominal size of coarse aggregate; from PCI design method, equation 6.1 with K as given in table

5. Choose a cement:water ratio (C/W) according to strength requirements and expected cement performance or durability requirements.

6. Estimate the water requirement of the mix, mainly by assessing the physical properties of the sand and taking the stone size into account.

7. Calculate the cement content of the mix, based on steps 5 and 6 above.

8. Calculate the absolute volumes of the stone, water and cement in 1 m^3 of the mix.

The sand content, in kg/m^3 of concrete,

$$= \left[1\ 000 - \left(\frac{St}{RD_{st}} + \frac{C}{RD_c} + \frac{W}{RD_w} \right) \right] \times RD_{sa}$$

where St, C and W are contents, in kg/m^3 of concrete, of stone, cement and water respectively; and RD_{st}, RD_c, RD_w and RD_{sa} are relative densities of stone, cement, water and sand respectively ($RD_w = 1,0$).

9. Make up a trial mix – usually a quantity of 10 or 20 ℓ is sufficient – using the above proportions and measure the slump and assess for stone content. If the slump is too high, adjustments can be made to the mix by adding pre-calculated increments of aggregate and mixing until a satisfactory slump is achieved. If, at or near the required slump, stone content is too high or too low then a new mix should be made with revised proportions.

10. Make compressive-strength specimens and test at suitable ages to verify the choice of C/W. Adjust C/W if necessary.

In practice, the calculations involved in the above steps are computerized in concrete laboratories and a trial mix can be made up quickly and easily. An acceptable stone content and slumps straddling the specified value are usually achieved with the first trial when the work is done by a competent operator.

The most important step in the PCI method is calculating the stone content (step 4 above). Stone content, and hence the ratio of stone to mortar in the mix, is a

fundamental factor which determines the workability of fresh concrete. Stone content, according to the PCI method, is a function of:

- stone size (nominal size or FM);
- sand FM;
- packing capacity of the stone (as represented by the compacted bulk density) which is influenced by grading and particle shape.

The method is successful because the above three factors very clearly characterize the aggregates and the way they relate to each other in a mix.

6.4.2. Limitations of the PCI method

The method was developed and has been used for conventional concrete:

- made with a C/W in the range of 1,2 to 2;

- having a water requirement, if 19-mm stone is used, of around 200 ℓ/m^3 (an average for concretes made with the, generally, rather poorly shaped South African aggregates).

The method does not make provision for the substantially higher C/Ws used for high-strength concretes or for the much lower water requirements of mixes made with modern high-performance water-reducing admixtures.

These limitations have been dealt with by isolating the effect of high cement contents from the effect of low water requirements, in the former case by considering results on unplasticized concretes, and in the latter case by using a superplasticizer. This is discussed below.

6.4.2.1 The effect of high cement contents (high C/W)

Abbreviations used in the following discussion are:

W, C, Sa and St are masses of water, cement, sand and stone respectively per cubic metre of concrete.

C/W is the ratio of cement to water by mass.

V_w, V_c, V_{sa} and V_{st} are solid volumes of water, cement, sand and stone respectively per cubic metre of concrete.

R_o is the cement:water ratio at which sand content is zero.

FM is fineness modulus; FM_{sa} and FM_{st} are fineness modulus of sand and stone respectively.

If C/W = 1,7 is taken as typical for conventional concrete, increasing C/W, while keeping water and stone contents constant and without using water-reducing admixtures, will result in increasing cement content and decreasing sand content. Figure 6.2 shows the solid volumes of the four phases of concrete: water, cement, sand and stone versus C/W ≥ 1,7 for constant water and stone contents.

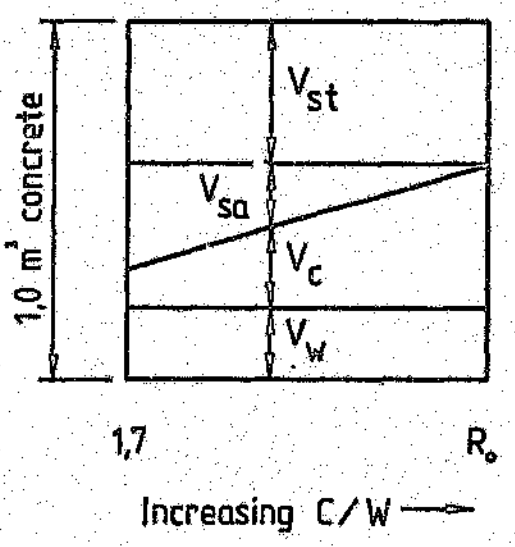


FIGURE 6.2: Solid volumes of phases of concrete versus cement:water ratio; constant stone content; constant water content

But, intuitively, V_w would not be expected to remain constant with increasing C/W. Figure 6.3 shows solid volumes versus C/W ≥ 1,7 for linearly increasing V_w but constant V_{st}.

Note that in Figure 6.3, V_c is a function of $(C/W)^2$, ie a second-order function. However, tests[6.11, 6.12, 6.13] have shown that V_w, for C/W > 1,7, is not a linear function of C/W but increases at an increasing rate as C/W is increased as shown in Figure 6.4. V_c is therefore a higher-than-second-order function of C/W.

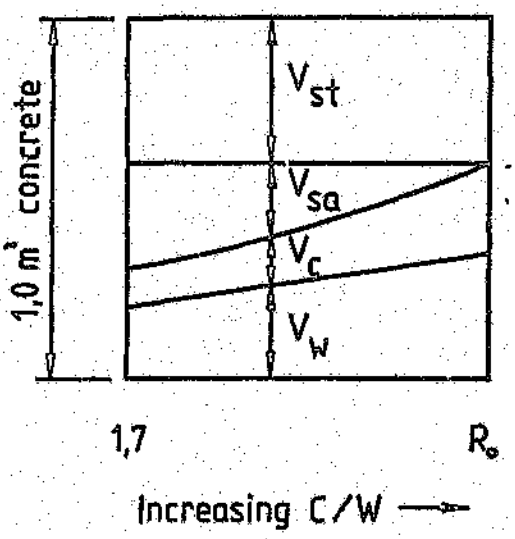


FIGURE 6.3: Solid volumes of phases of concrete versus cement:water ratio; constant stone content; water content increases linearly with increasing C/W

Up to this point, V_{st} has been kept constant. Whether this is justified is discussed below.

According to the PCI method, stone content, for good workability and for a given stone, is a function of the fineness of the sand. However, it is really the fineness of the mortar, or sand–cement combination, that is the factor which determines stone content, particularly for high cement contents. The use of sand FM as the determining factor in the method is a simplification which works within the range of conventional concrete mixes but not necessarily for high–strength mixes.

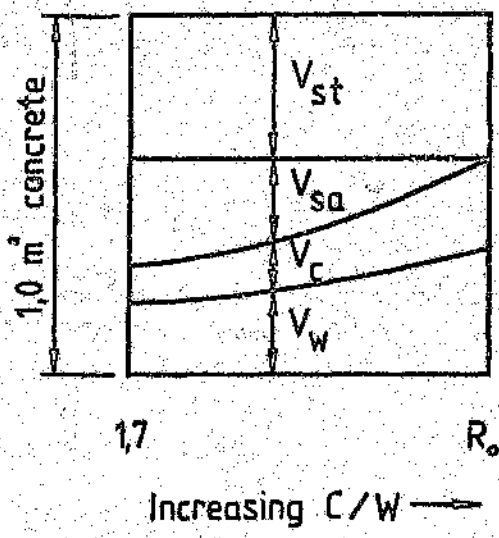


FIGURE 6.4: Solid volumes of phases of concrete versus cement:water ratio; constant stone content; water content increases at an increasing rate as C/W increases

Consider the effect of V_{sa}, varying as shown in Figure 6.3, on the fineness of the sand–cement combination. This effect is shown schematically in Figure 6.5. Note that the FM of cement is zero by definition.

At C/W = R_o :

V_{sa} = O and can therefore not influence V_{st} ;

V_{st} is a function of the size and packing capacity of the stone.

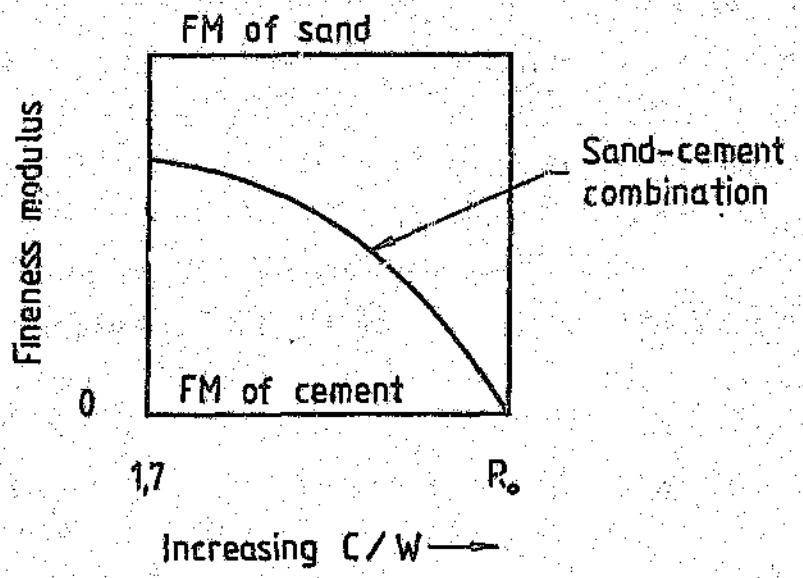


FIGURE 6.5: Fineness modulus of combination of sand and cement versus cement:water ratio; sand content is zero at C/W = R_o

The implication of the above is that V_{si} should be increased non-linearly with increasing C/W.

Developing a design method to deal with high values of C/W therefore involves determining:

R_o for each stone size;

optimum V_{si} at C/W = R_o;

a suitable function for increasing V_{si} as C/W is increased from 1,7 to R_o.

A series of laboratory trials was undertaken to investigate these factors.

Optimum stone content is the highest stone content which permits the achievement of good workability.

Individual stone particles in the mix must be able to rotate and move relative to each other without distending the overall volume of the concrete. This is possible only if the mortar coating around each stone particle is thick enough. On the other hand mortar

content must not be excessive because a concrete mix which contains an excessive amount of mortar tends to segregate while being compacted. After the concrete has been compacted the layer of mortar at the top surface should be no more than about 5 mm thick. (This thickness depends on the coarseness of the sand; the finer the sand the thinner the mortar layer should be.) Excessive mortar contents are also associated with increased water requirements which are detrimental to the dimensional stability of the hardened concrete and, by increasing the cement content, have an adverse effect on the cost of the mix.

Assessing the stone content of a trial mix is a qualitative process which relies on the skill of a trained and experienced concrete technologist. Some techniques used by staff of the Portland Cement Institute are:

- While the mix is in the mixer pan, lift the paddles clear of the concrete. Use a steel float to push the concrete into a compact mass against the side of the pan.

  Tamp and smooth the concrete with the float and observe how easily holes between the stones are filled with mortar. After the surface of the concrete has been compacted, scrape away excess mortar and observe the thickness of mortar covering the stone nearest to the surface. It should be fairly easy to compact the surface; the mortar layer formed on the surface should not be more than about 5 mm thick.

  Push the edge of the float into the concrete and assess the resistance offered by the stone particles. If stone content is not excessive, stone particles should move easily to either side as the edge of the float advances.

- Assess the resistance offered by the stone particles while rodding the concrete in the slump cone. If stone content is not excessive, stone particles should move away from the tip of the rod as it is pushed into the concrete. If stone

content is excessive, stone particles appear to grate against each other and obstruct penetration of the rod.

- Place sufficient concrete in a container to ensure that the depth of concrete will be at least four times the nominal stone size when the concrete is compacted. Compact the concrete by the means which are to be used – eg moderate vibration if the slump is about 75 mm. Smooth the top surface with a steel float. If stone content is optimum, no stones should protrude from the surface and the stone particles nearest to the top surface should be covered by mortar not more than 5 mm thick.

All of the above techniques were used in the investigation.

A target slump of 75 mm was imposesd on all mixes. This consistence is commonly used in South Africa for conventional mixes compacted by moderate vibration. By using a 75–mm slump it was intended to make the design method as relevant as possible to normal construction practice.

Stone sizes used were 13,2 mm, 19 mm, 26,5 mm and a blend of 26,5 mm and 6,7 mm. The first three sizes are those used for most reinforced–concrete structures in South Africa; they are nominally single–sized with grading limits prescribed by SABS 1083.[6.10] These sizes were used in order to make the method applicable to standard aggregate sizes. The blend of 26,5–mm and 6,7–mm stone was used to represent a graded stone.

It was found from the trials that:

R_o = 3,2 for concrete made without superplasticizer; R_o = 5,2 for concrete made with sulphonated naphthalene formaldehyde (SNF) superplasticizer; Stone content for optimum workability at R_o was 0,77 x CBD (0,77 x content in the compacted state).	for all the stone sizes and types used

Note that although it is impracticable to make high-strength concrete without a superplasticizer, the tests on unplasticized concrete made it possible to isolate the effects of high cement contents from those of low water requirements – the subject of the next part of this section.

A function of the form $y = x^2$ for the increase of V_{ss} with increasing C/W was found to give satisfactory results for unplasticized concrete. When this form of function was tried for superplasticized concrete (using sulphonated naphthalene formaldehyde superplasticizer), it was found that V_{ss} increased too rapidly initially as the C/W ratio was increased from 1,7. A function of the form $y = x^3$ was then tried and found to be satisfactory. These functions are shown graphically in Figure 6.6.

The information contained in Figure 6.6 can be used to deal with the effect of increasing C/W. However, calculating stone contents for various values of C/W based on a function of the type $y = x^3$ is tedious and time consuming. A chart to derive stone contents quickly and easily is described later in section 6.4.3.

6.4.2.2 The effect of low water requirements

The first mixes in the investigation which included superplasticizer had C/W values of 1,7 to 2,0 and were proportioned with stone contents according to the formula $St = CBD\ (K - 0{,}1\ FM_{ss})$ (equation 6.1). Rather surprisingly, it was found that they were noticeably too stony for placing in practice.

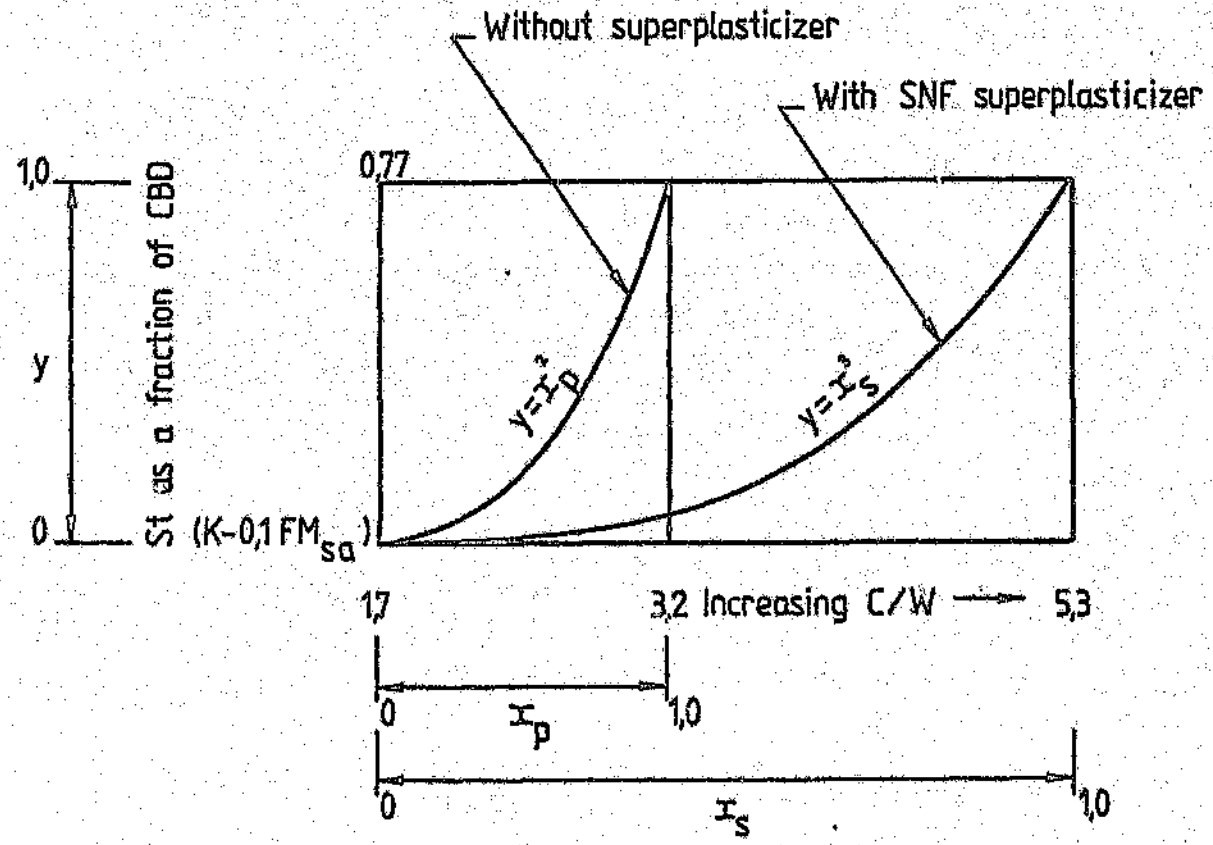


FIGURE 6.6: Stone content versus cement:water ratio for concretes made without and with a superplasticizer

The explanation for this phenomenon became clear when the composite FMs of the sand–cement mixtures were calculated for unplasticized and plasticized mixes at C/W = 2,0.

	Mix proportions (kg/m³)				FM		
Admixture	Water	Cement	Sand	Stone	Sand	Cement	Sand+ cement
None	215	430	914	950	3,08	0	2,17
Superplas-ticizer	175	350	1 100	950	3,08	0	2,40

The higher FM of the sand–cement combination in the mix with the superplasticizer is equivalent to increasing the sand FM from 3,08 to 3,4.[b]

While this calculation serves to explain why the stone should be reduced when the water content is drastically reduced, the correction required in the stone content is not easy to quantify.

Figure 6.7 shows three different mixes in terms of solid volumes. Mixes A and B depict the preliminary mixes without and with an admixture respectively shown in the above table. In Mix C, water and cement contents are the same as Mix B while stone content has been reduced by an arbitrary 10 %. The FM of the sand–cement combination for Mix C is 2,44 compared with 2,40 for Mix B. This means that in correcting the stone content, the factor requiring the correction is worsened instead of improved. Using the FM of the sand–cement combination is therefore not a logical basis for determining stone content.

Instead, an empirical approach was tried. Using a blend of 26,5–mm and 6,7–mm stone (with an FM equivalent to a 19–mm stone), mixes were made up using various admixture dosages which altered water requirement for the same slump. All mixes were made to C/W = 1,7 and stone content adjusted until it appeared to be satisfactory.

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[b] Calculated on the basis of weighted solid volume

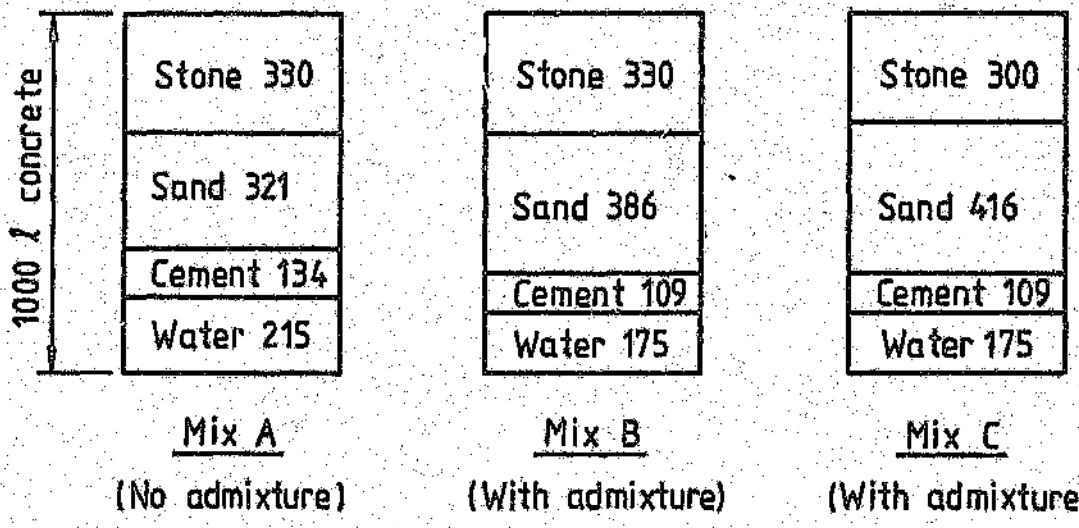


FIGURE 6.7: Solid volumes of phases of concrete in three different mixes

Analysis of the mix proportions was based on the hypothesis that for a given consistence, the optimum average hydraulic radius of mortar surrounding stone particles depends on the volumetric proportion of solids in the mortar.

$$\text{Hydraulic radius of mortar} = \frac{\text{volume of mortar}}{\text{surface area of stone}}$$

The surface area of the stone is unknown, but, for a given sample of stone, it is a function of stone volume. Hydraulic radius index (HRI_m) was therefore defined as follows:

$$HRI_m = \frac{\text{mortar volume}}{\text{stone volume}}$$

This index was used instead of hydraulic radius.

The proportion of solids in the mortar (PS_m)

$$= \frac{\text{solid volume of sand + solid volume of cement}}{\text{volume of mortar}}$$

HRI_m and PS_m were calculated for each mix and the results were as follows:

Water content (ℓ/m^3)	200	160	155	140
PSm	0,64	0,72	0,74	0,77
HRI_m	1,26	1,36	1,47	1,55

These results are shown plotted in Figure 6.8 and show a fairly good visual correlation. Although only four points were established, extensive subsequent testing validated the method based on these results.

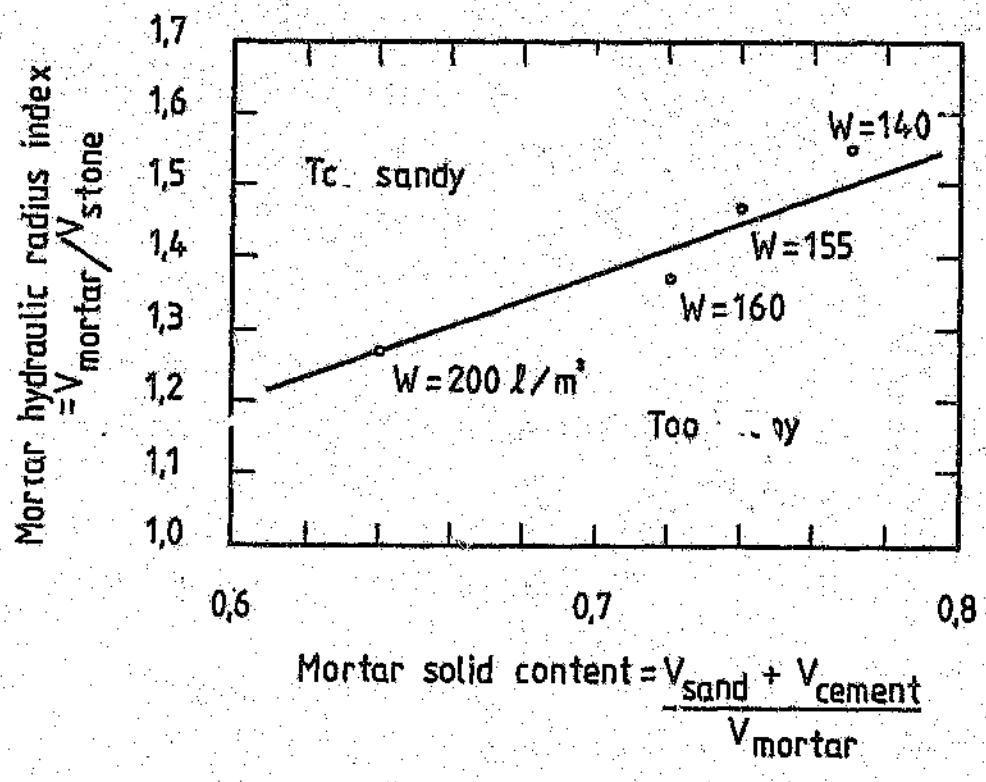


FIGURE 6.8: Mortar hydraulic radius index versus mortar solid content

A straight line, passing through the point referring to the mix with W = 200 ℓ/m^3, was drawn in by eye in Figure 6.8. (The use of a linear regression was considered not to be justified because the assessment of stone content in the mixes was by eye without any measurement.) This line represents optimum stone content; points falling below the line would indicate an excessively stony mix and vice versa.

The variables on both axes in Figure 6.8 are determined by interrelated factors. This figure can therefore not be used as a mix-proportioning chart. To develop such a chart, various stone contents were assumed and the water content corresponding to each derived from Figure 6.8. The steps in the calculation were:

1. Stone volume
2. Mortar volume = Total volume - Stone volume
3. $HRI_m = \frac{\text{Mortar volume}}{\text{Stone volume}}$
4. PS_m from graph
5. Sand volume + Cement volume = Mortar volume x PS_m
6. Water volume = Total volume - Stone volume
   - (Sand volume + Cement volume).

The calculation is, in effect, the derivation of a different constant K, in equation 6.1, for each water requirement. The result, in graphic form, is a series of parallel lines if FM_{sa} is plotted against stone content.

The mix-proportioning chart is shown in Figure 6.9.

6.4.3 Design charts

The design charts shown below were derived from Figure 6.6 (ie the function $y = x_s^5$) and Figure 6.9.

A family of curves is required for stone contents at C/W values between 1,7 and 5,2. Calculating and plotting such curves would be extremely tedious and an alternative

approach was therefore used: the scale of the x or C/W axis was distorted so that curves of the form $y = x^a$ would plot as straight lines as shown in Figure 6.10.

Design charts for concrete made to a slump of 75 mm with 13,2-mm, 19-mm and 26,5-mm stone are given in Figures 6.11, 6.12 and 6.13 respectively. These charts can also be used for graded or blended stone if its FM is matched to the equivalent stone size. The possibility of deriving charts for other slumps is discussed later.

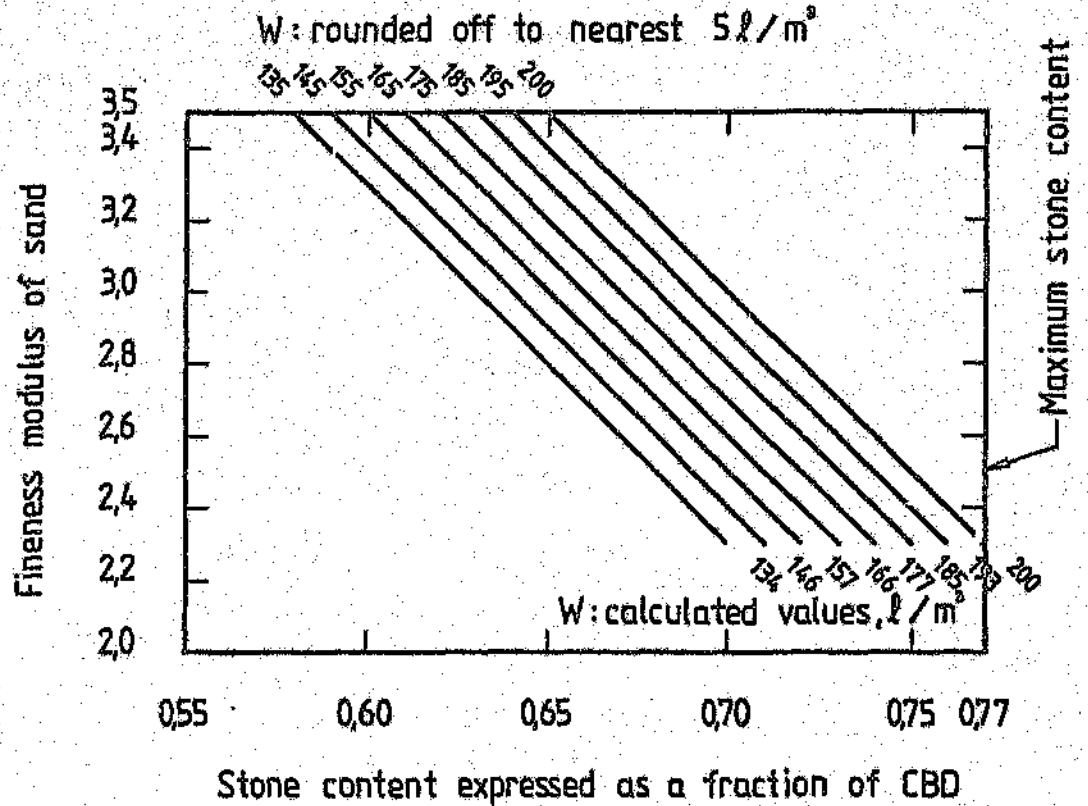


FIGURE 6.9: Stone content of concrete as a function of fineness modulus of sand and water content; nominal stone size 19 mm; cement:water ratio 1,7

6.4.4 Worked example

To illustrate the use of these charts, the following worked example is given:

Aggregate	Type	Relative density	Size and/or fineness	CBD (kg/m^3)
Stone	Andesite, crushed	2,93	75/25 blend of 25,5–mm and 6,7–mm stone FM of blend=7,1	1 750
Sand	Andesite, crushed	2,91	FM = 3,5	–

According to the FM of the stone blend, the chart given in Figure 6.12 should be used (the blend is equivalent to a single–sized 19–mm stone).

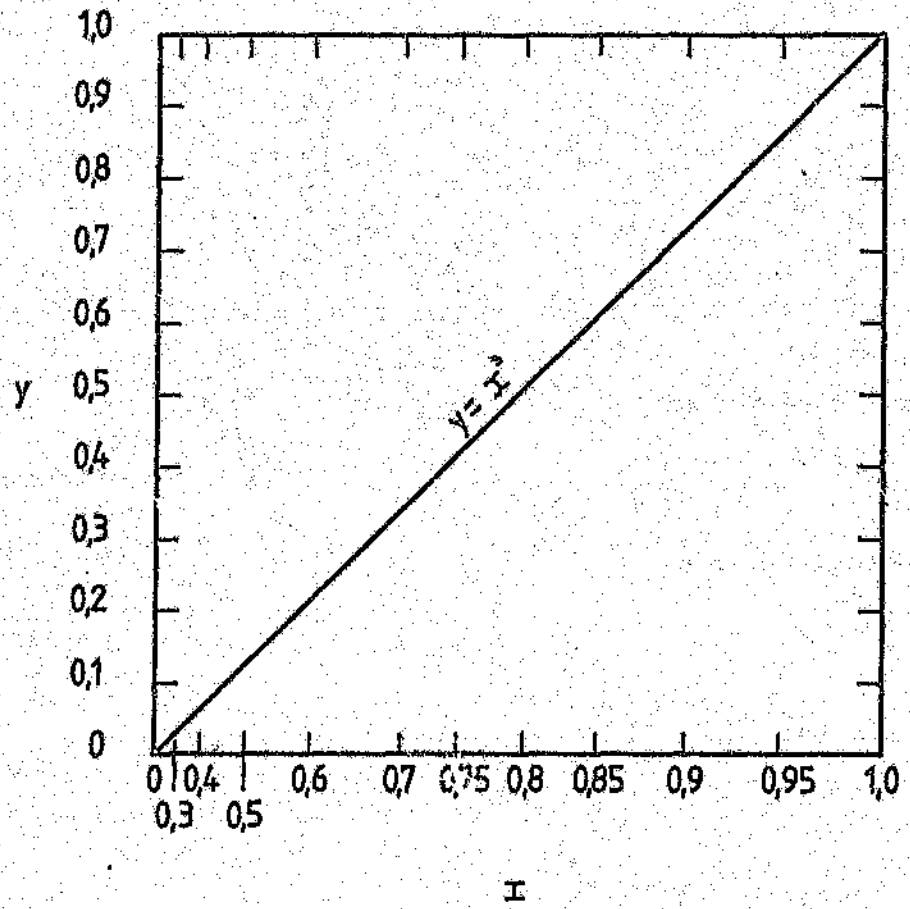


FIGURE 6.10: Distorted scale on x-axis to enable $y = x^3$ to be plotted as a straight line

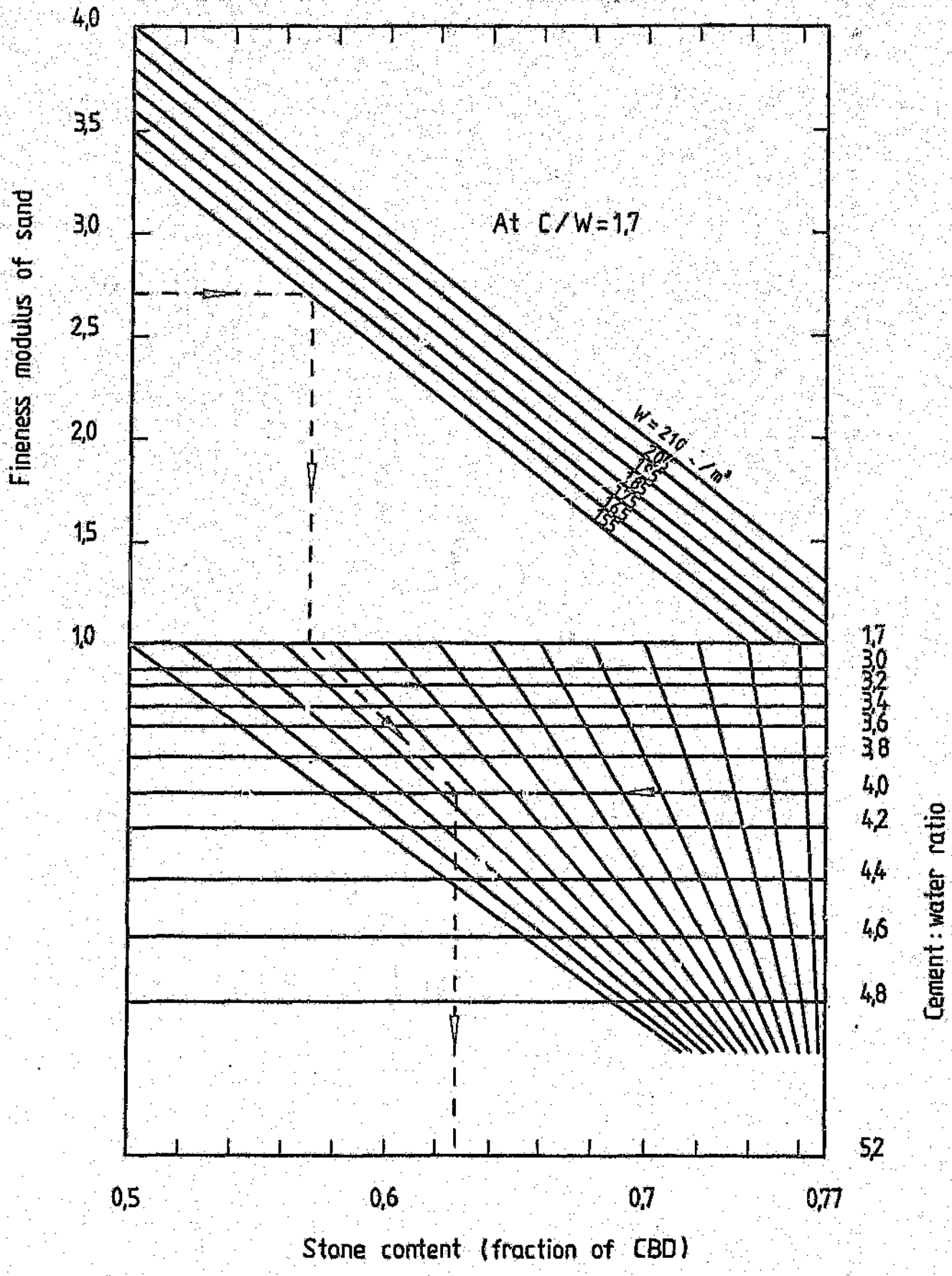


FIGURE 6.11: Chart for stone content; nominal stone size 13,2 mm (6,35 ≤ FM of stone < 6,75)

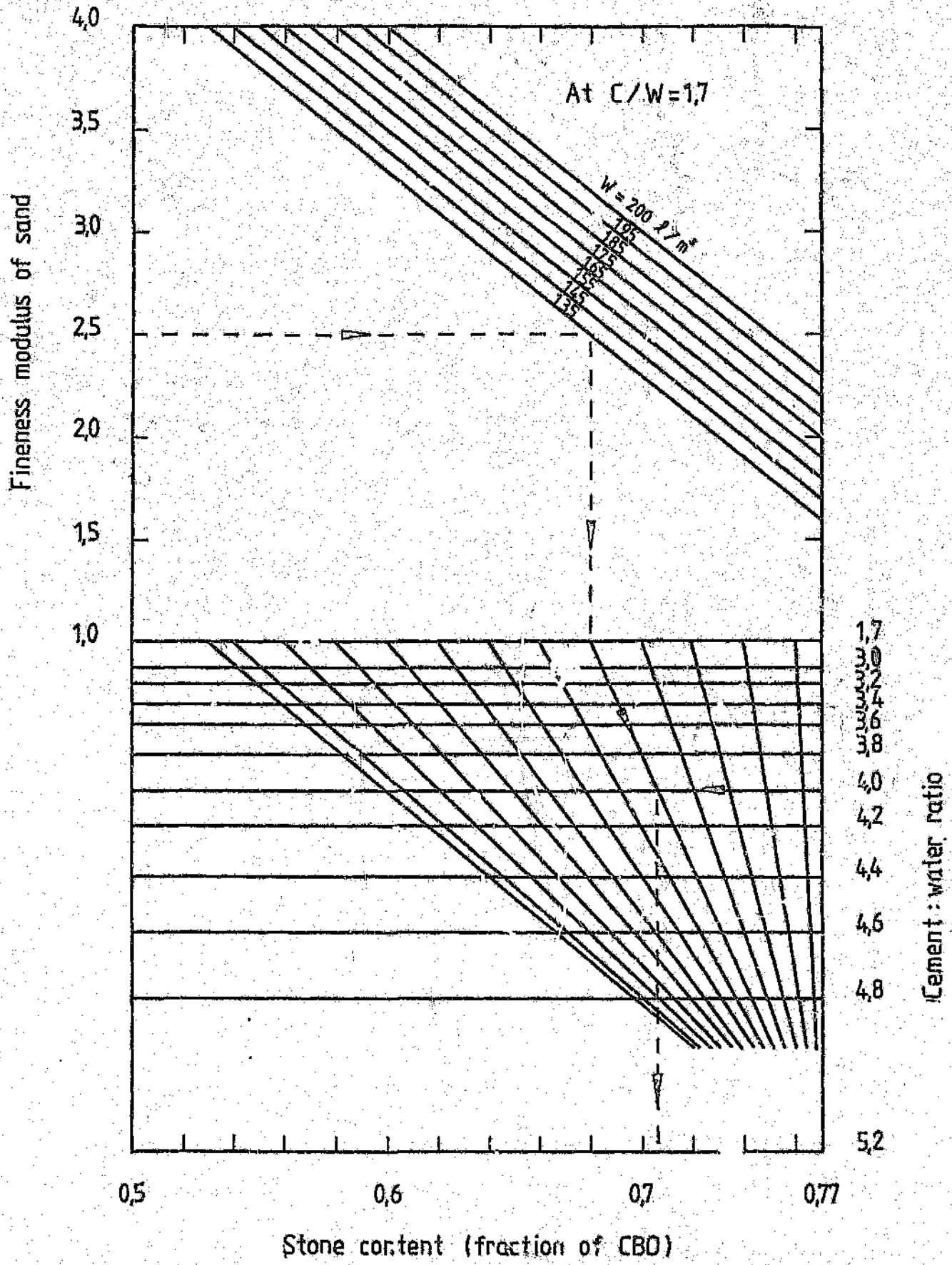


FIGURE 6.12: Chart for stone content; nominal stone size 19 mm (6,75 ≤ FM of stone ≤ 7,15)

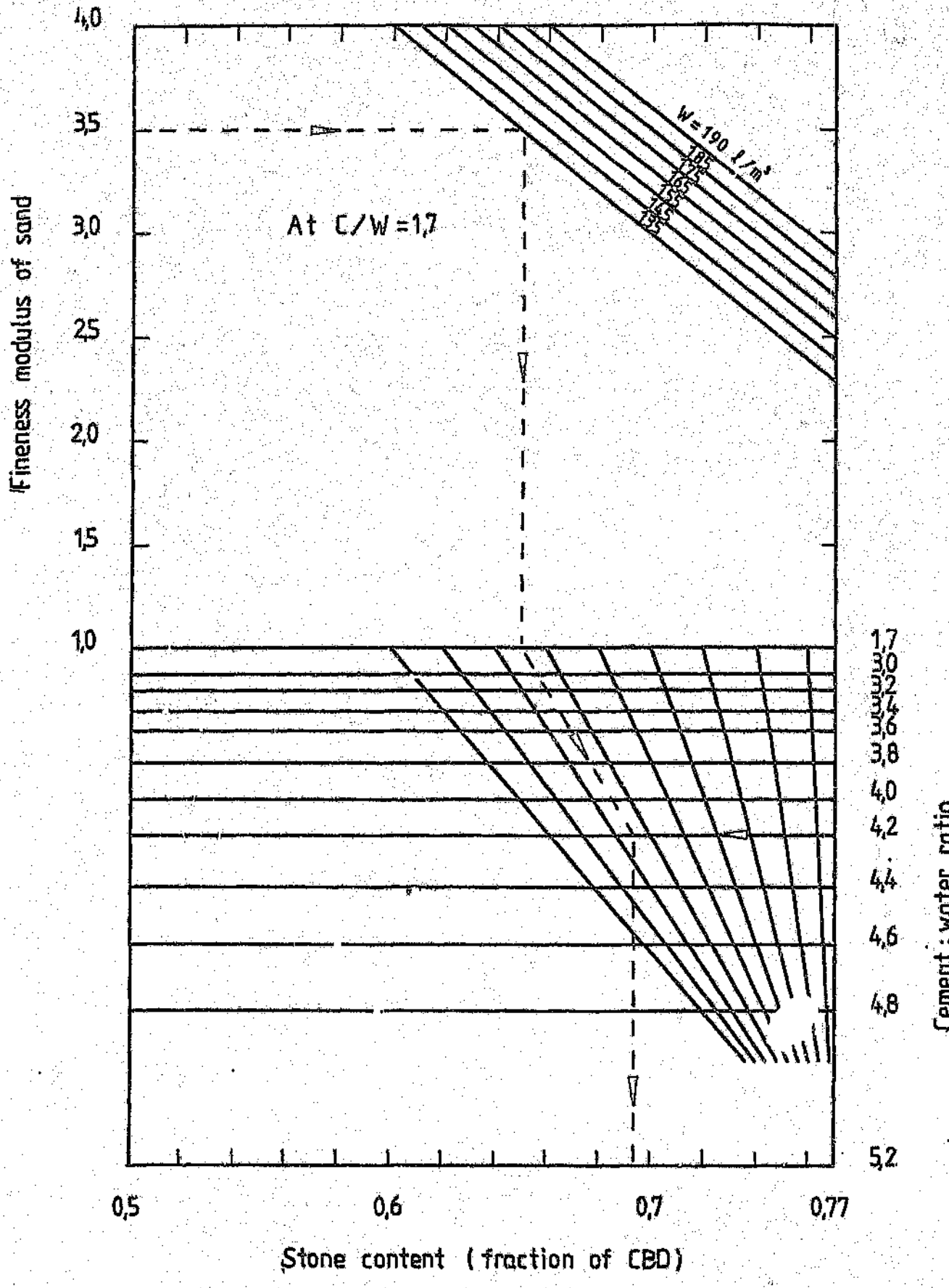


FIGURE 6.13: Chart for stone content; nominal stone size 26,5 mm (7,45 ≤ FM of stone ≤ 8,0)

Estimated water requirement at C/W = 1,7 is 175 ℓ/m³.

Enter the top part of the chart at sand FM = 3,5, move horizontally across until the diagonal line for W = 175 ℓ/m³ is intersected, move vertically down from the intersection point (stone content = 0,62 x CBD).

Make up a trial mix based on a stone content of 0,62 x 1 750 = 1 085 kg/m³ and use pre-calculated increments of aggregates to adjust W to values greater than, equal to and less than 175 ℓ/m³ and measure slump at each increment. Once W at C/W = 1,7 has been determined for a slump of 75 mm, the lower chart can be used to find stone contents for higher values of C/W.

For example (assuming W = 175 ℓ/m³):

C/W	2,2	2,5	2,8	3,3	4,4
Stone content (fraction of CBD)	0,63	0,64	0,66	0,68	0,71

Make up a series of mixes,° at suitable intervals of C/W, using aggregate increments to adjust W for a slump of 75 mm. Make sets of compressive-strength specimens from each mix and crush at the required ages to relate strength to C/W. Select the appropriate C/W for the specified strength.

6.4.5 Design charts for slumps other than 75 mm

Such charts could be designed by carrying out tests in a concrete laboratory and using the methodology outlined above. Laboratory work would be:

° This is in contrast to proportioning conventional mixes where a single value of C/W is usually sufficient. The reason for the difference is that in conventional concrete, strength is normally a predictable function of C/W; in high-strength concrete, strength is a function of C/W but is significantly influenced by other factors such as water requirement, total surface area of aggregate, aggregate strength and aggregate-surface characteristics

1. For concrete with a zero sand content, find the stone content for optimum workability (as a fraction of CBD) and C/W.

2. Derive the relation between HRI_m and PS_m at C/W = 1,7 similar to Figure 6.8.

Using the information obtained above, charts similar to Figures 6.11, 6.12 and 6.13 can be drawn.

6.4.6 Computer program

The information conveyed by Figures 6.11, 6.12 and 6.13 can also be given in the form of a computer program. A program, using LOTUS, was developed. For each stone size the upper half of the chart (C/W = 1,7) is replaced by a table which gives K for each water content. The program then calculates stone content for any required value of C/W by the formula:

$$\frac{St}{CBD} = (K - 0{,}1\ FM_{sa}) + \left[\ 0{,}77 - (K - 0{,}1\ FM_{sa})\ \right] \left[\frac{C/W - 1{,}7}{3{,}5}\right]^3$$

6.4.7 Conclusion

The mix-proportioning method discussed in this section has been extensively tested with aggregates manufactured from a variety of different rock types and having a wide range of characteristics such as average size or fineness, grading and particle shape and with a number of different portland cements and other cementitious materials.

Variations in the 165 mixes used to test the mix-proportioning method were:

- Stone sizes 13,2 mm, 19 mm, 26,5 mm and a blend of 26,5 mm and 6,7 mm.
- Two different blends of rapid-hardening portland cement (RHPC) and condensed silica fume.
- A blend of RHPC and ground granulated blastfurnace slag.

- A blend of RHPC and fly ash.
- Six different RHPCs.
- Six different aggregate types: andesite, dolerite, dolomite (two), granite, quartzite.
- Cement:water ratios ranging from 2,0 to 3,6.

Details of mixes are given in sections 8.3, 8.4, 8.5, 8.6 and 9.2.

The method in all cases resulted in concrete with an acceptable stone content which would facilitate placing and compacting in practice.

The method is based on easily determined aggregate properties: packing capacity and fineness modulus of stone, and fineness modulus of sand. It is therefore suitable for coarse and fine aggregates manufactured by the crushing of rock or broken down by natural processes and for single–sized or graded stone.

A paper,[6.14] based on sections 6.1 to 6.4 and co–authored by Alexander, was accepted for the second international symposium on high–strength concrete held in Berkeley, California in 1990.

6.5 The hydraulic radius of cement paste on aggregate surface

The mix–proportioning method developed in section 6.4 is based on the hypothesis that, for a given consistence and combination of materials, optimum hydraulic radius of mortar, which coats stone particles, depends on the volumetric content of solids in the mortar.

The relation between hydraulic radius of the paste, which coats all aggregate particles, and content of solids (ie cement) in the paste is investigated in this section. As will be apparent, the investigation is not in absolute but relative terms.

A series of mixes made with various sizes of andesite stone was analysed. (The mixes and aggregates are fully described in section 8.3.) To calculate relative surface areas of aggregates, a mathematical relation between apparent particle size and fineness modulus was derived as follows:

Fineness modulus does not indicate an exact average particle size, based as it is on steps, each of which may include particles of which the largest is virtually twice the size of the smallest. For example, the material passing the 600–µm sieve and being retained on the 300–µm sieve could include particles very slightly smaller than 600 µm, particles very slightly larger than 300 µm, and of course any size between these. It is also important to note that particle sizes are not directly measured by sieve analyses which categorize particles according to whether they pass through square openings of various sizes. This is why it is appropriate to refer to "apparent" particle size.

Fineness modulus is defined as a factor obtained by adding the cumulative percentages of material retained on each standard sieve from 150 µm and bigger and dividing the total by 100. For example if 50 % of the material is retained on the 150–µm sieve but passes the 300–µm sieve and 50 % is retained on the 300–µm sieve but passes the 600–µm sieve the FM = $\frac{150}{100}$ = 1,5.

In fact this calculation is the process of using moments to determine the centroid of the histogram showing proportions of material between successive sieve sizes. The position of the centroid is relative to a scale in which 0,5 corresponds to 150 µm, 1,5 to 300 µm, etc, as shown in Figure 6.14.

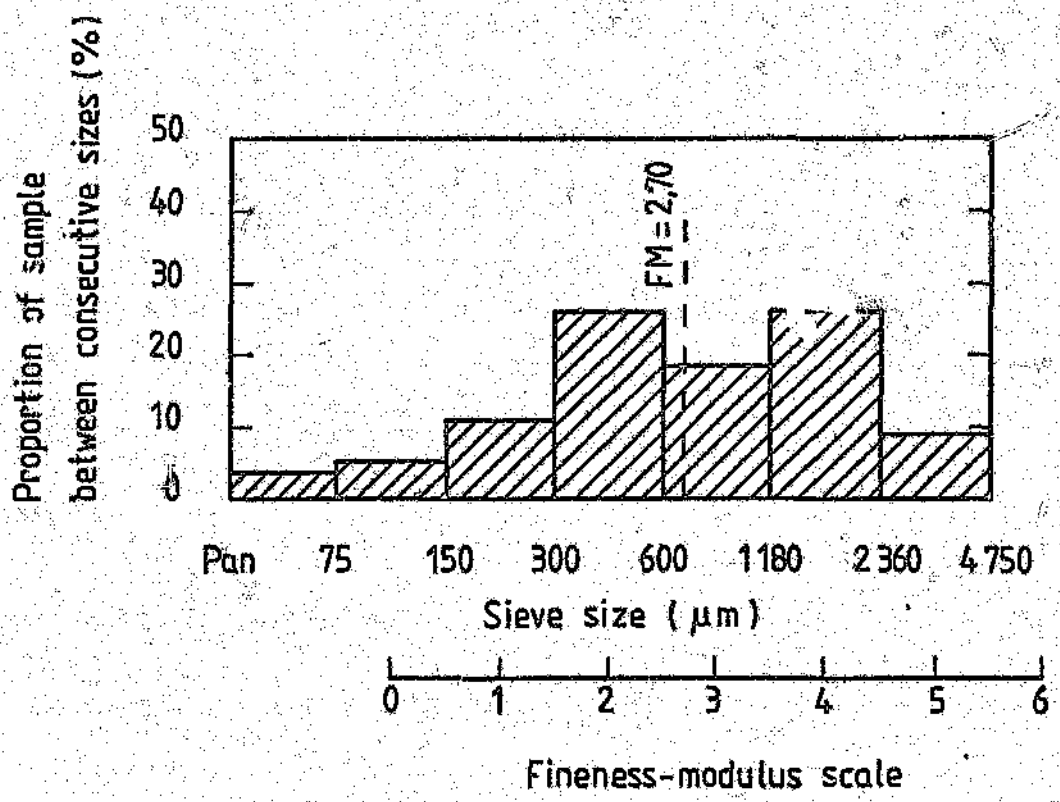


FIGURE 6.14: Distribution of particle sizes shown in histogram form according to sieve size and equivalent FM scale

Expressed mathematically, FM is a log function of apparent particle size.

$$FM = \log_2 \left[\frac{\text{Size}}{Y} \right]$$

where apparent size is in µm and Y is a constant dimension, also in µm.

If apparent size is assumed to be 150 µm when FM is 0,5

$$0,5 = \log_2 \left[\frac{150}{Y}\right]$$

$$\therefore Y = 106 \text{ µm}$$

$$\therefore FM = \log_2 \left[\frac{\text{Size}}{106}\right]$$

For example, if apparent particle size is 600 µm,

$$FM = \log_2 \left[\frac{600}{106}\right] = \log_2 (5,66) = 2,5$$

Or if FM is known, apparent particle size can be calculated:

$$\text{Size in µm} = 106 \times 2^{FM}$$

or $$\text{Size in mm} = 0,106 \times 2^{FM}$$

For the aggregates in question, FMs and calculated average apparent particles sizes were:

	FM	Average apparent particle size (mm)
Sand	3,53	1,22
13,2–mm stone	6,70	11,02
26,5–mm stone	7,82	23,95
19–mm stone	7,21	15,69
Blend of 26,5–mm and 6,7–mm stone	7,25	16,14

Assume the particles are perfect cubes. (As mentioned above, the comparisons are relative and not absolute.) The surface area of a cube 1 m in size is 6 m^2. The surface area of an aggregate of sixe X mm is therefore $\frac{1\ 000}{X}$ x 6 m^2/m^3 of solid volume. Surface areas for the aggregates calculated on this basis were:

	m^2/m^3	m^2/kg
Sand	4 918	1,690
13,2–mm stone	544	0,186
26,5–mm stone	251	0,086
19–mm stone	382	0,130
Blend of 26,5–mm and 6,7–mm stone	372	0,127

Aggregate surface areas are given in the following tables. (Hydraulic radius of paste is the ratio of paste volume to surface area of aggregate.)

Stone size 13,2 mm

	C/W	2,0	2,4	2,8	3,2	3,6
Surface area (m^2/m^3 of concrete)	Sand	2 028	1 928	1 803	1 697	1 638
	Stone	151	153	154	159	165
	Total	2 179	2 081	1 957	1 856	1 803
Paste volume per m^3 of concrete (ℓ)		309	324	347	360	361
Hydraulic radius of paste (µm)		142	156	177	194	200

Stone size 19 mm

	C/W	2,0	2,4	2,8	3,2	3,6
Surface area (m^2/m^3 of concrete)	Sand	1 965	1 849	1 725	1 611	1 445
	Stone	124	124	125	125	130
	Total	2 089	1 973	1 850	1 736	1 575
Paste volume per m^3 of concrete (ℓ)		276	298	319	340	361
Hydraulic radius of paste (µm)		132	151	172	196	229

Stone size 26,5 mm

	C/W	2,0	2,4	2,8	3,2	3,6
Surface area (m^2/m^3 of concrete)	Sand	1 820	1 793	1 683	1 575	1 511
	Stone	89	89	89	90	91
	Total	1 909	1 882	1 772	1 665	1 602
Paste volume per m^3 of concrete (ℓ)		276	280	300	320	329
Hydraulic radius of paste (µm)		145	149	169	192	205

80/20 blend of 26,5–mm and 6,7–mm stone

	C/W	2,0	2,4	2,8	3,2	3,6
Surface area (m^2/m^3 of concrete)	Sand	1 729	1 682	1 562	1 442	1 296
	Stone	135	137	138	139	142
	Total	1 864	1 819	1 700	1 581	1 538
Paste volume per m^3 of concrete (ℓ)		284	289	309	330	351
Hydraulic radius of paste (µm)		152	159	182	209	244

Total surface area of aggregates is plotted against C/W in Figure 6.15. Hydraulic radius of paste is plotted against volumetric fraction of solids in paste in Figure 6.16.

(Volumetric fraction of solids in paste depends only on the cement:water ratio and is not affected by the total volume of paste.)

It can be seen from the tables and figures that:

- Surface area of aggregates is dominated by the sand.
- Hydraulic radii of paste are very similar for all stone sizes.

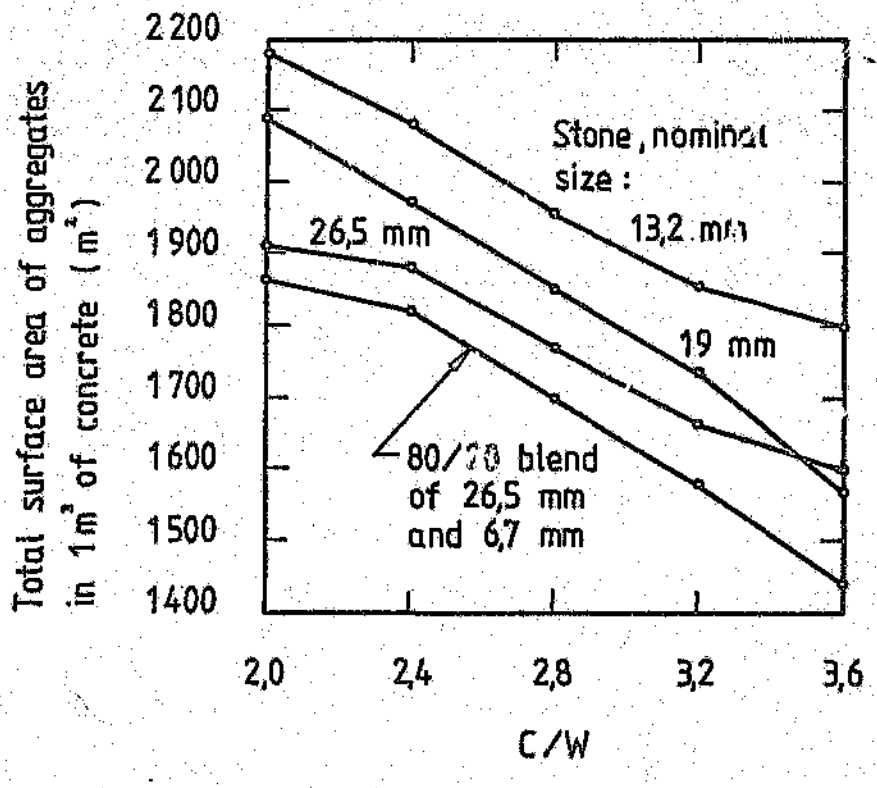


FIGURE 6.15: Total surface area of aggregates versus cement:water ratio for actual mixes; calculated on the basis of cubical particles

- The relation between paste hydraulic radius and volumetric proportion of solids is not linear. Lines joining points in Figure 6.16 generally become increasingly steeper as the proportion of solids increases.

- Hydraulic radius for the concrete made with blended stone is highest of the mixes for all values of C/W. This is because the superior packing capacity of the blended stone permits higher stone contents and therefore lower sand contents.

One way to consider fresh concrete is as a system of large particles coated with, and separated by, a semi-liquid material consisting of small particles suspended in water.

At one level of scale, stone particles are coated with mortar. At another level, stone and sand particles are coated with cement paste.

The mix-proportioning method described in section 6.4 is based on the assumption that optimum hydraulic radius of the mortar, which coats the stone particles, depends on volumetric content of solids in the mortar.

The above results demonstrate that hydraulic radius of the cement paste, which coats coarse and fine aggregate particles, is a function of solids content of the paste.

It appears therefore that hydraulic radius of coating and solids content of coating are related at both levels of scale of coating: mortar and cement paste.

Because of its relative nature, the investigation into cement paste as a coating serves only as an illustration and could not be used as a means of proportioning mixes. The workability of concrete is a complex result of many inter-related factors. To be manageable, any method of proportioning has to be a simplification and further refinement of the method described in section 6.4 would not be justified.

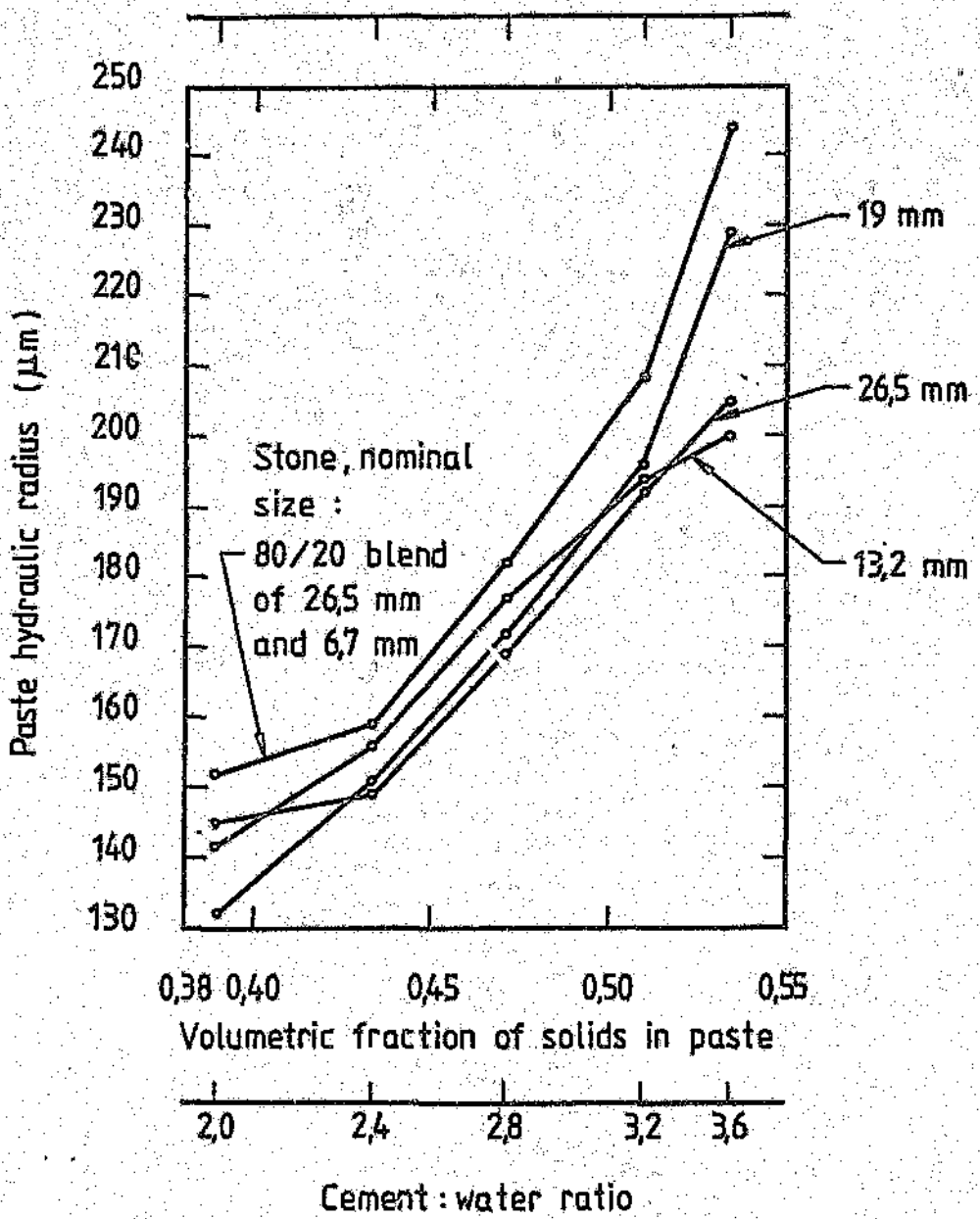


FIGURE 6.16: Paste hydraulic radius versus volumetric fraction of solids in paste for actual mixes; calculated on the basis of cubical particles

7. THE EFFECT OF HIGH CEMENT:WATER RATIOS ON THE HYDRATION OF CEMENT

The hydration of cement is subject to constraints determined by the chemical reactions involved, the relative volumes of participating substances and reaction products, and the availability of water.

Each of these factors is discussed below.

7.1 Chemical reactions

The following abbreviations are used:

C	=	CaO:	calcium oxide
S	=	SiO_2:	silicon dioxide
A	=	Al_2O_3:	alumina
F	=	Fe_2O_3:	ferric oxide
H	=	H_2O:	water
CH	=	$Ca(OH)_2$:	calcium hydroxide

Portland cement consists mainly of compounds:

C_3S:	tricalcium silicate
C_2S:	dicalcium silicate
C_3A:	tricalcium aluminate
C_4AF:	tetracalcium aluminoferrite

plus a small amount of gypsum, which serves to retard hydration, and small amounts of other compounds including alkalis.[7.1]

Reactions involved in the hydration of cement are:[7.1]

$$2\,C_3S + 6H \rightarrow C_3S_2H_3 + 3\,CH \quad \text{..... (7.1)}$$
$$2\,C_2S + 4H \rightarrow C_3S_2H_3 + CH \quad \text{..... (7.2)}$$
$$C_3A + CH + 12H \rightarrow C_4AH_{13} \quad \text{..... (7.3)}$$
$$C_4AF + 4CH + 22H \rightarrow C_4AH_{13} + C_4FH_{13} \quad \text{..... (7.4)}$$

Reactions 7.1 and 7.2 supply the CH used in reactions 7.3 and 7.4. Reaction 7.3 has intermediate stages involving calcium sulphate from the gypsum in the cement but these are not shown. (The reactions are taken from Reference 7.1. It should not be inferred that they are exact. In fact, the stoichiometry of cement hydration is not completely understood and there is not consensus in the literature. Nevertheless these reactions do serve to explain aspects of the hydration process. It must always be remembered however that the values calculated from these reactions may be approximate.)

By substituting atomic masses in reactions 7.1 to 7.4, the mass of water which combines chemically with unit mass of each of the four main cement compounds may be calculated.

Atomic masses, rounded off to whole numbers are:

Elements		Oxides		Compounds	
Ca	40	C	56	C_3S	228
Si	28	S	60	C_2S	172
O	16	A	102	C_3A	270
Al	27	F	160	C_4AF	486
Fe	56	H	18	CH	74
H	1			C_4AH_{13}	560
				$C_3S_2H_3$	342
				C_4FH_{13}	618

From reaction 7.1

$$\frac{\text{Mass of water}}{\text{Mass of } C_3S} = \frac{6 \times 18}{2 \times 228} = 0{,}24$$

In other words, the mass of water which combines with unit mass of C_3S is 0,24.

From reaction 7.2

$$\frac{\text{Mass of water}}{\text{Mass of } C_2S} = \frac{4 \times 18}{2 \times 172} = 0{,}21$$

From reaction 7.3

$$\frac{\text{Mass of water}}{\text{Mass of } C_3A} = \frac{12 \times 18}{270} = 0{,}80$$

From reaction 7.4

$$\frac{\text{Mass of water}}{\text{Mass of } C_4AF} = \frac{22 \times 18}{486} = 0{,}81$$

As stated above, portland cement consists of C_3S, C_2S, C_3A, C_4AF and other compounds. Let the mass of each of these compounds in unit mass of cement be $(C_3S)_u$, $(C_2S)_u$, $(C_3A)_u$, $C_4AF)_u$ and $Other_u$.

Then

$(C_3S)_u + (C_2S)_u + (C_3A)_u + (C_4AF)_u + Other_u = 1.$

Therefore, from the water:compound ratios calculated above, the mass of water W_u which combines with unit mass of portland cement is

$$W_u = 0{,}24 \times (C_3S)_u + 0{,}21 \times (C_2S)_u + 0{,}80 \times (C_3A)_u + 0{,}81 \times (C_4AF)_u \quad \ldots\ldots\ldots (7.5)$$

During 1988 a number of portland cement samples, with analyses, were obtained from various South African factories. Average percentage composition, calculated according to the Bogue formula,[7.2] was:

C_3S	58
C_2S	18
C_3A	6
C_4AF	8
Other	10
	100

Using these proportions in equation 7.5:

$$W_u = (0,24 \times 0,58) + (0,21 \times 0,18) + (0,80 \times 0,06) + (0,81 \times 0,08) = 0,29.$$

Czernin[7.1] states that W_u is generally in the region of 0,28, even of cements of quite considerable differences in chemical composition.

But this does not mean that a mix made of 100 mass units of cement and only 29 mass units of water will be able to react completely. The reason becomes apparent when we consider the volumes involved in hydration.

7.2 Volumetric aspects of hydration

References 7.3 and 7.4 give formulae for calculating the volumetric ratios of the hydration of cement at normal temperatures. These ratios are specific to each cement and are functions of compound composition and particle relative density. Formulae are as follows:

$$V_{hs} = (N + n)w_n$$

$$\frac{V_{hs}}{cV_c} = \frac{(N + n)w_n}{cV_a}$$

$$p_g = 3,6\ kw_n$$

$$N = 0,\overline{5}\ (1 + 4k)$$

$$k = 0,23 \times (C_3S) + 0,32 \times (C_2S) + 0,317 \times (C_3A) + 0,368 \times (C_4AF)$$

$$n = \frac{V_c}{(w_n/c)^\circ}$$

where V_{hc} = volume (in mℓ) of cement in hydrated state

N = volume (in mℓ) by which the solid phase (including gel pores) increases when w_n increases by 1 g

n = volume (in mℓ) of original cement that becomes hydrated when w_n increases by 1 g

w_n = nonevaporable water (in g)

(C_3S), (C_2S), (C_3A) and (C_4AF) are calculated proportions of respective compounds in the cement

$(w_n/c)^\circ$ = mass ratio of non–evaporable water to cement when all the cement is hydrated

c = mass of cement (in g)

V_c = specific volume of cement (in mℓ/g)

p_g = volume of gel pores (in mℓ)

$k = V_m/w_n$ where V_m = the number of grams of water required to cover the surface of the hydrated cement in the sample with a layer averaging one molecule deep.

Using the average composition of cement as given above, these formulae give (assuming V_c = 0,319 mℓ/g and specific volume of water = 1,0 mℓ/g):

$$k = 0{,}24$$

$$\therefore N = 0{,}75\ (1 + 0{,}96) = 1{,}47$$

$$(w_n/c)^\circ = 0{,}29 \text{ (from reactions 7.1 to 7.4)}$$

$$\therefore n = \frac{0{,}319}{0{,}29} = 1{,}1$$

7.2.1 Volume of hydrated cement

$$V_{hc} = (1{,}47 + 1{,}1)\ w_n$$
$$= 2{,}57\ w_n$$

$$\therefore \frac{V_{hc}}{cV_c} = \frac{2{,}57w_n}{0{,}319c} = 8{,}06\ \frac{w_n}{c}$$

At full hydration $w_n/c = (w_n/c)^\circ = 0{,}29$

$$\therefore \frac{V_{hc}}{cV_c} = 8{,}06 \times 0{,}29 = 2{,}34$$

In other words 1 volume of cement hydrates to a volume of 2,34. (Powers obtained a value of 2,22 for the specific cement considered in Reference 7.4.)

If unit volume of cement hydrates to a volume 2,34, unit volume of hydrate forms in the space originally occupied by the cement and 1,34 volume of hydrate forms in the space originally occupied by the water. The volume of water in this case is 1,34. Assuming a specific volume of cement of 0,319 mℓ/g and specific volume of water of 1,0, the mass ratio of cement to water (C/W) is

$$\frac{1}{0{,}319 \times 1{,}34} = 2{,}34$$

This ratio of C/W is such that the hydrate exactly fills the space available. Similarly, if unit volume of cement hydrates to a volume of 2,22 as found by Powers, C/W is 2,57.

These two values of C/W differ by about 10 %.

Because the aim of this discussion is to indicate merely the order of magnitude of the porosity of cement paste and because the basis of these calculations is so inexact, a simplified approach seems to be justified. Czernin[7.5] uses a single value of C/W = 2,5 for all portland cements for the condition in which hydrate exactly fills the space available (ie unit volume of cement hydrates to a volume of 2,254, say 2,25). This value of C/W is used for the rest of the work.

7.2.2 Porosity of hydrate

From the above formulae:

$$\frac{p_g}{V_{hg}} = \frac{3{,}6kw_n}{(N+n)w_n} = \frac{3{,}6 \times 0{,}24}{2{,}57} = 0{,}34$$

In other words the gel is 34 % gel pores and 66 % solids. (Powers found values of 0,33 and 0,67 respectively for the cement considered in reference 7.4.)

These concepts are shown graphically in Figures 7.1, 7.2 and 7.3. which are based on the following:

- Unit volu[illegible] cement hydrates to a volume of 2,25.
- The mass of water which combines chemically with unit mass of cement is 0,29.
- Gel is 34 % gel pores and 66 % gel solids.
- Specific volume of cement is 0,319 mℓ/g.
- Specific volume of water is 1,0 mℓ/g.

The term "cement–saturation" which is used in these figures, is explained below.

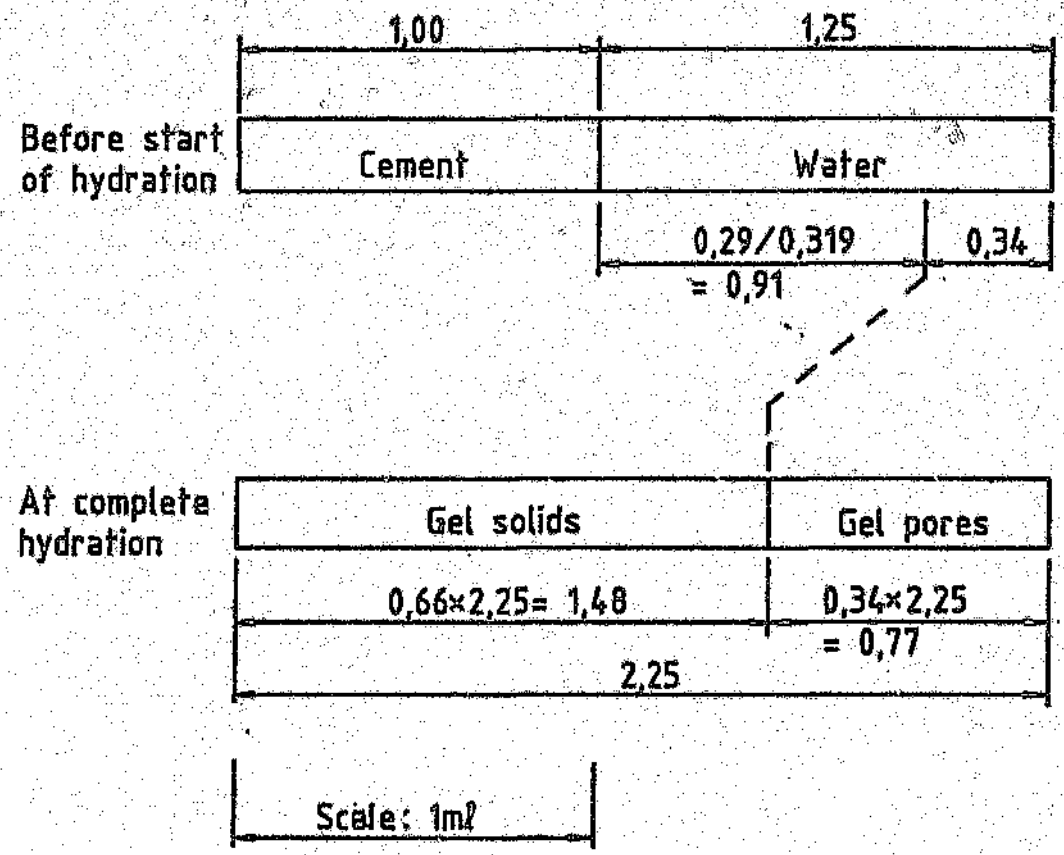


FIGURE 7.1: Absolute volumes of phases of cement paste for cement–saturated mix

7.2.3 The concept of cement–saturation

In Figure 7.1, the mix ratio is such that there is exactly the space required for the products of complete hydration of the cement. The gel:space ratio, which is the ratio of the volume of gel to the volume of the space in which it forms, is equal to 1,0. Cement:water ratio is 2,5. (Water:cement ratio is 0,40.)

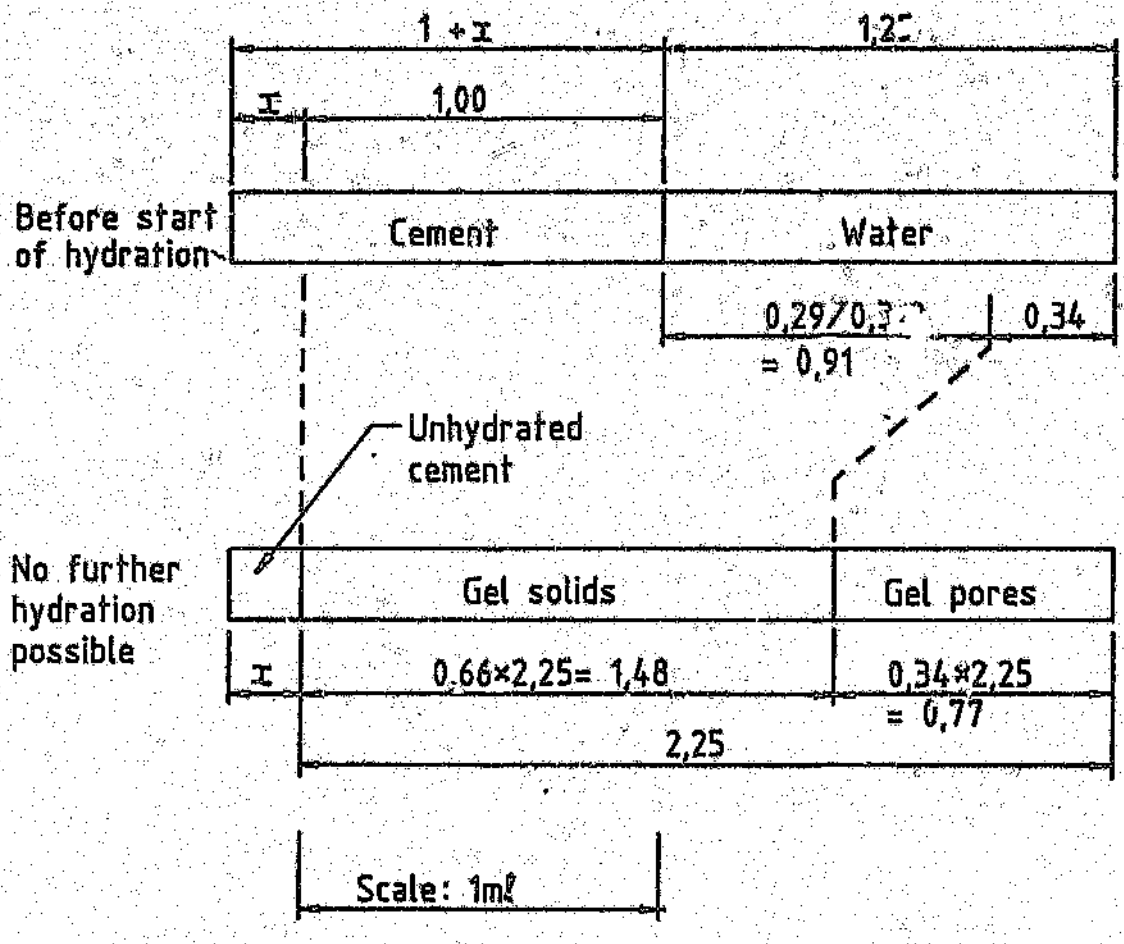


FIGURE 7.2: Absolute volumes of phases of cement paste for cement–oversaturated mix

The system, after hydration is complete, is exactly filled with hydrate or gel, and "cement–saturated" is proposed as a term to describe such a mix.

Figure 7.1 also shows that the total volume of cement plus water used for hydration is greater than the volume of gel solids produced:

$$1{,}0 + 0{,}91 > 1{,}48$$

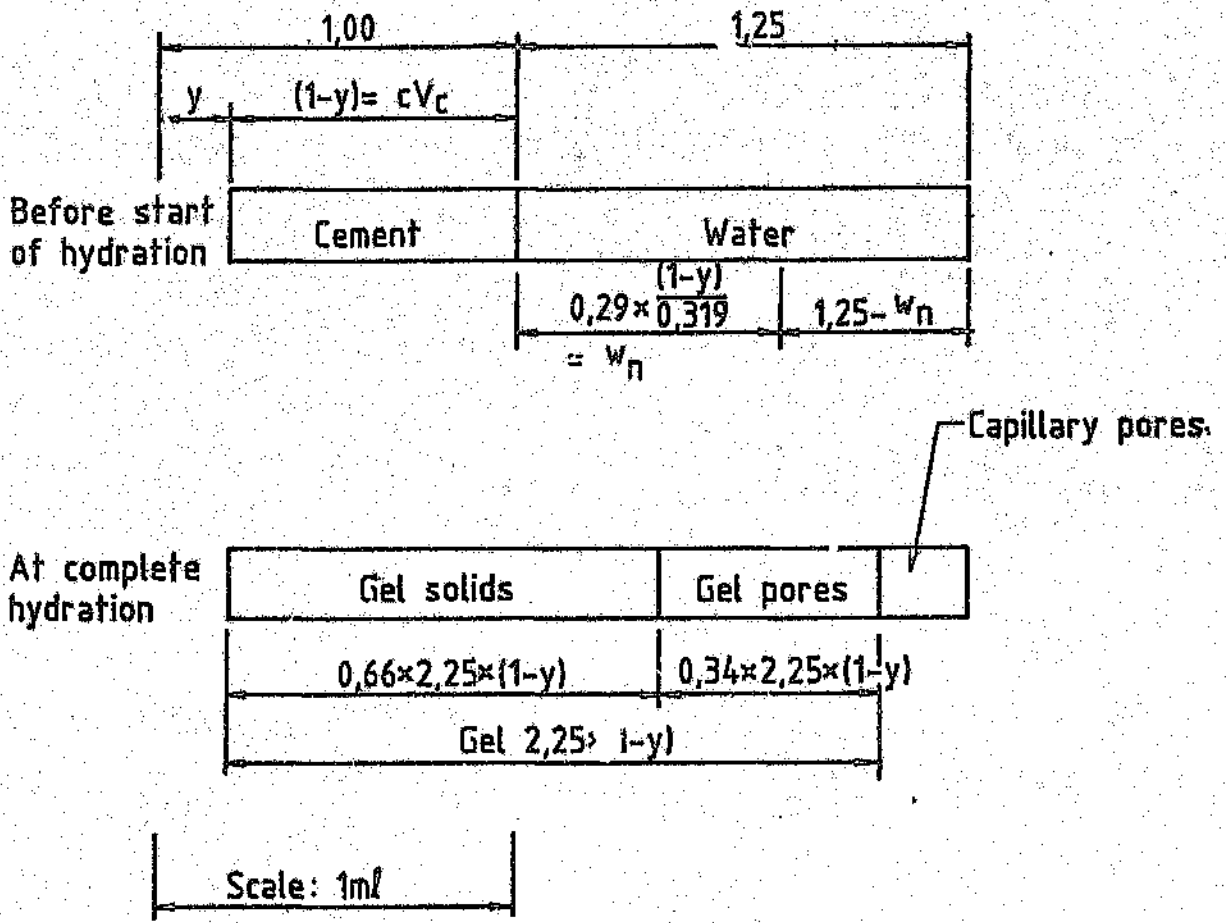


FIGURE 7.3: Absolute volumes of phases of cement paste for cement-undersaturated mix

This means that the density of the gel solids is greater than the weighted average density of the reacting substances by a factor of 1,91/1,48 = 1,3.

Furthermore, the water not used for hydration is insufficient to fill the gel pores. At first glance the shortfall appears to be 0,77 – 0,34. But the water in the gel pores is actually "compressed" by the forces of adsorption to a density some 10 % higher than free water.[7.6] The volume of water required to be drawn into the system, if it is not to become desiccated, is therefore approximately (0,77 x 1,10) – 0,34 = 0,51 mℓ. The

mass of this water is about 16 % of the mass of cement hydrated. The possible self desiccation of hydrating cement paste is discussed in section 7.3 below.

In Figure 7.2, the mix ratio is such that there is too little space for the cement to hydrate completely. The system, after no further hydration is possible, is filled partly with unhydrated cement. "Cement–oversaturated" is proposed as a term to describe such a mix. Similar to Figure 7.1, there are no capillary pores in the mature system.

In Figure 7.3, the mix ratio is such that there is more than enough space for the cement to hydrate completely. The system, after hydration is complete, consists of hydrate and capillary pores. "Cement–undersaturated" is proposed as a term to describe such a mix.

Gel pores and capillaries differ essentially as follows:

The total volume of gel pores is a constant, or nearly constant, fraction of overall gel volume. Gel pores are extremely small: between 200 and 400 nm wide as estimated by Powers.[7.8]

Capillary pores are spaces between bodies of gel and the total volume of capillary pores is independent of the volume of gel. Capillary pores are much larger in size than gel pores and are the residue of the original water–filled spaces in the fresh paste. They may be interconnected or separated from each other by gel.

It should also be noted that Figures 7.1, 7.2 and 7.3 are idealized to some extent by showing complete hydration. The dense layer of gel which forms on the surface of a cement particle tends to stifle hydration. It has been found that after 28 days in water, cement particles are hydrated to a depth of only 4 μm.[7.7] Powers[7.4] calculated that complete hydration was possible only for particles smaller than 50 μm.

The spacing of cement particles in water may also serve to constrain hydration. As explained in chapter 5, particles are not spaced at regular intervals. Instead particles

arrange themselves close together to form agglomerations. It is possible that, within these agglomerations, the water-filled space is insufficient for full hydration to take place. In other words, within the agglomerations the mix is cement-oversaturated. The consequence is that the cement is used inefficiently. If this is true, it should be possible to improve the efficiency of cement usage by spacing the particles more evenly in the water.

Can the effects of particle size and packing on hydration be quantified?

Figure 7.4 is an example of the particle-size distribution of a rapid-hardening portland cement. Particle-size distribution was determined by the cement manufacturer using a laser scattering technique. In the figure, 5 % of the cement consists of particles larger than 50 µm. Based on the statement from Reference 7.4 above, roughly 5 % of the cement would be unable to hydrate completely to the centres of particles. Particles larger than 50 µm would however hydrate partially. This means that the proportion of cement not available for hydration is much less than 5 %.

Unfortunately the packing of cement particles in water cannot be quantified. The presence of large particles and agglomeration of particles will both have the effect of increasing the cement:water ratio for cement-saturation to a value greater than the ratio calculated above.

Cement:water ratio for cement-saturation depends on the proportion of cement unable to hydrate and can be calculated approximately from Figure 7.2.

Let fraction of cement that is unable to hydrate be x.

$$\text{Then } C/W = \frac{1 + x}{0{,}319 \times 1{,}25} = \frac{1 + x}{0{,}399}$$

Therefore: if x = 0, C/W = 2,5
if x = 0,05, C/W = 2,63
if x = 0,10, C/W = 2,76

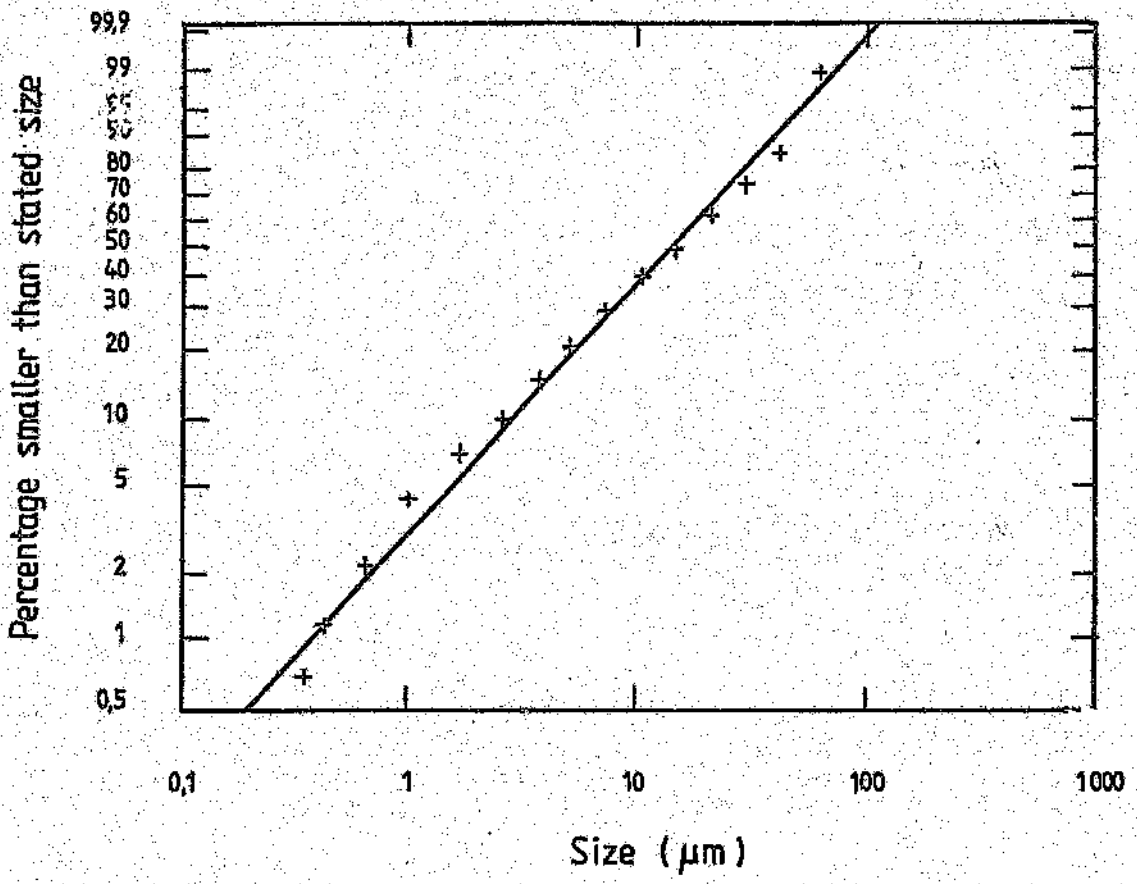


FIGURE 7.4: Particle-size distribution of cement sample

To investigate the role of unhydrated cement in mature paste consider the effects of cement-saturation on the properties of concrete.

Possibly the concrete property that is easiest to measure is compressive strength. The first part of this discussion will therefore deal with cement-saturation and compressive strength.

Taking a given quantity of water, say per cubic metre of concrete, the compressive strength of the concrete at a given age increases approximately linearly with increasing cement content within the range of $1 < C/W < 2$.[7.6]

At higher values of C/W the rate of increase of strength with increasing C/W decreases steadily and eventually becomes almost negligible. This is shown schematically in Figure 7.5.

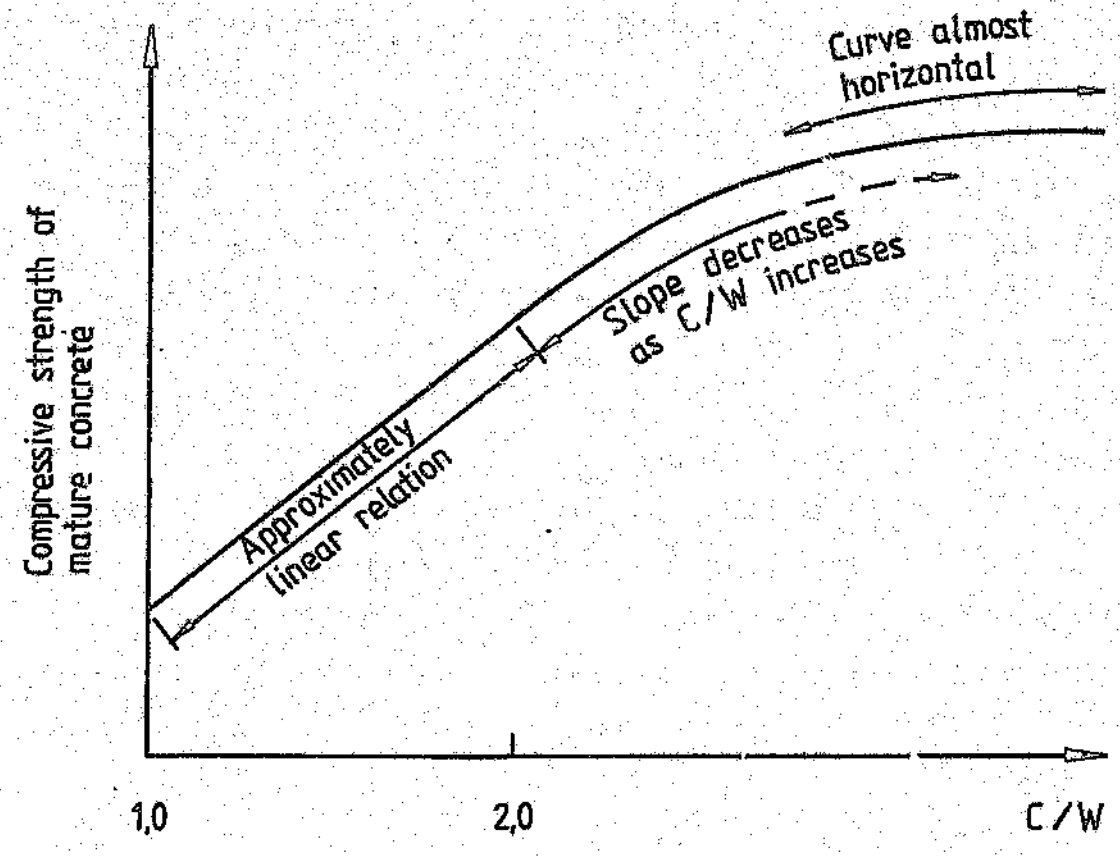


FIGURE 7.5: Compressive strength of mature concrete versus cement:water ratio

Is the zone in which the slope of the curve in Figure 7.5 rapidly becomes flatter where cement–saturation takes place?

Consider firstly a cement–undersaturated mix. If a small amount of extra cement is added, it will have sufficient space in which to hydrate and, by hydrating, will convert some of the water into a solid material. This reduction in porosity will effect an increase in strength.

Now consider a mix in which C/W is exactly at the point of cement–saturation; gel:space ratio = 1,0. Assume that full hydration is possible throughout. If a small amount of extra cement is added it will not increase the amount of cement able to hydrate and a fraction of the cement, equivalent to the size of the increment, will remain unhydrated and act as fine "aggregate" particles in the mature paste. The increment as such will therefore not be able to contribute directly to increased strength by converting pores to solids; the cement as a whole will be less efficient than before the increment was added. But it is possible that the cement increment could increase strength indirectly. Each "cement aggregate" particle will be at or near the centre of a partially hydrated particle and will be strongly attached to the surrounding layer of hydrate. The important difference between these "cement–aggregate" particles and true aggregate particles is that the unhydrated cement is strongly bound to the hardened–cement paste whereas ordinary aggregate particles are relatively weakly bound. The unhydrated portion of the cement, which would replace aggregate of equivalent volume, could therefore increase compressive strength indirectly by improving the average strength of bond at the paste–aggregate interface.

It is important to note that the increase in strength which results from an additional amount of cement is caused by fundamentally different mechanisms in cement–undersaturated and cement–saturated mixes. In the former, strength is increased by a reduction in capillary porosity, ie a volumetric effect. In the latter, strength is increased by improving interface bond, ie a surface effect. It seems reasonable to expect that the effect on strength of a given cement increment would be more effective in a cement–undersaturated mix than it would be in a cement–saturated mix.

The phases of concrete for $2 < C/W < 4$, as calculated from Figures 7.1, 7.2 and 7.3, are shown in Figure 7.6.

It is therefore probable that the zone of C/W in which cement–saturation takes place is where the slope of the strength–vs–C/W curve rapidly flattens. To investigate this in detail it is necessary to look more closely at the efficiency of cement usage with respect to strength.

To quantify cement efficiency the concept of a "cement–efficiency factor" (CEF) is proposed. CEF is a measure of the efficiency of the cement's contribution to strength and is defined as cube strength in MPa per 100 kg of cement per cubic metre of concrete.

Preferably data obtained from mixes at different C/Ws but constant water content should be used to calculate CEFs. This is because original water content has a strong influence on porosity of the mature concrete and porosity in turn influences strength.[7.9]

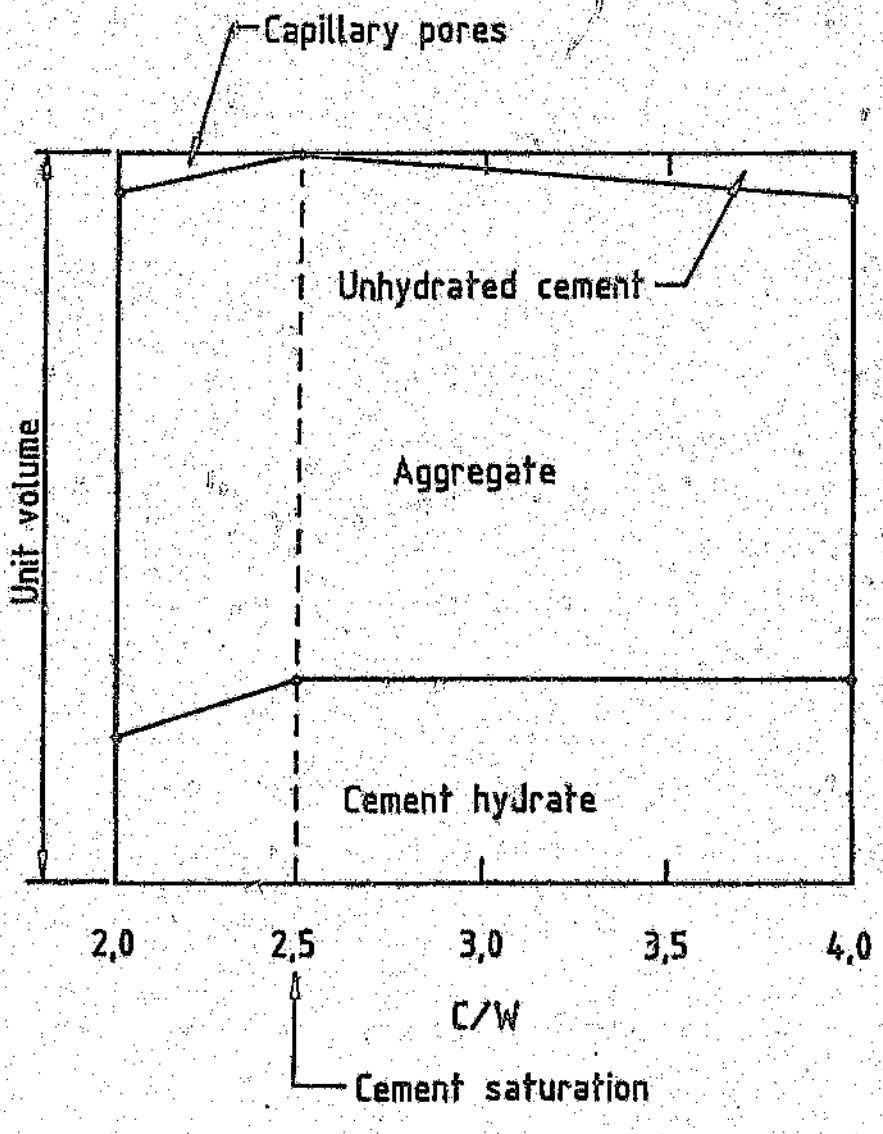


FIGURE 7.6: Volume proportions of phases of hypothetical concrete for $2 \leq C/W \leq 4$; constant volume of mixing water 150 ℓ/m^3

Fortunately some concretes made in the laboratory with sulphonated naphthalene formaldehyde (SNF) superplasticizing admixture had similar water contents over a wide range of C/W.

To effect a comparison, results from two sets of mixes were adjusted by: changing water contents to a constant value equal to the average for the set; changing cement content to the value for the average water content at each specific C/W; and

increasing or decreasing cube strength by 0,2 MPa for each litre by which water content is decreased or increased respectively bringing it to the average.[a]

Theoretical CEFs according to these adjustments were calculated and are shown in Tables 7.1 and 7.2.

Table 7.1 gives results from mixes made with dolomite stone (a 75/25 blend of 26,5 and 6,7 mm) and dolomite crusher sand. Table 7.2 gives results from mixes made with andesite stone (a 75/25 blend of 26,5 and 6,7 mm) and andesite crusher sand. All mixes were made with rapid–hardening portland cement and SNF admixture (in liquid form dosed at 2 parts by volume to 100 parts of cement by mass).

---

[a] Adjusting strength according to water adj ·l wa. sed on the following:

Kellerman[7,10] found that with OPC, at a constant C/W of 1,9 and a constant stone content, a 10–ℓ increase in water content caused an average reduction in 28–day strength of 1,9 MPa.

To ascertain whether the effect was similar with high–strength concrete, various mixes were made up in the laboratory and strength measured at 7 and 28 days. Results were as follows:

Cement: RHPC type "J"
Stone: 13,2–mm dolomite; content 1 025 kg/m^3
C/W: 2,6

Water content (kg/m^3)	Slump (mm)	Cube strength (MPa)	
		7 days	28 days
205	45	63,1	78,9
220	70	61,0	73,9
235	85	58,2	72,5

These indicate a change in strength at 28 days of 2,1 MPa per 10–ℓ change in water content.

A strength difference of 0,2 MPa/ℓ change in water content was therefore used to adjust mix results.

Table 7.1: Strengths and CEFs of concrete made with dolomite aggregates

Mix		C/W	2,0	2,4	2,8	3,2	3,6
		Water content (ℓ/m^3)	145	140	135	130	130
Actual results	At age 28 days	Cube strength (MPa)	66	81	94	103	107
		CEF (MPa/100 kg cement/m^3)	22,8	24,1	24,9	24,8	22,9
	At age 56 days	Cube strength (MPa)	–	89	102	112	119
		CEF (MPa/100 kg cement/m^3)	–	26,5	27,0	26,9	25,9
Calculated values based on theoret-ical assump-tion that water is constant at 136 ℓ/m^3	At age 28 days	CEF (MPa/100 kg cement/m^3)	25,0	25,2	24,7	23,4	21,6
	At age 56 days		–	27,6	26,8	25,5	24,1

Table 7.2: Strengths and CEFs of concrete made with andesite aggregates

Mix		C/W	2,0	2,4	2,8	3,2	3,6
		Water content (ℓ/m^3)	160	155	155	150	150
Actual results	At age 28 days	Cube strength (MPa)	68	87	101	109	116
		CEF (MPa/100 kg cement/m^3)	21,2	23,4	23,3	22,7	21,5
	At age 56 days	Cube strength (MPa)	74	94	104	112	117
		CEF (MPa/100 kg cement/m^3)	23,1	25,3	23,0	23,3	21,7
Calculated values based on theoret-ical assump-tion that water is constant at 136 ℓ/m^3	At age 28 days	CEF (MPa/100 kg cement/m^3)	22,4	23,5	23,4	21,1	20,8
	At age 56 days		24,4	25,4	24,1	22,5	20,9

Theoretical CEF, as shown in the tables, is plotted against C/W in Figure 7.7. The figure shows that maximum CEF at 28 and 56 days for series 2, 3 and 4 is at C/W of about 2,4 to 2,6.

Notwithstanding the relatively crude approach – see section 7.2.1 – the C/W for maximum CEF at 56 days is similar to C/W assumed for cement-saturation (C/W = 2,5). It is also probable that very little hydration could be expected after 56 days.

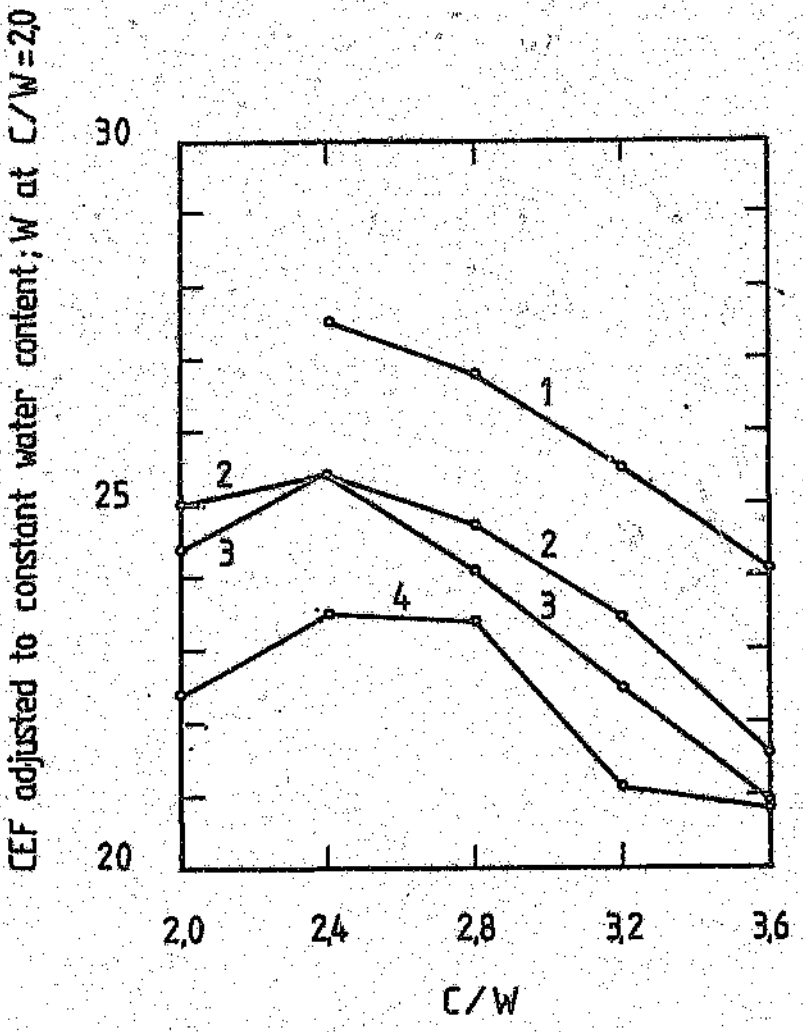


FIGURE 7.7: Cement–efficiency factor adjusted to constant water content versus cement:water ratio

1. Dolomite aggregate; age 56 d
2. Dolomite aggregate; age 28 d
3. Andesite aggregate; age 56 d
4. Andesite aggregate; age 28 d

It may therefore be accepted tentatively that CEF at 56 days is a maximum when the mix is at or close to cement–saturation.

It also seems reasonable to regard cement–saturation as a phenomenon which manifests itself not at a precise value on the C/W scale but in the range of 2,4 < C/W < 2,6.

It is interesting to observe the performance of real mixes. In Figures 8.14 and 8.16 for example, although individual results are erratic, envelope curves drawn around the results show maximum CEF between C/W 2,4 and 2,8. Similar envelope curves in Figures 8.21 and 8.23 reach maximum values at C/W = 2,6 and 2,8.

To summarize the above discussion it can be said that:

- The maximum strength attainable with given materials is at a C/W higher than C/W for cement–saturation.

- The strength–giving properties of cement are used most efficiently at a C/W roughly the same as C/W for theoretical cement–saturation.

7.3 Self–desiccation of hydrating cement paste

As explained above in section 7.2.3, hydrating cement paste needs to draw water into itself if hydration is to continue in a completely water–filled environment.

The pastes of high–strength concrete, because of their high cement:water ratios, rapidly become very impermeable as hydration progresses. Is it possible therefore that the surface layer of concrete prevents water from reaching the internal regions? If this is so these regions would become desiccated as mixing water is used up and further hydration would be prevented.

According to Reference 7.11, the only time in the life of the concrete when water can move freely into it is the first 24 hours after mixing and compacting. The reference states that:

> After a day, these low water–cement ratio concretes are so impermeable that very high pressures are required to force additional water into them.

If this statement is correct, even if concrete is cured in water, hydration of the cement should cease as soon as self–desiccation has depleted the water in the concrete.

In Reference 7.12 Powers discusses the question: how dry may the concrete become without stopping hydration altogether? (In this respect drying can of course be caused by self desiccation or by evaporation from the surface.) Powers reports that, when fresh cement pastes were placed in sealed bottles and the vapour pressure inside the bottles measured from time to time, it was found that the vapour pressure decreased rapidly at first, then decreased less rapidly, and then stayed roughly constant. Relative humidity (RH) did not drop below about 75 % to 85 %. In another investigation, samples of dry cement were stored for 6 months in atmospheres of different relative humidity. It was found that samples stored at relative humidity lower than 50 % remained free–running and did not change in appearance. At 70 % RH the sample became somewhat lumpy. At 80 % RH it formed a caked mass which could be broken up without much difficulty. At 90 % RH the mass was harder and in a saturated atmosphere the sample became a solid cake with considerable strength.

To appreciate the implications of these findings it is necessary to consider the conditions required for hydration.

Hydration products form only in water–filled space; they are for example unable to fill air bubbles in paste.[7.12] In an atmosphere with less than 100 % RH, hydration products form only in the collar of water formed by capillary condensation which occurs at the points of contact between grains of cement. The amount of water that condenses under these conditions, and therefore the space available for the formation of hydration products, is a function of the relative humidity of the atmosphere.

This accounts then for the differences in hardness of the cement samples placed in atmospheres of different relative humidities as described by Powers above. The higher the relative humidity the greater the volume of the water collar formed at the point of contact between each pair of grains and the stronger the connecting links of hydration products.

From the experimental results Powers concludes that, for practical purposes, hydration continues in an atmosphere of RH > 80 % but is unable to continue if RH drops below 80 %. This threshold value of RH is not absolute. It probably depends on the concentration of dissolved electrolytes which in turn depends on the original ratio of water to cement in the paste and the chemical characteristics of the cement.

How much of the water in a water-filled void must be removed to produce a relative humidity of 80 %?

Consider a hermetically sealed container with a volume of 1 m^3. The container is fitted with inlet and outlet valves. When the container is filled with water it contains 10^6 g of water, all in the liquid state. Assume that if water is let out of the container only dry air can enter. If part of the water is let out, and the system allowed to achieve equilibrium, the air above the water will be saturated with vapour. The amount of vapour per unit volume contained in the air space will depend on the temperature. If more water is let out, the air space will continue to be vapour saturated as long as there is a free surface of liquid water present. If the quantity of liquid water is slowly reduced until it is only very little more than zero, and then reduced to zero, the whole volume will contain vapour-saturated air. The mass quantity of water in the vapour state required to cause vapour saturation depends on the temperature. At 20 °C, for example, 17 g of vapour will saturate 1 m^3 of air. At 80 % relative humidity the vapour pressure is 80 % of the pressure of saturated air. But the concentration of vapour is roughly proportional to the relative vapour pressure. The quantity of vapour in 1 m^3 at 20 °C at a relative humidity of 80 % is therefore about 0,8 x 17 = 14 g. To produce a relative humidity of 80 %, 999 986 g out of the original 1 000 000 g must be removed. For practical purposes then we can say that

when all of the water in a water–filled void is removed, the relative humidity will be such that hydration of cement in the void will cease.

Cessation of hydration of a sealed specimen therefore occurs when desiccation reaches the point that the capillary voids have been emptied of water.

In sealed pastes with low cement:water ratios we could expect that, because there is such an excess of water, this point would never be reached. Sealed pastes with high cement:water ratios on the other hand would soon reach the point where hydration ceases. At some intermediate cement:water ratio, capillary water would be used up exactly when hydration is completed. At this critical ratio, self desiccation would not prevent full hydration. The critical cement:water ratio is calculated as follows:

Volumetric ratios are as shown in Figures 7.1, 7.2 and 7.3.

Consider the hydration of unit volume (1 mℓ) of cement.

The original mass of cement is

$$\frac{1}{0{,}319} = 3{,}13 \text{ g.}$$

The mass of water chemically bound with the cement is 0,29 x 3,13 = 0,91 g.

The cement hydrates to a volume of 2,25 mℓ, ie the volume of gel is 2,25 mℓ.

The gel is 34 % gel pores and 66 % solids. Therefore the volume of gel pores is 0,34 x 2,25 = 0,77 mℓ and the volume of gel solids 1,48 mℓ.

As shown in Figure 7.8, 1,0 mℓ of cement combines with 0,91 mℓ of water to form 1,48 mℓ of gel solids. This is a volume reduction of 1,91 – 1,48 = 0,43 mℓ. Also in the figure, 0,85 mℓ of mixing water is required to fill 0,77 mℓ of gel pores – a volume reduction of 0,85 – 0,77 = 0,08 mℓ. Total volume reduction is 0,43 + 0,08 = 0,51 mℓ.

This is the volume of water that would be drawn into the system if water could get into it from outside. The overall volume when mixed is 1,0 + 0,91 + 0,85 = 2,76 mℓ. The volume of gel (solids + pores) is 2,25. But overall vol ime must be the same before and after hydration; therefore the volume of capillary pores is 2,76 – 2,25 = 0,51.

In other words the volume of capillary pores is equal to the volume of internal shrinkage due to hydration.

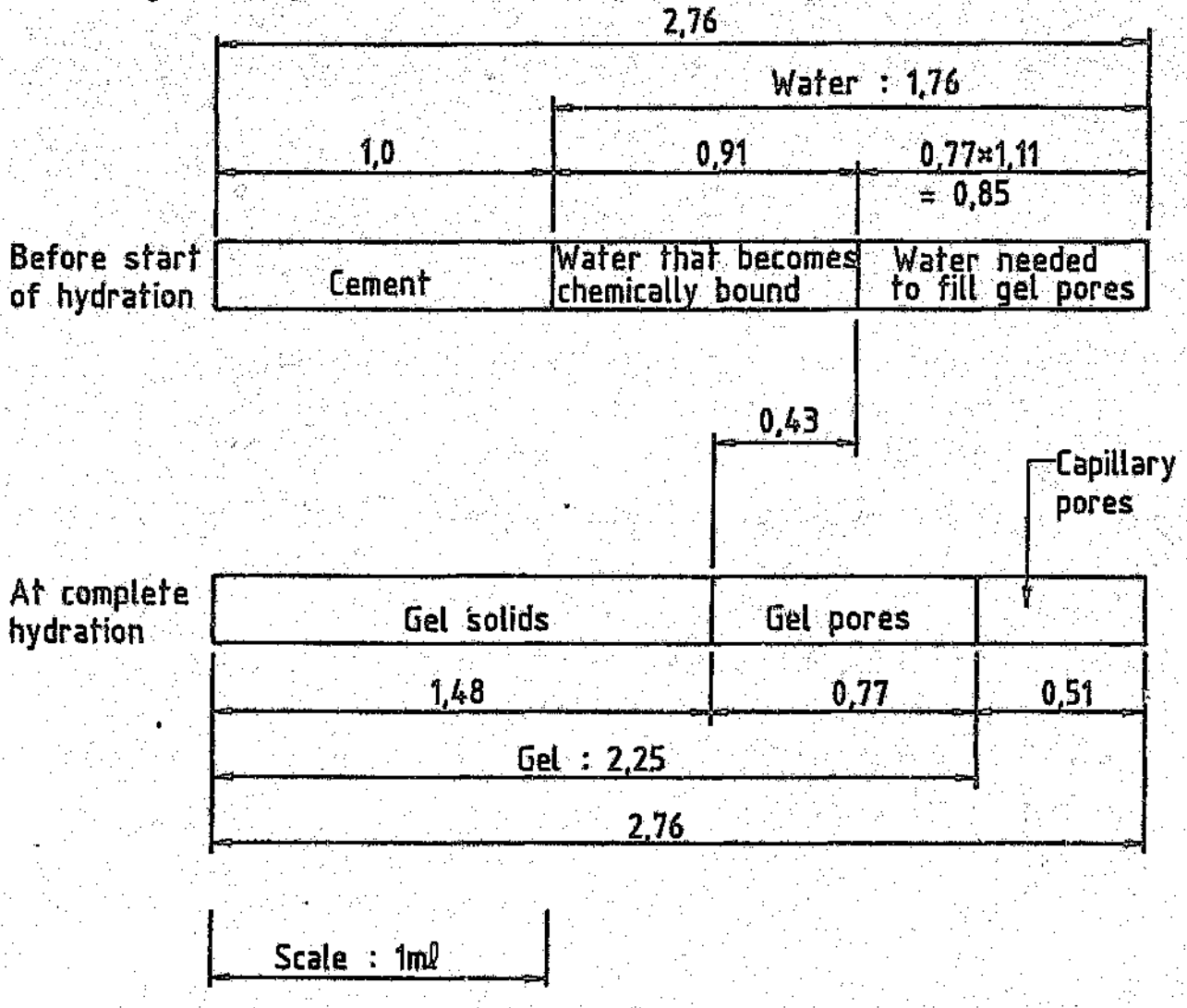


FIGURE 7.8: Calculation of critical cement:water ratio

If the system is sealed the capillary pores will not contain any water when hydration is complete.

The original cement:water ratio shown in Figure 7.8 is therefore the critical ratio.

$$\text{Critical } C/W = \frac{3{,}13}{1{,}76} = 1{,}8.$$

(In Reference 7.12, Powers, by a different approach and for the specific cement considered, arrives at a value of 2,0. Calculations of this sort are tentative and the values determined should be regarded as approximate at best.)

This means that:

If C/W ≤ about 1,8 a sealed specimen contains sufficient water to hydrate fully.

If C/W > about 1,8, a sealed specimen will cease to hydrate when the mass of **hydrated** cement reaches 1,8 times the mass of water in the original mix.

High–strength concrete would normally have a cement:water ratio well in excess of 1,8.

The statement quoted above from Reference 7.11 implies that, after the age of one day, the surface layer of a specimen of high–strength concrete effectively prevents the ingress of water to the internal concrete, even if the specimen is immersed in water. Such a specimen would then behave like a sealed specimen and would self–desiccate rapidly due to hydration. The core of the specimen would, after a relatively short time, cease to gain strength. If this were to happen, strengths would show very little gain after say 7 days.

Actual results obtained with water cured specimens with C/W ranging from 2,0 to 3,6 showed rapid strength gains up to 7 days and steady strength gains between 7 and 56 days. See Figure 7.9. This indicates that water is able to move into high–strength concrete even after 28 days of curing in water.

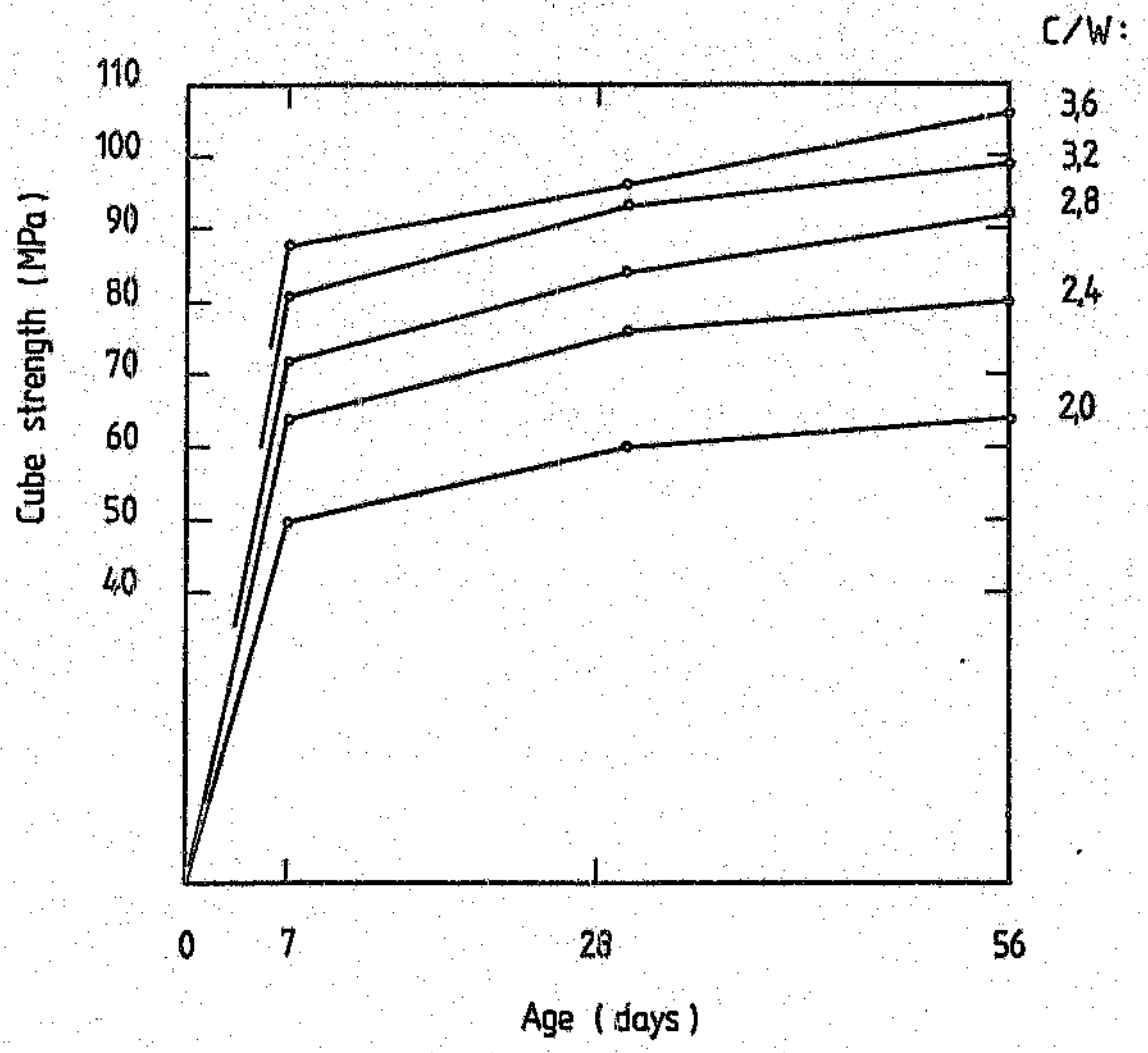


FIGURE 7.9: Strength development of actual concretes versus age; concrete made with RHPC type J, 13,2–mm dolomite stone, dolomite crusher sand, SNF admixture; specimens cured in air with 100 % RH for first 24 h then in water; testing done in wet state

The above discussion leads therefore to the following conclusions:

- The statement made in Reference 7.11 is incorrect; it is in fact based on no more than a comparison of cubes (with W/C = 0,3) that were sealed and cubes that were ponded with water during the first 24 hours after being cast. Ponded cubes were marginally stronger at seven days; the difference in strength decreased thereafter and was negligible at 365 days. The author of

reference 7.11 contradicts his statement by showing that of both sets of cubes increased in strength by about 24 % between 28 and 365 days. This increase could happen only if water is able to move into the concrete. In fact he shows only that a lack of additional water during the first 24 hours retards but does not prevent strength development.

- High-strength concretes should be cured by making excess water available on the surface of the concrete. In practical terms this could be achieved by wrapping with water-saturated absorbent fabric and sealing this with plastic sheeting.

- Curing should start as soon as it is possible to demould the concrete without damaging it.

- High-strength concretes should be water cured for as long as possible.

7.4 Conclusions

The potential hydration of cement is a function of the chemical characteristics of the cement but actual hydration depends on the availability of space and water.

Cement-saturation is a point on the C/W scale which effectively divides pastes into two categories with fundamentally different properties. The properties of cement-undersaturated pastes are dominated by porosity, especially capillary porosity; those of cement-oversaturated pastes by the properties of interfaces. The value of C/W for cement-saturation can however not be determined precisely.

Pastes made to a C/W above a critical ratio are unable to continue hydrating once the original water is used up. Concretes made with such pastes must therefore be cured by providing extra water at the surface. Very rich mixes, although highly impermeable, appear to be able to take in water for hydration.

8. STRENGTH OF CONCRETE

8.1 Introduction

Strength is the property of concrete which is specified and measured more often than any other property.

Of the various parameters of strength: compressive, flexural, splitting–tensile and direct tensile, compressive strength is by far the most widely used as an indicator of concrete quality.

Basic to the understanding of the compressive strength of concrete is the recognition of the fact that failure occurs mainly in a tensile mode.[2.46, 8.1, 8.2, 8.3] A model of a chain in tension can therefore be used to illustrate the strength of concrete.[2.18] The links in this chain are aggregate, hardened cement paste (HCP), and the interface between aggregate and HCP. The model is valid for any strength of concrete but the importance of each link, and which is the weakest, may change as strength increases.

The first part of this chapter deals with the relative importance of aggregate and HCP in determining the compressive strength of concrete.

Selecting the size of coarse aggregate is an important option available to the concrete technologist. References in the literature[2.17, 2.18] tend to favour smaller sizes (c.10 mm) for high–strength concrete. Optimum size of coarse aggregate, using South African materials, is investigated in the second part of this section.

In order to investigate the performance of South African materials in high–strength concrete, the effect on compressive strength of portland cements from different sources, blends of portland cement and different cement extenders, and some aggregate types is compared.

Splitting–tensile strength and flexural strength of concrete made with local materials are studied by using different aggregates and various cement:water ratios.

Compressive strength of concrete is determined by loading plane-ended specimens and measuring maximum load. Test methods specify, inter alia, the shape and aspect ratio of specimens, the number of specimens required for a test, and the permitted variability within a batch of specimens comprising a test.

Because shape and aspect ratio can affect the measured strength of the concrete, results obtained by different methods are not necessarily the same and may require manipulation by factors to make them comparable. The possibility that these factors are influenced by the strength of the concrete is investigated.

The within-batch variability of compressive strength of high-strength concretes made in the laboratory is investigated and the relevance of test methods developed originally for conventional concrete strengths is assessed.

8.2 The relative importance of aggregate and hardened cement paste with regard to strength

As discussed above, the factors which determine the strength of mature concrete are: the strength of the hardened cement paste (HCP), the strength of the aggregate, and the strength of the bond between paste and aggregate.

In conventional concrete, the strength of the HCP is the dominant factor, providing that aggregates of acceptable strength are used. Davis,[8.4] for example, found that the compressive strength of normal structural-grade concrete made with the types of stone commonly used in South Africa (10 % fines aggregate crushing test (FACT)[8.5] of 160 kN and over) is not significantly affected by the crushing value of the aggregate.

Experience gained in the Portland Cement Institute laboratory, in which mixes made with many different aggregate types are proportioned, has also shown that cement:water ratio is a good indicator of strength in the conventional range and that aggregate type has only a minor effect.

The strength of the bond between HCP and aggregate is a function of: the strength of the HCP; the strength and roughness of the surface of the aggregate; and the degree to which voids or other defects are present at the interface. This bond strength is difficult to study in isolation but for a given aggregate it is a function of the strength and microstructure of the HCP. Conversely, for a given quality of paste, bond strength is a function of the surface properties of the aggregate.

An examination of compression specimens of conventional concrete after failure shows that most fractures occur in the cement paste and on interfaces between paste and aggregate. With increasing compressive strength, failures through aggregate particles become increasingly evident. These observations, which are confirmed in the literature,[2.48] indicate that aggregate properties become increasingly important as concrete strength is increased.

To determine the relative importance of the aggregate and the HCP at different concrete strengths, a series of mixes made with various aggregate types: andesite, dolomite from two sources, dolerite, quartzite with washed and unwashed sand, and granite, was analysed. In Figure 8.1 the coefficient of variation (CV) of strengths, at 56 days, is plotted against cement:water ratio (C/W). (CVs at 7 and 28 days showed a much higher scatter).

(Full details of the mixes are given later in section 8.5.)

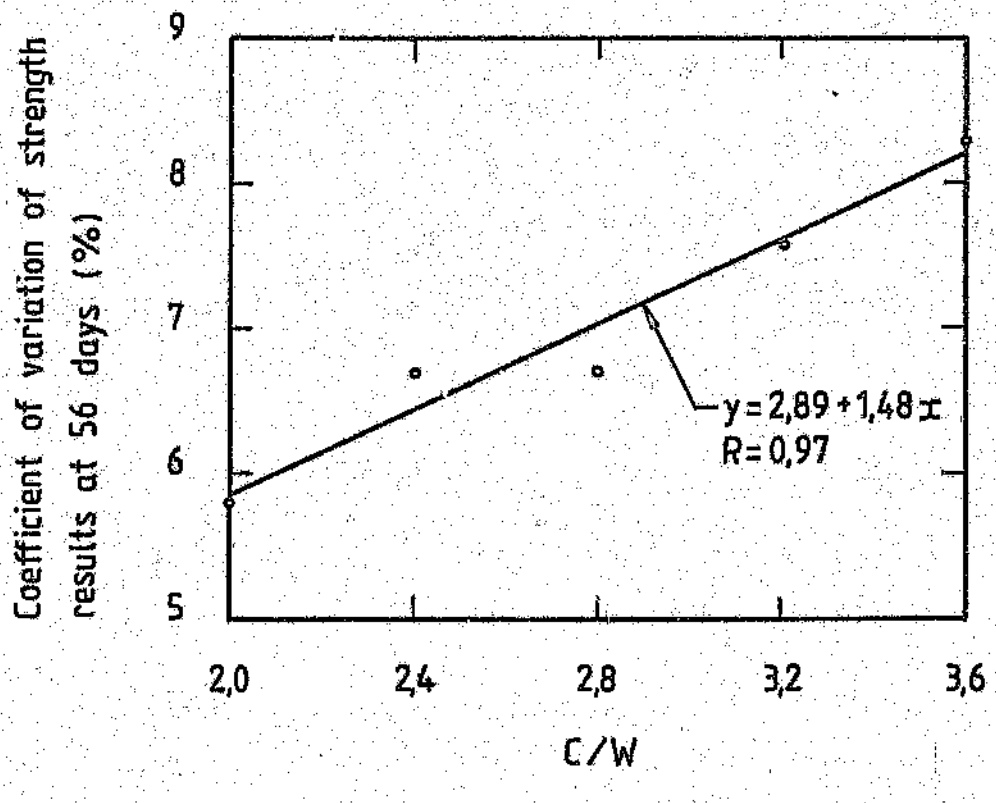


FIGURE 8.1: Concrete made with different types of aggregate, coefficient of variation versus cement:water ratio; each point represents seven mixes each made with different aggregates

The figure, which incidentally shows a surprisingly high degree of correlation, shows that the coefficient of variation is greater at C/W = 3,6 than at C/W = 2,0. The same cement was used for all the mixes so this variation is caused by a combination of sampling and testing variability and the differences between aggregates. If, as could be reasonably expected, the CV of sampling and testing is the same for all values of C/W, the differences are an indication of the effect of the aggregates. This effect could be related to the intrinsic strength of the aggregate or to the interaction between aggregate and paste.

To quantify these effects, albeit tentatively, assume that CV of sampling and testing is 3 %.

$$CV_T^2 = CV_{s,t}^2 + CV_a^2$$

where CV_T = total CV

$CV_{s,t}$ = CV of sampling and testing

CV_a = CV of aggregates

At C/W = 2,0

$$CV_a = \sqrt{5,85^2 - 3^2} = 5,0$$

At C/W = 3,6

$$CV_a = \sqrt{8,22^2 - 3^2} = 7,7$$

In other words the differences between aggregates have a greater effect on the variability of strengths at C/W = 3,6 than at C/W = 2,0. Aggregate properties therefore have a greater influence on the strength of mature concrete at high C/Ws than at low C/Ws. If the above estimate of CV of sampling and testing is reasonable, the effect of aggregates at C/W = 3,6 is about one-and-a-half times greater than at C/W = 2,0.

The choice of suitable aggregates is consequently more important for high-strength concrete than for conventional concrete.

8.3 Optimum stone size

The technical literature, for example references 2.17 and 2.18, includes statements to the effect that smaller stone sizes are preferable to larger sizes for the manufacture of high-strength concrete because higher strengths, at a given cement:water ratio, are possible with smaller stone.

To test the validity of these statements with regard to South African aggregates, mixes were made in the laboratory with different sizes of stone from the same source. Cubes were made and tested in accordance with SABS Method 863.[8,17]

8.3.1 Materials used

Cement

Rapid-hardening cement type "R" was used for all mixes. An analysis for the cement was not available.

Aggregates

Andesite from Eikenhof Quarry was used in the following nominal sizes: 26,5 mm, 19 mm, 13,2 mm, 6,7 mm and crusher sand.

Physical properties of the aggregates are shown in Table 8.1.

Admixture

A sulphonated naphthalene formaldehyde (SNF) superplasticizer in powder form manufactured by a well known admixture company was used for all mixes. It is referred to below as type "C".

8.3.2 Mix proportioning

All mixes were proportioned according to the method described above in section 6.4.

Table 8.1: Physical properties of andesite aggregates

% passing sieve size (mm)	26,5–mm stone	19–mm stone	13,2–mm stone	6,7–mm stone	Crusher sand
37,5					
26,5	100	100			
19,0	11	72	100		
13,2	1	12	97		
9,5	1	1	26	100	
6,7	1	1	3	99	100
4,75	1	1	2	76	99
2,36	1	1	2	13	60
1,18				4	35
0,60				4	24
0,30				3	17
0,15				3	13
0,075				3,1	10,6
Fineness modulus	7,82	7,21 (1)	6,70	4,97	3,53
Relative density	2,93	2,93	2,93	2,93	2,91
Loose bulk density (kg/m^3)	1 450	1 460	1 460	1 550	1 580
Compacted bulk density(2) (kg/m^3)	1 540	1 560	1 570	1 680	1 810
% of solids in compacted state	52,6	53,2	53,6	57,3	62,2

Note

(1) The upper limit according to grading limits given in SABS 1083[8.6] is 7,15. The sample was therefore slightly coarser than a 19–mm stone complying with SABS 1083.

(2) The compacted bulk densities of the three largest stone sizes were very nearly the same. This afforded an opportunity to isolate the effect of stone size on concrete properties such as water requirement, cube strength and cement–efficiency factor.

8.3.3 Stone blend

Apart from concretes made with one size of stone, a blend of two sizes was used for one series of mixes. Blending was done to determine the effect on concrete of a stone with improved packing capacity. 26,5–mm stone and 6,7–mm stone were mixed together at various ratios and compacted bulk density measured for each ratio. Results are shown in Table 8.2

Table 8.2: Blends of 26,5–mm and 6,7–mm stone – compacted bulk density and fineness modulus

Percentage in blend		Compacted bulk density (kg/m^3)	FM
26,5–mm stone	6,7–mm stone		
100	0	1 540	7,82
80	20	1 750	7,25
73	27	1 830	7,05
67	33	1 880	6,88

It was decided to use the 80/20 blend for trial mixes. The FM of this blend was very nearly the same as that of the 19–mm stone (FM = 7,21) while the compacted bulk density of the blend was 13,6 % higher than that of the 19–mm stone. A comparison of the mixes made with the blended stone and the 19–mm stone was therefore an opportunity to isolate the effect of packing capacity on the strength and cement–efficiency factor of (CEF) concrete.

8.3.4 Results of mixes

Results of mixes made with 13,2–mm, 19–mm, 26,5–mm and a blend of 26,5–mm and 6,7–mm stone are shown in Appendix A8.1.

Mixes were assessed for stone content in the fresh state and all mixes had an acceptable stone content.

Figure 8.2 shows water requirement, to the nearest 5 ℓ/m^3, versus C/W. Figures 8.3 and 8.4 show cube strength versus C/W at 7 and 28 days respectively. Figures 8.5 and 8.6 show CEF versus C/W at 7 and 28 days respectively.

The following trends emerged from the results:

- Water requirement for various stone sizes was, generally, in descending order: 13,2 mm; 19 mm; blend of 26,5 mm and 6,7 mm; 26,5 mm.

- The difference in water requirement between 13,2–mm and 19–mm stone decreased as C/W increased. But differences in cube strength remained more or less the same with increasing C/W.

- Cube strengths at 7 days for various stone sizes were, in ascending order: 13,2 mm; 19 mm; 26,5 mm; blend of 26,5 and 6,7 mm.

- Cube strengths at 28 days followed the general trend of strengths at 7 days but the order was reversed in some cases.

- CEFs at 7 days for various stone sizes were in ascending order (apart from one small exception at C/W = 2,0): 13,2 mm; 19 mm; blend of 26,5 mm and 6,7 mm; 26,5 mm.

- The ranking of stone sizes in respect of CEF was similar for 7 and 28 days but CEF of the blend of 26,5–mm and 6,7–mm stone dropped slightly below the CEF of 19–mm stone at C/W = 2,0.

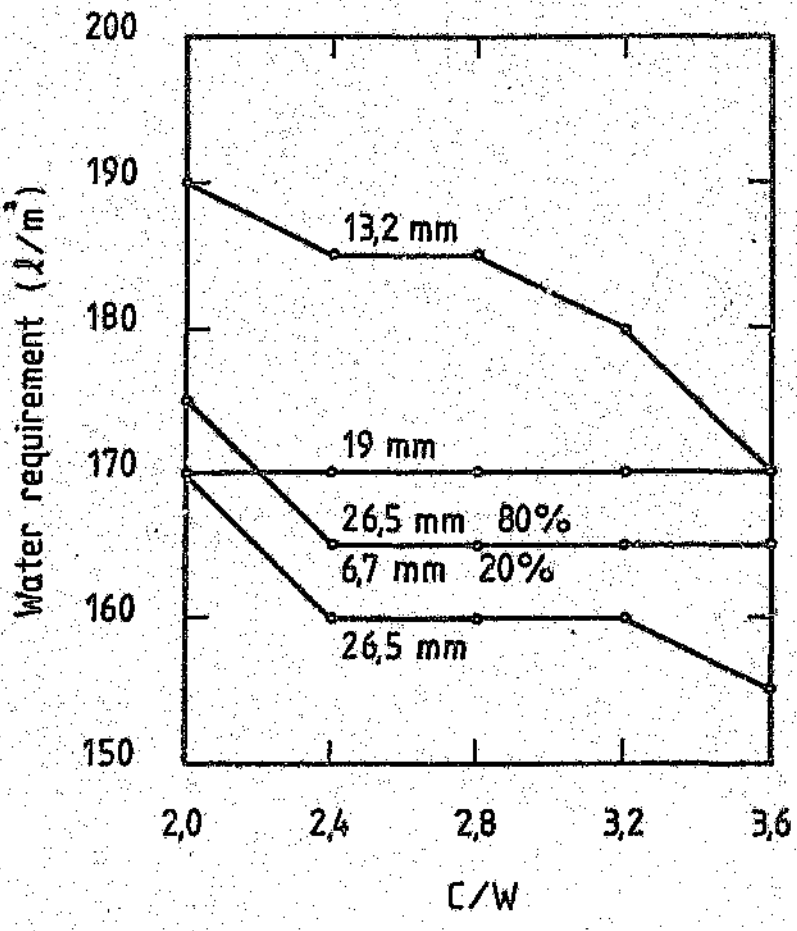


FIGURE 8.2: Water requirement versus cement:water ratio for concrete made with stone of different sizes

8.3.5 Analysis of results

The above test series was done mainly to investigate the effect of stone size on some properties of HSC. But the results also provided an opportunity to look at another aggregate property, namely packing capacity, on the strength of hardened concrete.

8.3.5.1 Packing capacity

The 19–mm stone and the 80/20 blend of 26,5–mm stone and 6,7–mm stone had very similar FMs (7,21 and 7,25 respectively) while their packing capacities, as reflected by compacted bulk densities, differed significantly (1 560 kg/m^3 and 1 750 kg/m^3 respectively). Comparison of the mixes made with these two stone gradings shows therefore the effect of packing capacity on water requirement, strength and CEF.

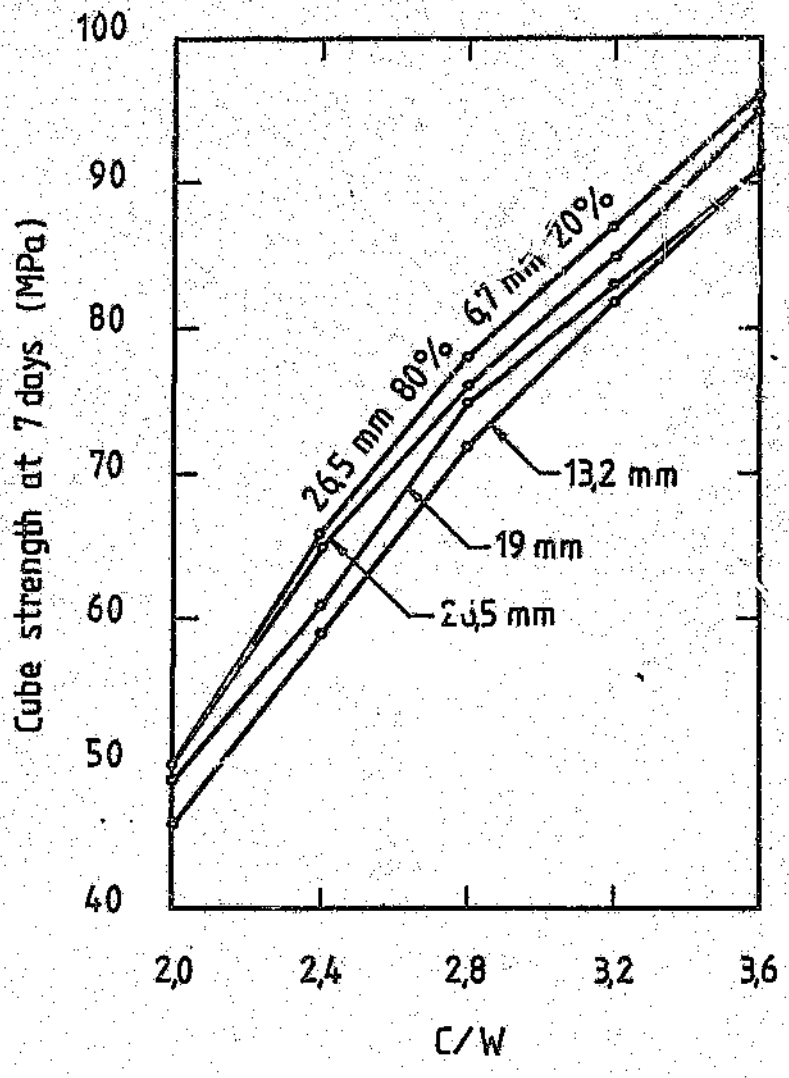


FIGURE 8.3: Cube strength at seven days versus cement:water ratio for concrete made with stone of different sizes

With the exception at C/W = 2,0, the blend had a lower water requirement than the single–sized stone. The difference is only 5 ℓ, which is about 3 % of 170 ℓ. Considering that the difference in packing capacity is about 14 % of that of the 19–mm stone, the effect of packing capacity on water requirement appears to be disproportionately small.

Mixes made with the blend generally gave higher strengths and CEFs at 7 and 28 days than those made with the single-sized stone.

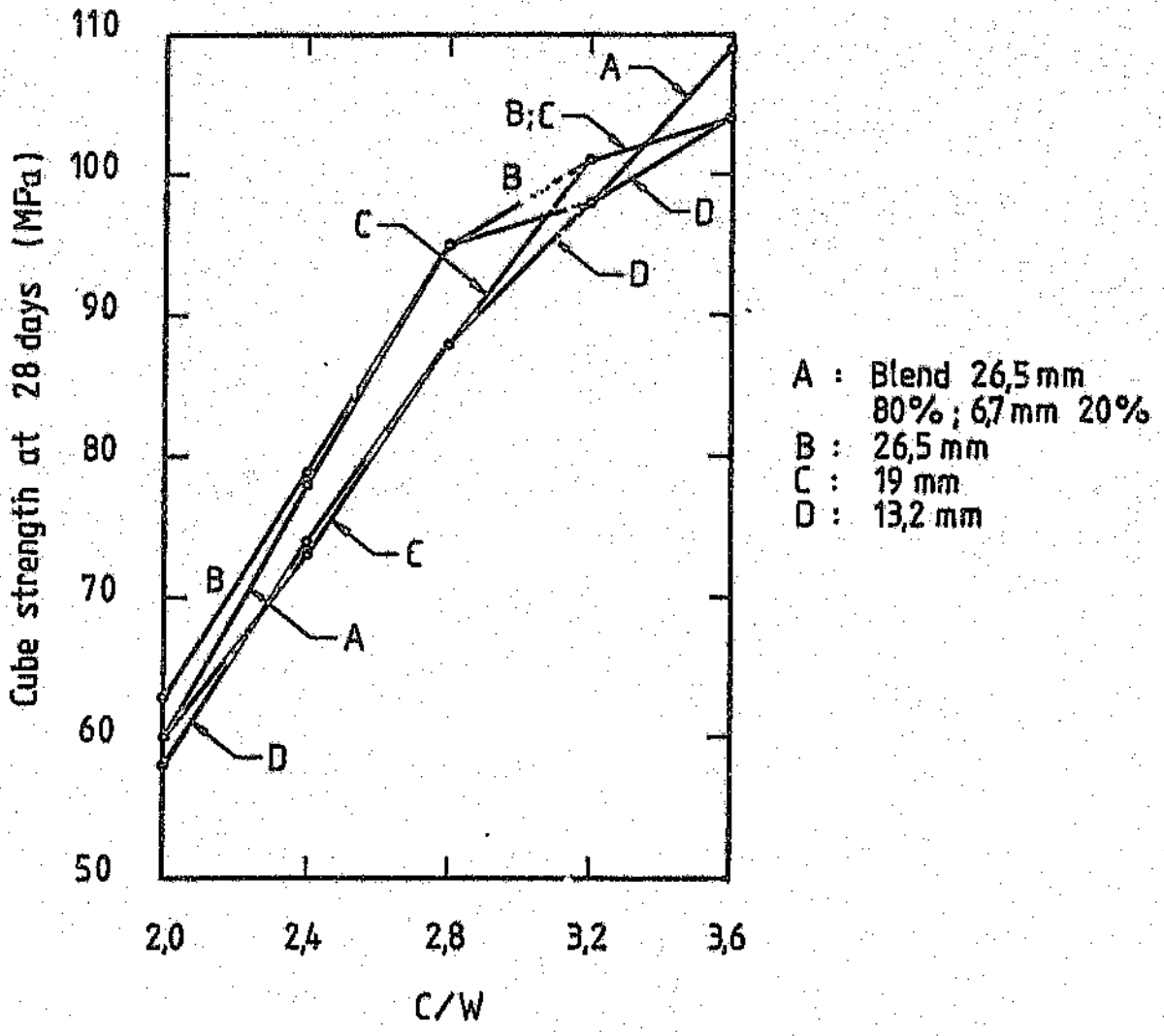


FIGURE 8.4: Cube strength at 28 days versus cement:water ratio for concrete made with stone of different sizes:

8.3.5.2 Size of coarse aggregate

Aggregate size plays a complex role in influencing the properties of fresh and hardened concrete. Influences may be direct or indirect. For example, the surface area of a given quantity of aggregate of given particle shape depends on aggregate size. And

the quantity of aggregate in unit volume of mix, for optimum workability, depends partly on aggregate size.

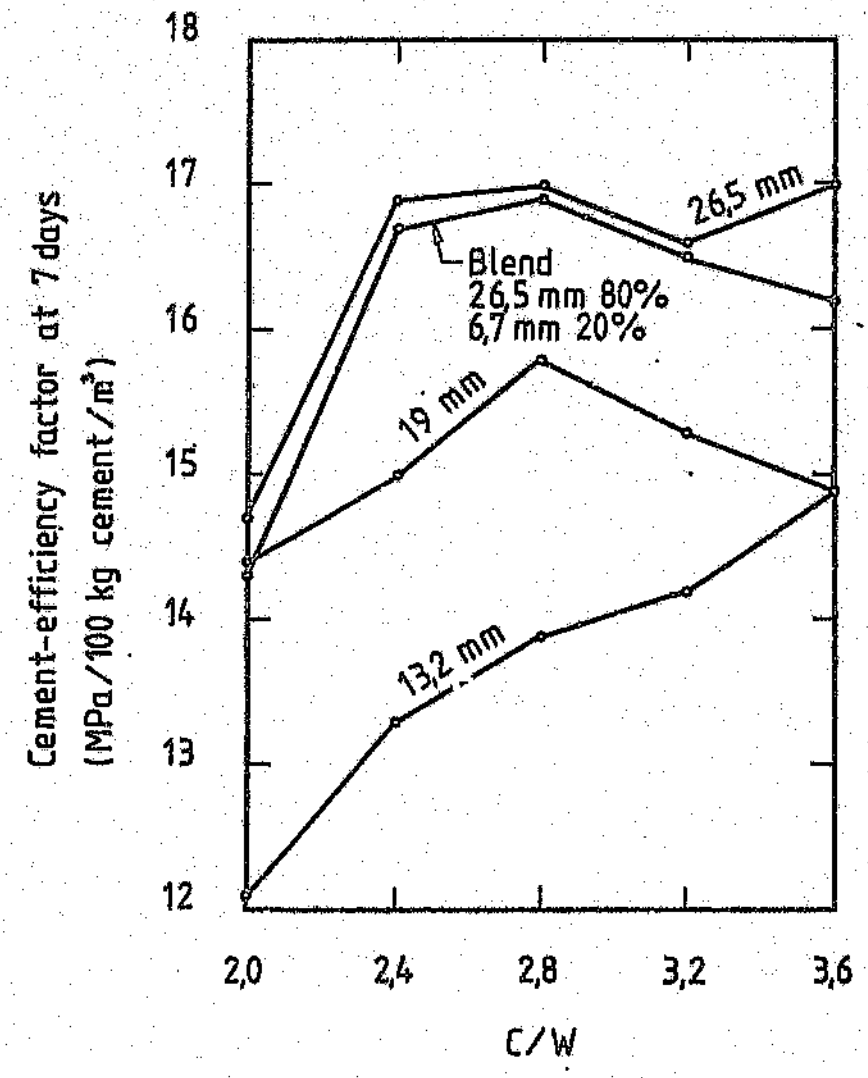


FIGURE 8.5: Cement–efficiency factor at seven days versus cement:water ratio for concrete made with stone of different sizes

Increasing the size of coarse aggregate in a mix would have two conflicting effects on the total surface area of coarse aggregate in the mix:

- The specific surface area of the coarse aggregate would be reduced.
- The quantity of coarse aggregate would be increased and the quantity of fine aggregate reduced for optimum workability. (See section 6.4.)

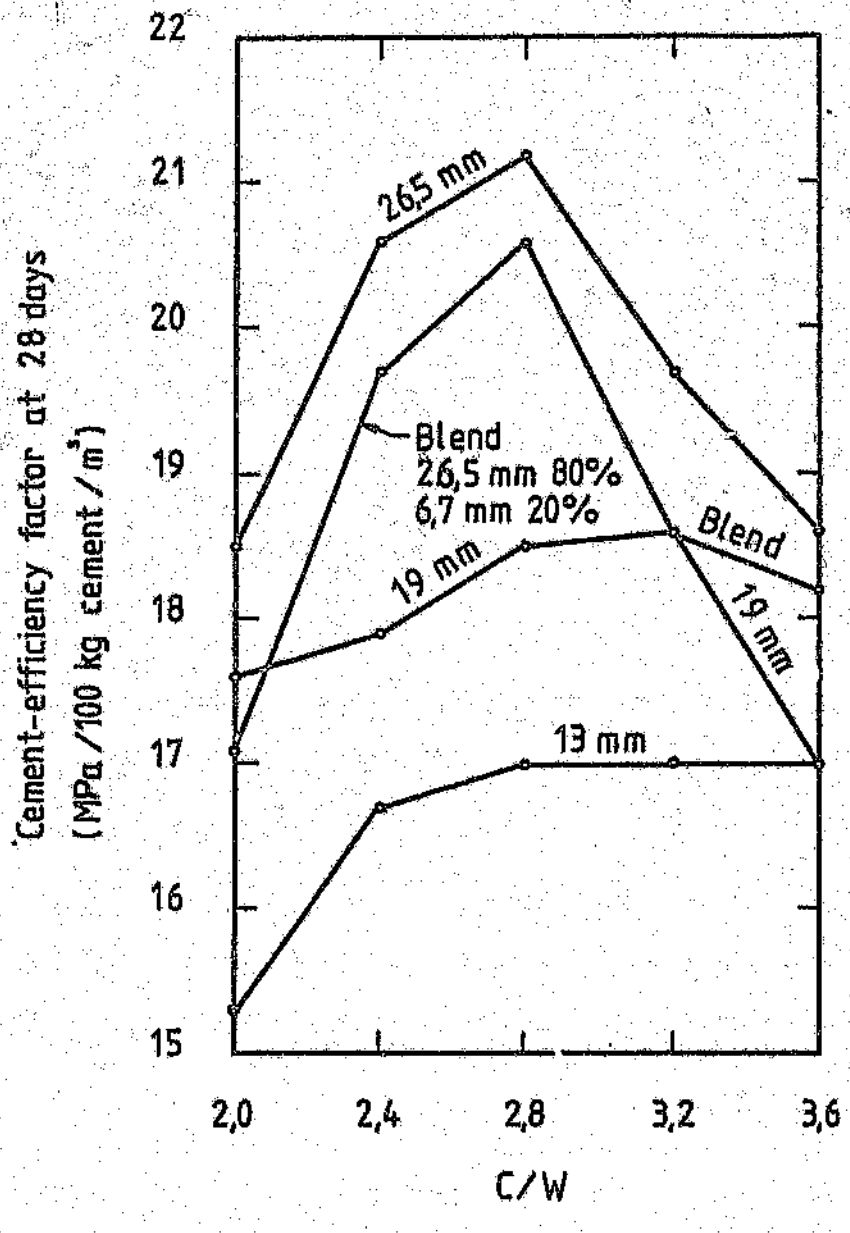


FIGURE 8.6: Cement efficiency factor at 28 days versus cement:water ratio for concrete made with stone of different sizes

If the effect of coarse-aggregate size on strength is considered, the following factors are involved.[2.16]

Larger sizes make the mix dimensionally more heterogeneous and particles are more likely to include critical flaws; both factors tend to result in lower strengths.

Larger sizes result in lower water requirements which should, by reducing overall porosity, result in higher strengths.

In addition, substituting a larger size for a smaller size normally changes the total surface area of aggregate and may affect stresses at paste–aggregate interfaces.

The same substitution also normally changes the total volume of aggregate. Because of differences in elastic stiffness between aggregate and paste, such a change in volume ratios can also affect interface stresses and local stresses in the paste and aggregate.

To investigate the effects of changing the surface area and volume fraction of aggregate on interface shear stress a mathematical model was developed. This model is described below.

8.3.5.2.1 Model for interface shear stress

The model is based on a composite prism-shaped element. The prism consists of a hard material, representing the aggregate, and a soft material, representing the paste, as shown in Figure 8.7.

It is assumed that:

- A uniform stress is applied at the top and bottom surfaces of the prism.
- No slip occurs between hard and soft materials.

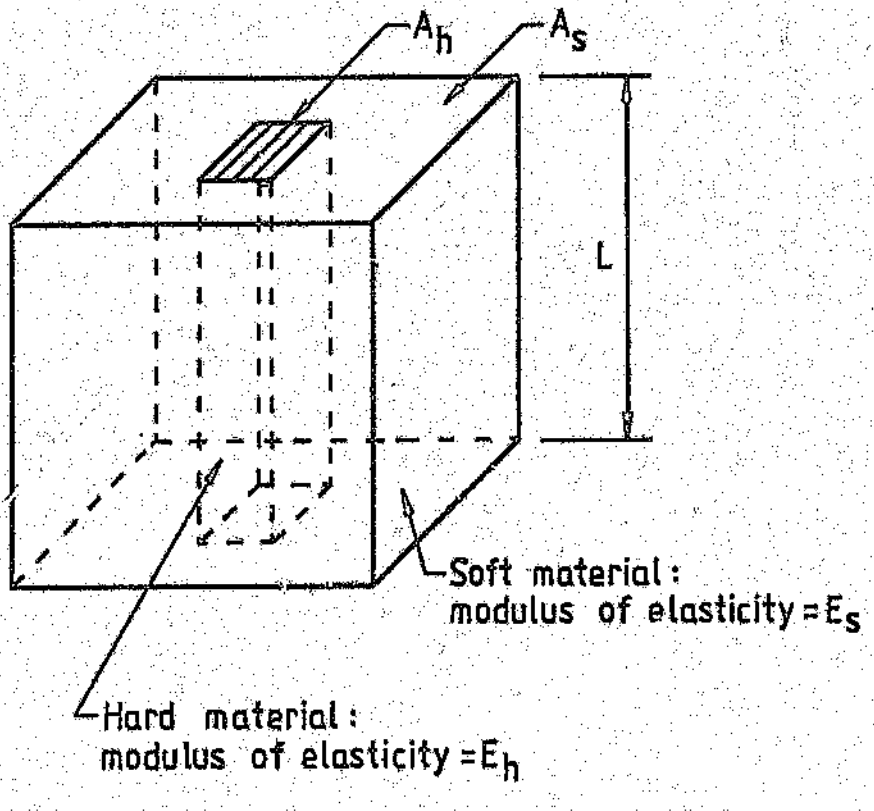


FIGURE 8.7: Isometric view of composite prism containing prisms of hard and soft phases

If the prism is subject to a stress of f on the top and bottom faces as shown in Figure 8.8 the prism shortens by ΔL.

$$\Delta L = \frac{fL\,(A_h + A_s)}{E_h A_h + E_s A_s} \qquad \text{.....(8.1)}$$

(The effective modulus of elasticity of the composite prism, when loaded vertically, is $[E_h A_h + E_s A_s] \div [A_h + A_s]$.)

Consider the hard material as shown in Figure 8.9.

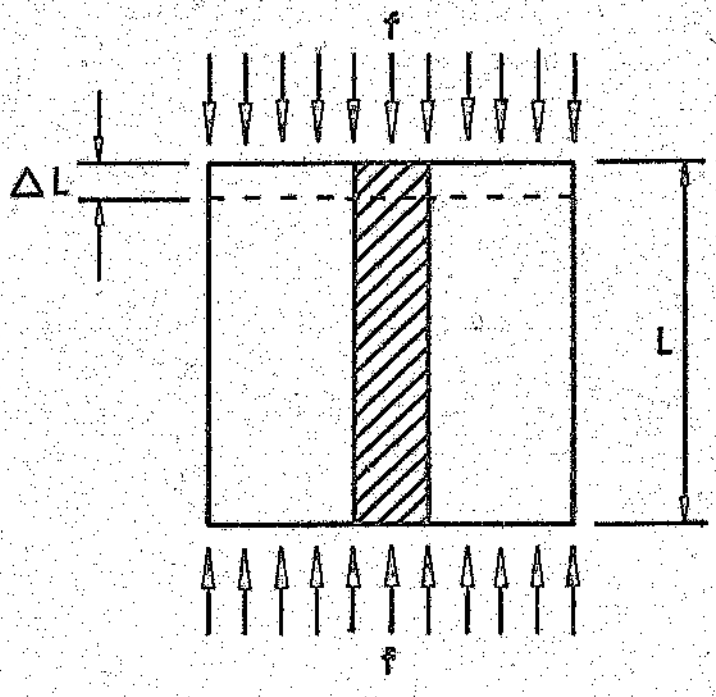


FIGURE 8.8: Shortening of composite prism under stress f

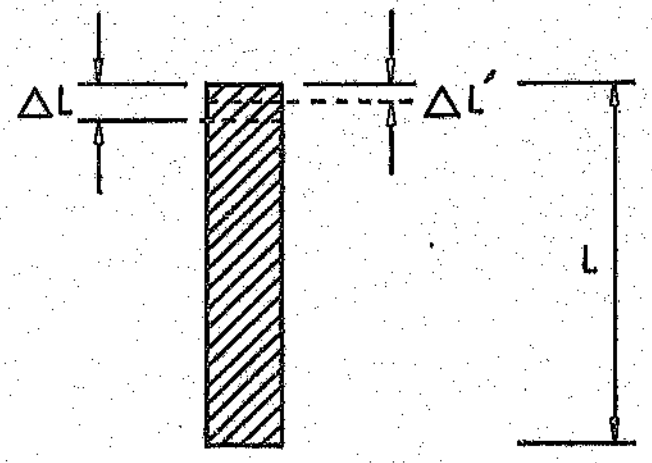


FIGURE 8.9: Shortening of hard prism

ΔL is the actual shortening in the composite prism.

$\Delta L'$ is the shortening that the hard material would undergo if in isolation.

$$\Delta L' = \frac{fL}{E_h} \quad \text{.....(8.2)}$$

The difference $\Delta L - \Delta L'$ is caused by a force on the interface between hard and soft materials.

$$\Delta L - \Delta L' = \frac{fL\,(A_h + A_s)}{E_h A_h + E_s A_s} - \frac{fL}{E_h} = fL\left[\frac{A_h + A_s}{E_h A_h + E_s A_s} - \frac{1}{E_h}\right] \quad \text{.....(8.3)}$$

Let F_i be the force on the interface which causes the difference between ΔL and $\Delta L'$.

$$F_i = E_h A \left[\frac{\Delta L - \Delta L'}{L}\right]$$

From equation 8.3:

$$F_i = \frac{E_h A_h fL}{L}\left[\frac{A_h + A_s}{E_h A_h + E_s A_s} - \frac{1}{E_h}\right]$$

Let f_i be the average stress on the interface and A_i be the area of the interface.

$$f_i = \frac{F_i}{A_i}$$

$$\therefore f_i = \frac{E_h A_h f}{A_i}\left[\frac{A_h + A_s}{E_h A_h + E_s A_s} - \frac{1}{E_h}\right] \quad \text{.....(8.4)}$$

Equation 8.4 can be used to investigate the effect on interface shear stress caused by changes in the volume fraction of hard material and changes in interface area. A_i is not known directly but can be assumed to be proportional to the total surface area of the hard material. Let $A_i = k_1$ x surface area of hard material. Similarly A_h and A_s can be taken as proportional to the volume fractions of hard and soft materials respectively. Let $A_h = k_2$ x volume fraction of hard material and $A_s = k_2$ x volume fraction of soft material. As stated above, hard material represents aggregate and soft material represents paste.

Compare a mix made with 26,5–mm stone with a mix made with 13,2–mm stone. Aggregate particle shape is not known but for the sake of comparison, assume particles to be cubical. Taking proportions of actual mixes (at C/W = 2,8 from Figure 6.15).

		Mix made with 26,5–mm stone	Mix made with 13,2–mm stone
Total surface area of aggregates per m^3 (m^2)		1 772	1 957
Volume fraction of aggregates		0,700	0,653
Therefore:	A_h	0,700 k_2	0,653 k_2
	A_s	0,300 k_2	0,347 k_2
	A_i	1 772 k_1	1 957 k_1

Assume E_h = 100 GPa)
) (See section 9.12.)
and E_s = 19 GPa)

Let $f_{i(26,5)}$ and $f_{i(13,2)}$ be the interface stresses in concrete made with 26,5–mm stone and 13,2–mm stone respectively.

It is not necessary, or indeed possible, to calculate actual stresses. But equation 8.4 can be used to calculate the ratio of $f_{i(26,5)}$ to $f_{i(13,2)}$.

From equation 8.4:

$$f_{i(26,5)} \div f_{i(13,2)} = 0{,}000127 \div 0{,}000131 = 0{,}97$$

In other words, concrete made with the larger stone has the lower interface shear stress and could consequently be expected to have a higher compressive strength than concrete made with the smaller stone. The difference in interface stress according to the model is 3 % of the higher strength.

Relevant properties of the two concretes were (from Appendix A8.1):

Size of stone (mm)	Water content (ℓ/m^3)	28-day cube strength (MPa)
26,5	160	95
13,2	185	88

The difference in 28-day cube strength is 7 MPa. The model accounts for a difference of 3 % of 95 say 3 MPa. To allow for the effect of the difference in water content, apply the rule-of-thumb that 1 ℓ increase in water content reduces strength by 0,2 MPa (see section 7.2.3). Water-content effect is therefore 0,2 x 25 = 5 MPa.

Together the model and water-content effect therefore account for 8 MPa which is similar to the strength difference measured.

It is extremely doubtful whether similar calculations would often work out as neatly as the example given. It is perhaps significant that the effect of the difference in water

content is the dominant factor. But the model does appear to be realistic in direction and magnitude.*

8.3.5.2.2 Strength and CEF

CEF illustrates, more dramatically than strength, differences in the performance of the various stone sizes. CEF is a function of strength and the reciprocal of cement content. At a given C/W, cement content is directly determined by water requirement; CEF is therefore a function of strength and the reciprocal of water requirement. Because strength tends to increase as water content decreases, CEF magnifies the differences in performance of the three stone sizes. In Figure 4.7, CEF at 28 days is greatest for the largest stone and vice versa at all values of C/W (although CEF is the same for 19–mm and 13,2–mm stone at C/W = 3,6).

Based on the above data, the most favourable stone size, of the three considered, for high–strength concrete is 26,5 mm and the least favourable 13,2 mm.

Conclusions

Compressive strength and CEF of concretes made for the investigation tended to increase with increasing stone size up to 26,5 mm. Only one aggregate type (andesite) was used for the investigation and it is reasonable to ask whether the tendency observed is perhaps specific to aggregate type. But work done independently by a colleague in Port Elizabeth showed that 19–mm stone always gave better strengths than 13,2–mm stone. Three types of stone were used in the study: quartzite pebbles, crushed quartzite, and dolerite.[8.7] These results are in contrast to references in the literature.

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* In the above calculation, all of the aggregate was taken as the hard substance and the paste as the soft substance in the composite. An alternative approach is to regard only the stone as the hard substance and the mortar (sand + paste) as the soft substance. In this case the modulus of elasticity of the mortar would be calculated according to Counto's model. This approach was in fact tried and produced quite meaningless results, mainly because the mortars in the two concretes differ only marginally while the surface areas of stone differ substantially.

Although concretes made with stone of different sizes, blended to improve packing capacity, performed better that a nominally single–sized stone of the same FM as the blend, they did not perform as well as the larger stone used on its own.

A tentative conclusion is that to make high–strength concrete with South African materials the optimum stone size would be 26,5 mm. Other sizes, in descending order of preference are: blend of 26,5 mm and 6,7 mm; 19 mm; 13,2 mm. The effectiveness of larger sizes, for example 37,5 mm, could be determined by comparative tests.

8.4 Performance of various cements

The investigation of the performance of concretes made with various cements was carried out in two phases:

Rapid–hardening portland cement (RHPC) and blends of RHPC with various cement extenders were compared with each other.

Different RHPCs were compared with each other.

Note

Although this chapter deals with the strength of hardened concrete, water requirement is discussed because it can have an indirect effect on strength and significantly influences the cement–efficiency factor at a given cement:water ratio.

8.4.1 RHPC and blends of RHPC and cement extenders

8.4.1.1 Cements and extenders

The following cement and cement–extenders were used:

- RHPC only
- 95 % RHPC + 5 % condensed silica fume (CSF)
- 90 % RHPC + 10 % CSF
- 85 % RHPC + 15 % fly ash (FA)
- 75 % RHPC + 25 % ground granulated blastfurnace slag (GGBS).

Reasons for using the above blend ratios were:
CSF, which is almost pure silica in extremely finely divided form, is a highly reactive pozzolan. It is generally regarded as a strength–enhancing extender and cement replacements of between 5 % and 10 % are usual. Higher replacement percentages tend to increase water requirement with consequent increases in paste content and reduced economy.

FA and GGBS are relatively impure materials with limited reactivity.[8.8] Their performance regarding strength of concrete at 28 days is typically inferior to that of ordinary portland cement when used at normal replacement levels of 30 % and 50 % respectively.[8.9] When used to partially replace rapid–hardening portland cement they could therefore be expected to perform relatively less effectively. Blend ratios of 85:15 and 75:25 for FA and GGBS respectively, ie half of the usual replacement levels, were used. It was felt that these levels would indicate the effect of these extenders without a severely adverse effect on performance.

Materials used were:

- RHPC – Type "J" (An analysis of the cement is shown in Table 8.5 below.)
- CSF
- FA
- GGBS

Obtained from local suppliers

8.4.1.2 Tests

Mixes at cement:water ratios (where "cement" is total of cement plus cement extender) of 2,0, 2,4, 2,8, 3,2 and 3,6 were made up with RHPC only and each blend of RHPC and cement extender. Mixes were all proportioned according to the method discussed in section 6.4 and satisfactory workability achieved in all cases. Slump was approximately 75 mm unless stated otherwise. A superplasticizer in liquid form based on sulphonated naphthalene formaldehyde (SNF) type "C" was used in all mixes at a

dosage of 2 ℓ per 100 kg of cement or cement plus cement extender. The admixture was 70 % water by volume.

Aggregates were dolomite from Olifantsfontein; properties are shown in Table 8.3

Cubes for each mix were made and tested in accordance with SABS Method 863.[8.17]

8.4.1.3 Results

Mix proportions and results of strength tests are given in Appendix A8.2

Note that in the results: water content quoted includes water in the admixture; cube strengths are an average of three cubes with the range less than 15 % of the average unless otherwise stated; cement-efficiency factor (MPa/100 kg cement/m^3) is based on the total mass of PPC and cement extender in the specific mix.

Results are shown graphically in Figures 8.10 to 8.16.

Table 8.3: Gradings and densities of dolomite aggregates

Aggregate size	26,5 (mm)	6,7 (mm)	Sand
SABS sieve size (mm)	Percentage passing		
37,5	100		
26,5	99		
19,0	37		
13,2	4		
9,5	2	100	
6,7	1	82	
4,75	1	38	100
2,36	1	1	77
1,18			37
0,60			30
0,30			21
0,15			16
0,075	0,4	0,3	13,8
CBD (kg/m³)**	*1 660	*1 530	–
Relative density	2,88	2,88	2,85
Solids content in compacted state (%)	*58	*53	–
FM	7,59	5,61	3,08

* A blend of 75 % 26,5–mm stone and 25 % 6,7–mm stone had a CBD of 1 850 kg/m³ and a solids content of 64 % in the compacted state.

** Compacted bulk density.

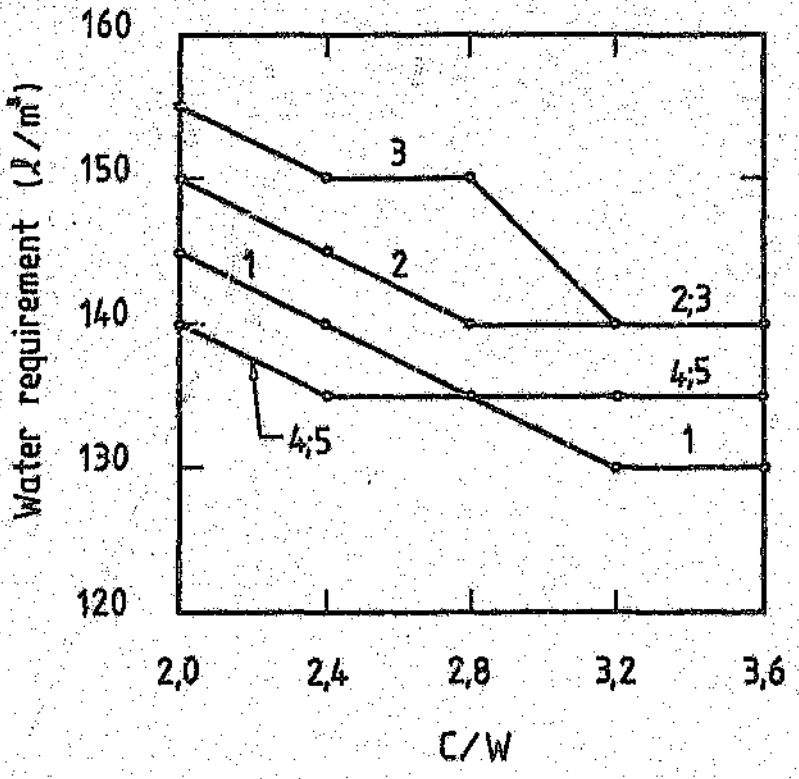


FIGURE 8.10: Water requirement (to nearest 5 ℓ/m³) for 75–mm slump versus cement:water ratio for concretes made with a blend of 26,5–mm and 6,7–mm stone, sulphonated naphthalene formaldehyde admixture type "C", and various cementitious materials:

1. RHPC only
2. RHPC/CSF 95/5
3. RHPC/CSF 90/10
4. RHPC/FA 85/15
5. RHPC/SL 75/25

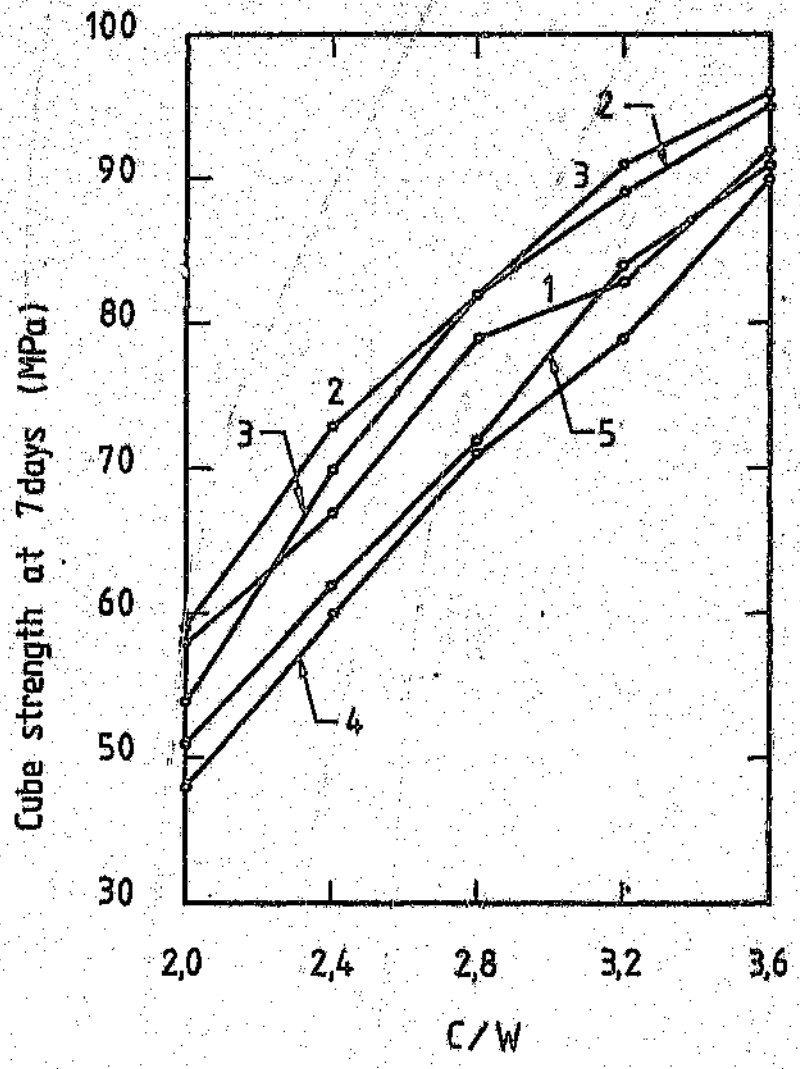


FIGURE 8.11: Cube strength at 7 days versus cement:water ratio for concretes made with a blend of 26,5-mm and 6,7-mm stone, SNF admixture type "C" and various cementitious materials

1. RHPC only
2. RHPC/CSF 95/5
3. RHPC/CSF 90/10
4. RHPC/FA 85/15
5. RHPC/SL 75/25

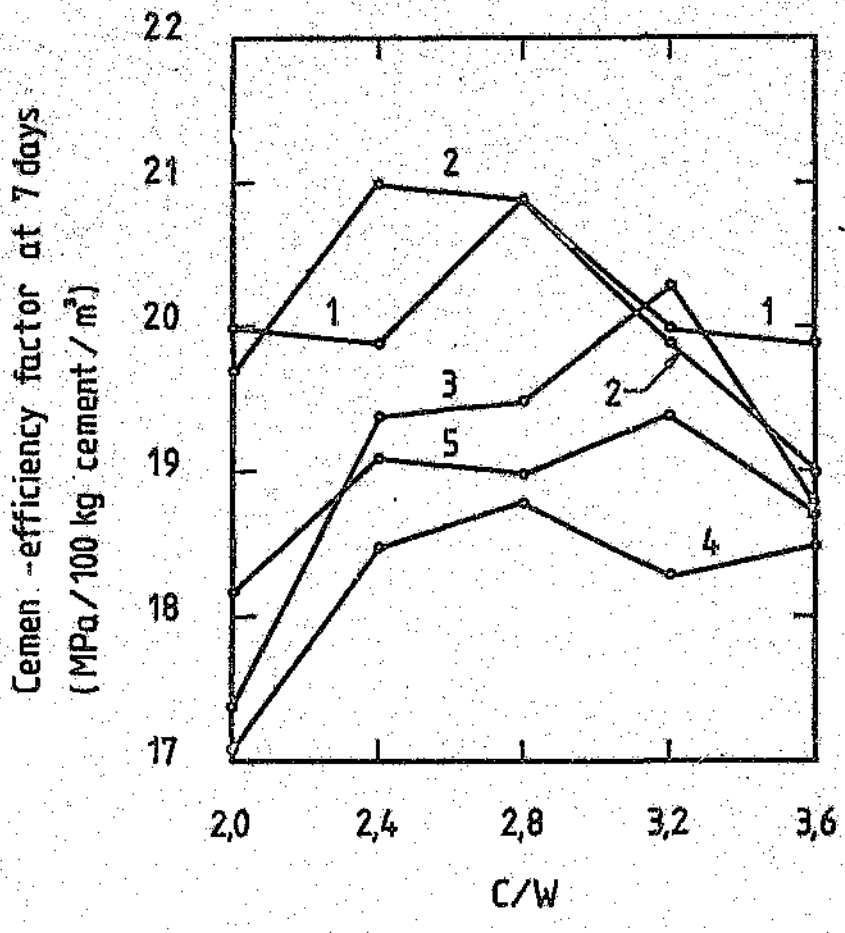


FIGURE 8.12: Cement–efficiency factor at 7 days versus cement:water ratio for concretes made with a blend of 26,5–mm and 6,7–mm stone, SNF admixture type "C" and various cementitious materials

1. RHPC only
2. RHPC/CSF 95/5
3. RHPC/CSF 90/10
4. RHPC/FA 85/15
5. RHPC/SL 75/25

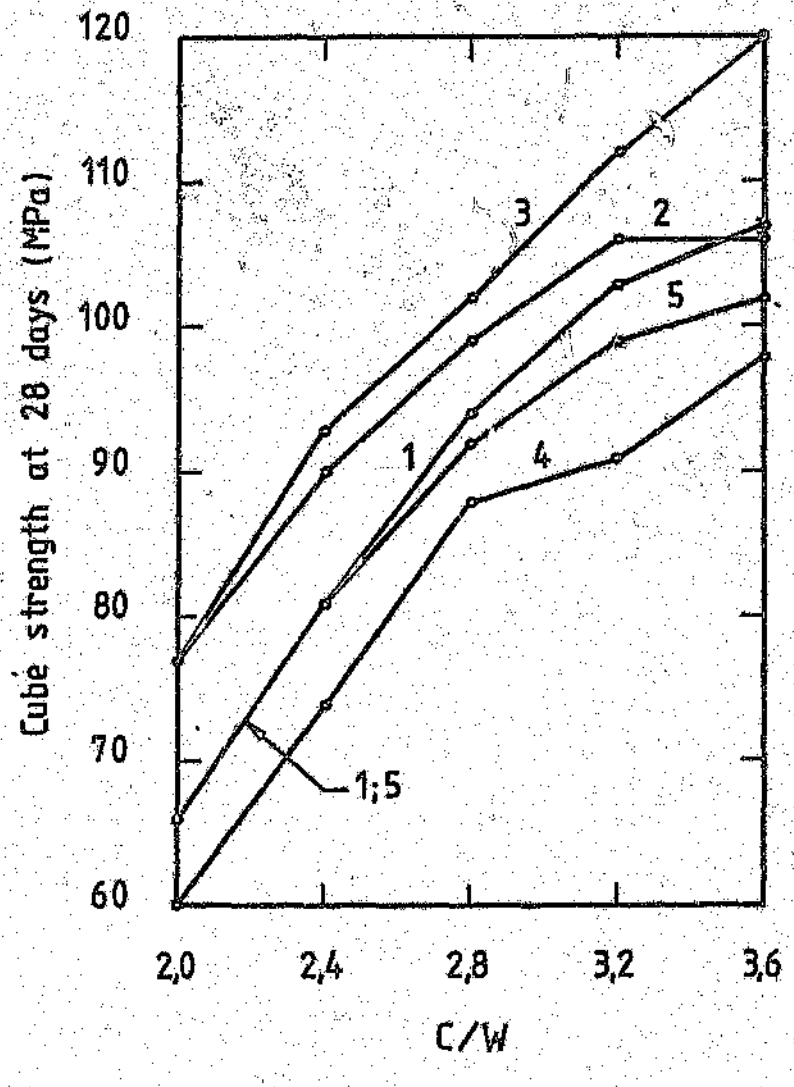


FIGURE 8.13: Cube strength at 28 days versus cement:water ratio for concretes made with a blend of 26,5–mm and 6,7–mm stone, SNF admixture type "C" and various cementitious materials

1. RHPC only
2. RHPC/CSF 95/5
3. RHPC/CSF 90/10
4. RHPC/FA 85/15
5. RHPC/SL 75/25

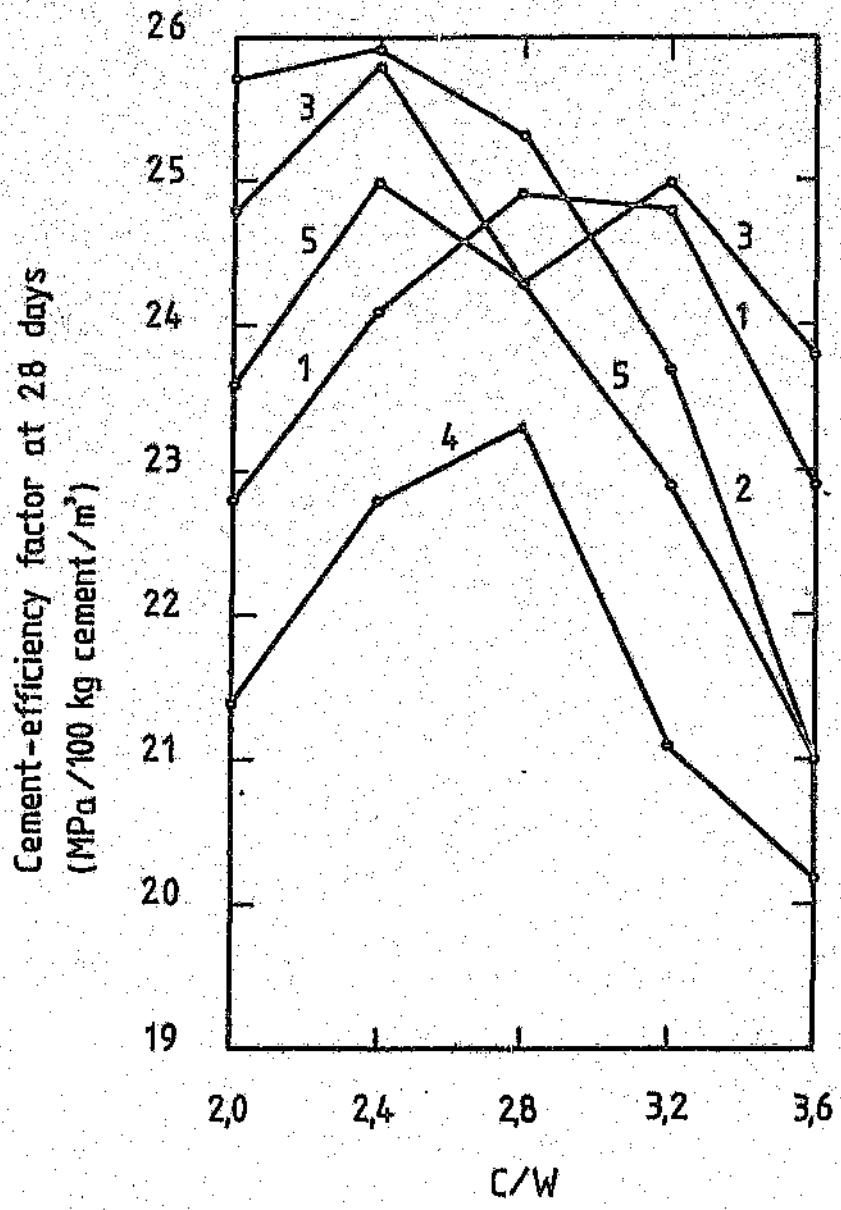


FIGURE 8.14: Cement–efficiency factor at 28 days versus cement:water ratio for concretes made with a blend of 26,5–mm and 6,7–mm stone, SNF admixture type "C", and various cementitious materials

1. RHPC only
2. RHPC/CSF 95/5
3. RHPC/CSF 90/10
4. RHPC/FA 85/15
5. RHPC/SL 75/25

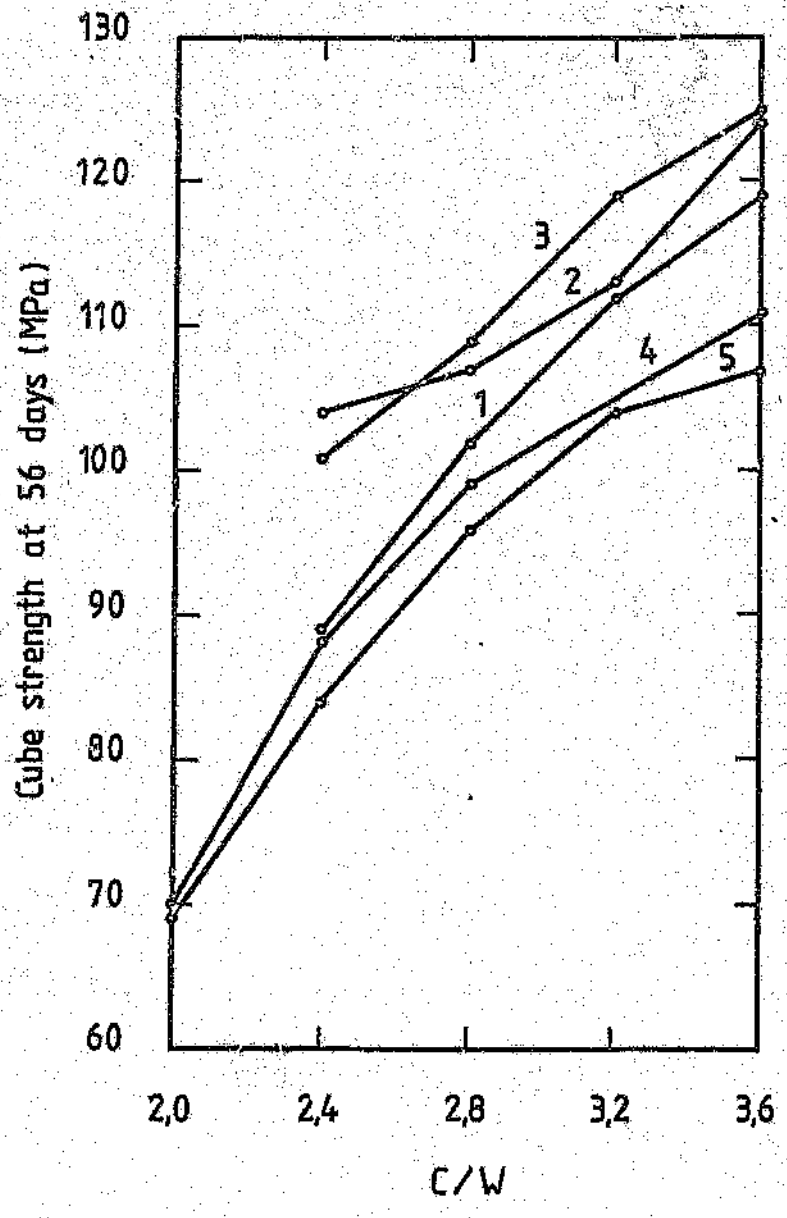


FIGURE 8.15: Cube strength at 56 days versus cement:water ratio for concretes made with a blend of 26,5–mm and 6,7–mm stone, SNF admixture type "C" and various cementitious materials

1. RHPC only
2. RHPC/CSF 95/5
3. RHPC/CSF 90/10
4. RHPC/FA 85/15
5. RHPC/SL 75/25

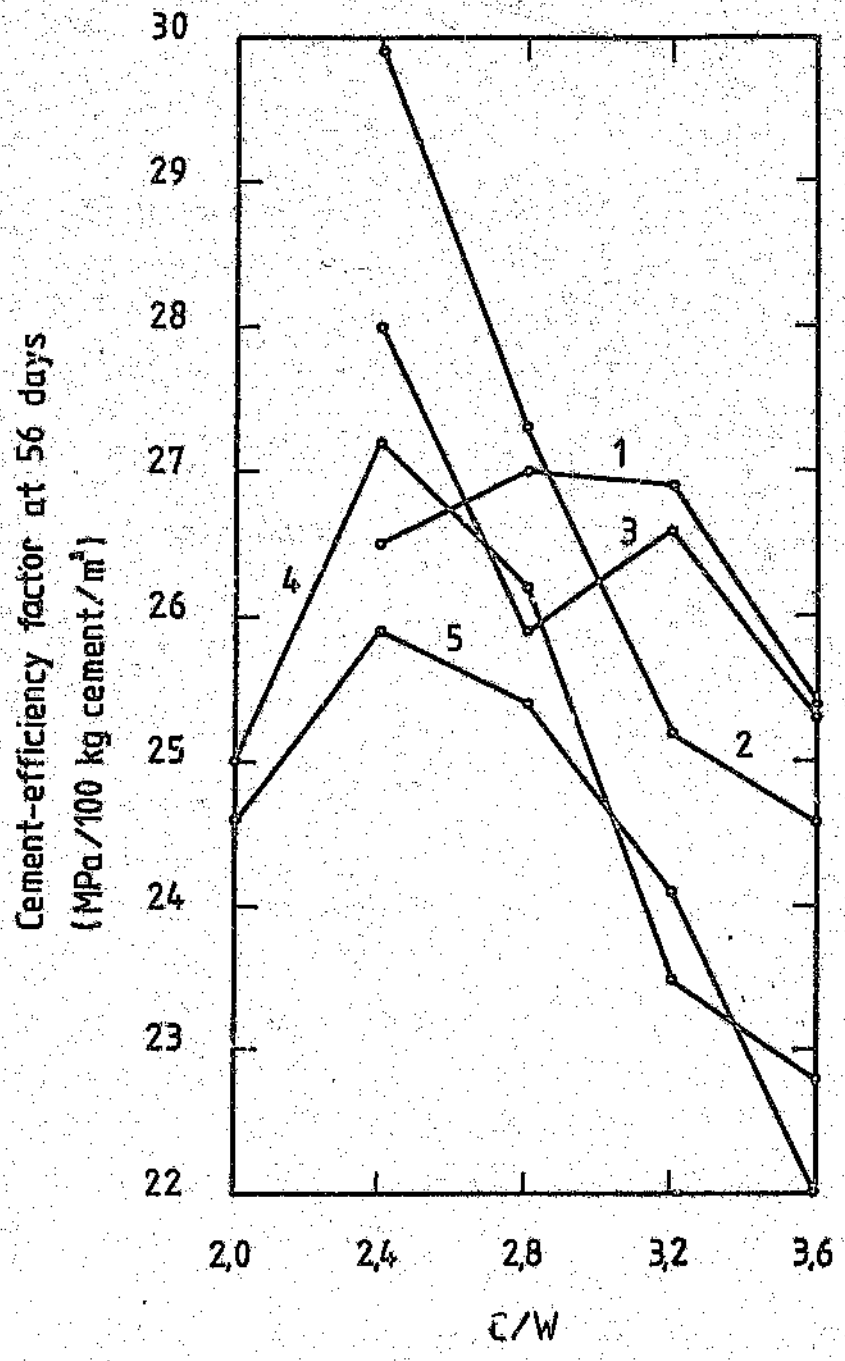


FIGURE 8.16: Cement–efficiency factor at 56 days versus cement:water ratio for concretes made with a blend of 26,5–mm and 6,7–mm stone, SNF admixture type "C" and various cementitious materials

1. RHPC only
2. RHPC/CSF 95/5
3. RHPC/CSF 90/10
4. RHPC/FA 85/15
5. RHPC/SL 75/25

Average rates of strength development, calculated from the results, are shown in Table 8.4.

Table 8.4: Average rate of strength development

Cement type	Strength ratios		
	7/28 days	56/28 days	56/7 days
RHPC only	0,87	1,08	1,24
95 % RHPC 5 % CSF	0,83	1,12	1,35
90 % RHPC 10 % CSF	0,78	1,06	1,36
85 % RHPC 15 % FA	0,85	1,14	1,34
75 % RHPC 25 % GGBS	0,78	1,05	1,35

8.4.1.4 Observations

8.4.1.4.1 Water requirement

Figure 8.10 shows the effect of the cement–cement extender on water requirement.

CSF increased water requirement. Although there was little difference between the 5 % CSF mixes and the 10 % CSF mixes, those with the higher CSF content had the higher water requirement. An interesting side effect of the increase in water requirement was that it tended to restore some of the thixotropic behaviour normally associated with concrete that has not been superplasticized.

FA and GGBS reduced water requirement in the leaner mixes but not to any significant degree. There was no apparent difference in workability when FA or GGBS was substituted for RHPC.

8.4.1.4.2 Compressive strength and CEF

Cube strength versus cement:water ratio at 7, 28 and 56 days is shown in Figures 8.11, 8.13 and 8.15 respectively. Cement–efficiency factor versus cement:water ratio at 7, 28 and 56 days is shown in Figures 8.12, 8.14 and 8.16 respectively.

In the following discussion, for the sake of clarity and brevity, mixes containing an extender are identified only by the type of extender and its percentage of the total mass of RHPC and extender. 25 % GGBS therefore indicates concrete made with cementitious material consisting of 75 parts of RHPC and 25 parts of GGBS by mass. Concrete made with RHPC only is referred to as 100 % RHPC.

General trends of strength and CEF were as follows:

At 7 days, strengths in decreasing order were: 5 % and 10 % CSF mixes, 100 % RHPC, 25 % GGBS, and 15 % FA. CEFs in decreasing order were: 5 % CSF, 10 % RHPC, 10 % CSF and 25 % GGBS overlapping, and 15 % FA.

At 28 days, strengths of 10 % CSF were highest followed by 5 % CSF, 100 % RHPC, 25 % GGBS, and 15 % FA. CEFs showed some overlapping with the best performances generally from the 5 % and 10 % SF mixes. Curves for 100 % RHPC and 25 % GGBS cross at C/W = 2,7 with GGBS mixes superior at lower values of C/W and vice versa. The 15 % FA mixes gave lowest CEFs at high C/Ws.

At 56 days, the CSF mixes gave the highest strengths followed by 100 % RHPC and then 15 % FA and 25 % GGBS. The margin between the CSF mixes and the others was greatest at the lower values of C/W. At C/W = 3,6, CSF mixes and the 100 % RHPC mix had similar strengths. CEFs were highest for the CSF mixes up to C/W about 2,8. At higher C/W values the best performer was 100 % RHPC with 15 % FA and 25 % SL showed significant decreases in performance with increasing C/W.

It is interesting that at all three ages the CSF mixes gave the most erratic results. In comparison, results obtained with other cements generally plotted fairly closely to smooth best-fit curves. The CSF used was supplied in the densified state (ie pelletized) and it is possible that the pellets did not break down and mix evenly in the mixer. This theory is supported by results of tests done at Portland Cement Institute to compare the bleeding of mortars made with densified and undensified CSF. These

tests showed that mortars made with undensified CSF bled consistently less than those made with densified CSF.

The following points may be noted from Table 8.4.

- With the exception of the 100 % RHPC mixes, the 56d/7d strength ratios were virtually the same.

- The 15 % FA mixes, followed closely by the 5 % CSF mixes, showed the highest 56d/28d strength ratios. This however does not necessarily indicate the best potential for further strength development (under favourable conditions) because trends cannot be extrapolated to later ages.

8.4.1.5 Conclusions

Based on 56-day results, it appears that the cement or cement extenders showing the greatest overall promise are 100 % RHPC and the 5 % and 10 % CSF mixes. If the erratic nature of the CSF mixes is considered unacceptable, then the best cement is 100 % RHPC.

In practice however, conditions for long-term curing are likely to be less than favourable and an assessment based on 28-day results would be more realistic. In this case the CSF mixes would be the first choice. Probably a cement replacement level between 5 % and 10 % would give good results.

It should be borne in mind that the above comparisons are based on results of tests using only one sample of RHPC. Other RHPCs could give different results as could another sample of RHPC taken from the same source at a different time.

In selecting the cementitious component of high-strength concrete it is recommended that trial mixes be made with the best available RHPC using 100 % RHPC and a blend

consisting of 92,5 % F/HPC and 7,5 % CSF. The choice of material can then be based on the strength and CEF results.

CEFs achieved in the investigation are compared in section 8.10 with those reported in the literature.

8.4.2 A comparison of various RHPCs

The object of this part of the work was to investigate and compare the performance of different, locally available, rapid–hardening portland cements in concrete and, if possible, to establish relationships between cement characteristics and performance.

8.4.2.1 Cements used

Samples of rapid–hardening cement were obtained from various local sources and are coded H, J, L, R, U and D.

Chemical analyses, and in some cases particle–size determinations, were provided by the manufacturers. Specific surface areas were measured in the Portland Cement Institute laboratory. The above are shown in Table 8.5.

8.4.2.2 Tests

Concrete mixes at cement:water ratios of 2,0, 2,4, 2,8, 3,2 and 3,6 were made using the various cements to study the effect of cement source on the properties of fresh and hardened concretes. Mixes were all proportioned according to the method discussed in section 6.4. Satisfactory workability was achieved in all cases. Slump was 75 mm plus or minus 20 mm for all mixes. A superplasticizer in liquid form based on sulphonated naphthalene formaldehyde (SNF) type "C" was used in all mixes at a dosage of 2 ℓ per 100 kg of cement. The admixture was 70 % water by volume. Cubes for each mix were made and tested in accordance with SABS Method 863.[8.17]

Aggregates were dolomite from Olifantsfontein; properties are shown in Table 8.3.

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Aggregates were dolomite from Olifantsfontein; properties are shown in Table 8.3.

Table 8.5: Cement characteristics

Cement code	H	J	L	R+	U	D
SiO_2	21,1	21,7	21,3	23,0	22,7	22,5
Al_2O_3	3,7	3,6	4,3	4,2	3,9	4,1
Fe_2O_3	2,3	2,6	2,7	2,7	2,8	2,8
Mn_2O_3	0,9	0,4	0,2	0,3	0,3	0,2
TiO_2	0,3	0,3	0,3	0,3	0,8	0,2
CaO	63,3	63,9	65,5	64,1	64,0	63,8
MgO	4,1	3,3	2,0	1,9	1,9	1,8
P_2O_5	0,06	0,07	–	–	–	–
SO_3	2,40	2,25	2,29	1,7	1,9	1,9
Free CaO	–	–	–	0,8	0,5	0,9
K_2O	0,6	0,5	0,2	–	–	–
Na_2O	0,1	0,0	0,1	–	–	–
Loss on ignition	0,8	1,1	–	1,1	1,4	1,4
Insoluble residue	–	–	–	0,4	0,8	0,7
Specific surface area (Blaine) (m^2/kg)	437	416	411	363	424	424
C_3S	**59,4	**58,0	**62,3	*48,9	**56,4	*53,4
C_2S	**15,8	**18,6	**14,1	*28,0	*20,3	*22,3
C_3A	5,9	5,2	6,8	6,6	5,6	6,1
C_4AF	7,0	7,9	8,2	8,2	8,5	8,5
Mean particle size (µm)	19,3	22,4	–	–	–	–

* Calculated by manufacturer on assumption that insoluble residue is silica in gypsum and must be deducted from stated SiO_2 content.

** By calculation assuming free Ca = 0,7 % (by Bogue formula, see section 7.2).

– Denotes not provided.

\+ Contains some GGBS according to manufacturer.

8.4.2.3 Results

Mix proportions and results of strength tests are given in Appendix A8.3. Note that in the results: water content quoted includes water in the admixture; cube strengths are an average of three cubes with the range less than 15 % of the average unless otherwise stated.

Results are also shown graphically in Figures 8.17 to 8.23.

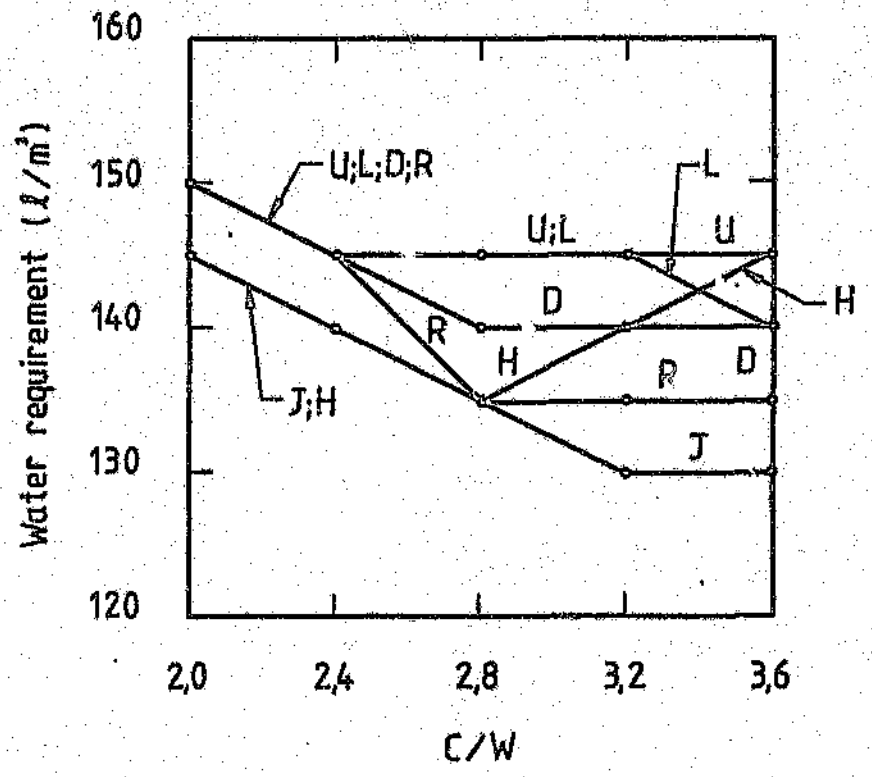


FIGURE 8.17: Water requirement (to nearest 5 ℓ/m³) for 75–mm slump versus cement:water ratio for concretes made with rapid–hardening portland cements from various sources

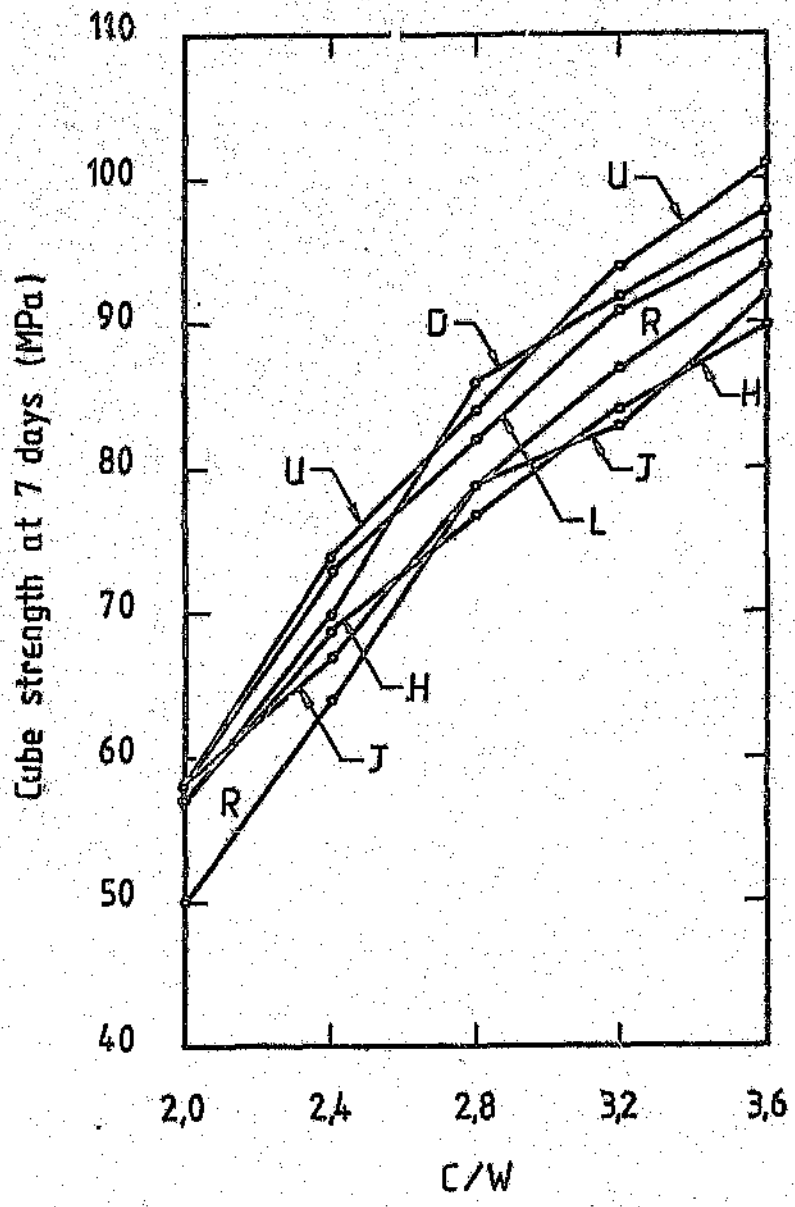


FIGURE 8.18: Cube strength at 7 days versus cement:water ratio for concretes made with RHPCs from various sources

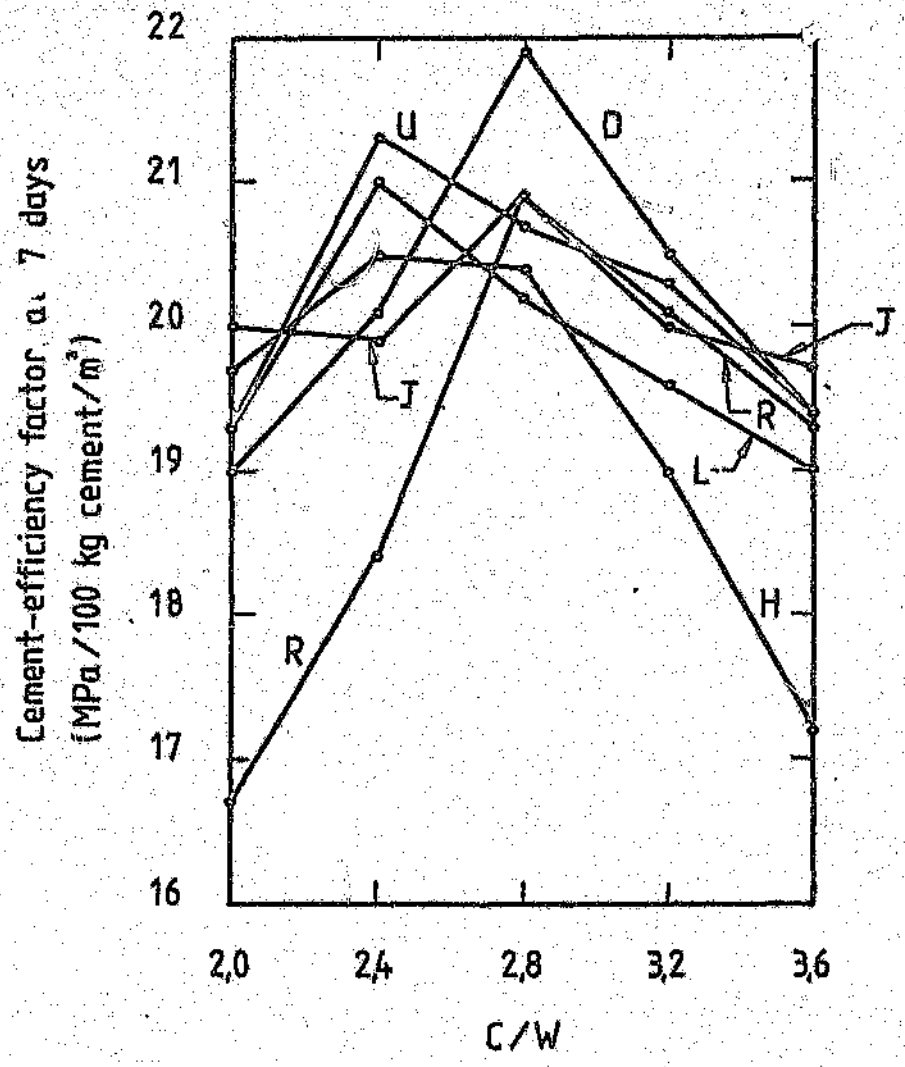


FIGURE 8.19: Cement-efficiency factor at 7 days versus cement:water ratio for concretes made with RHPCs from various sources

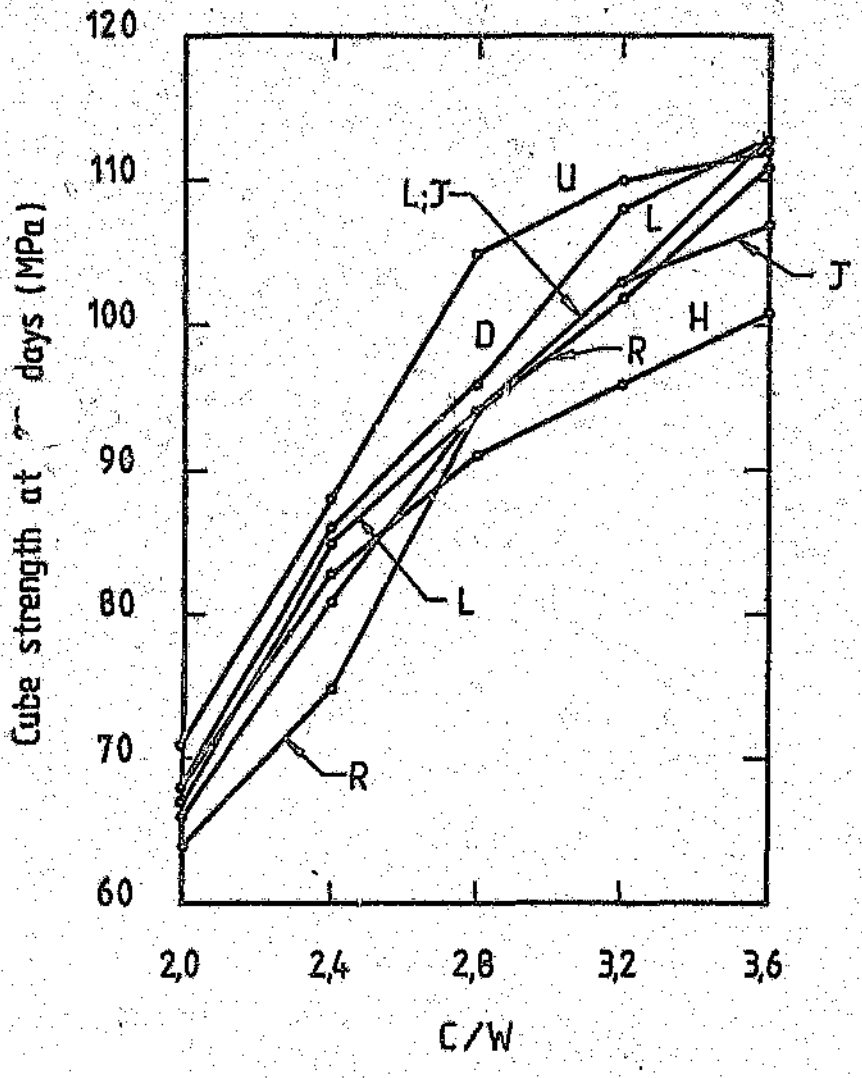


FIGURE 8.20: Cube strength at 28 days versus cement:water ratio for concretes made with RHPCs from various sources

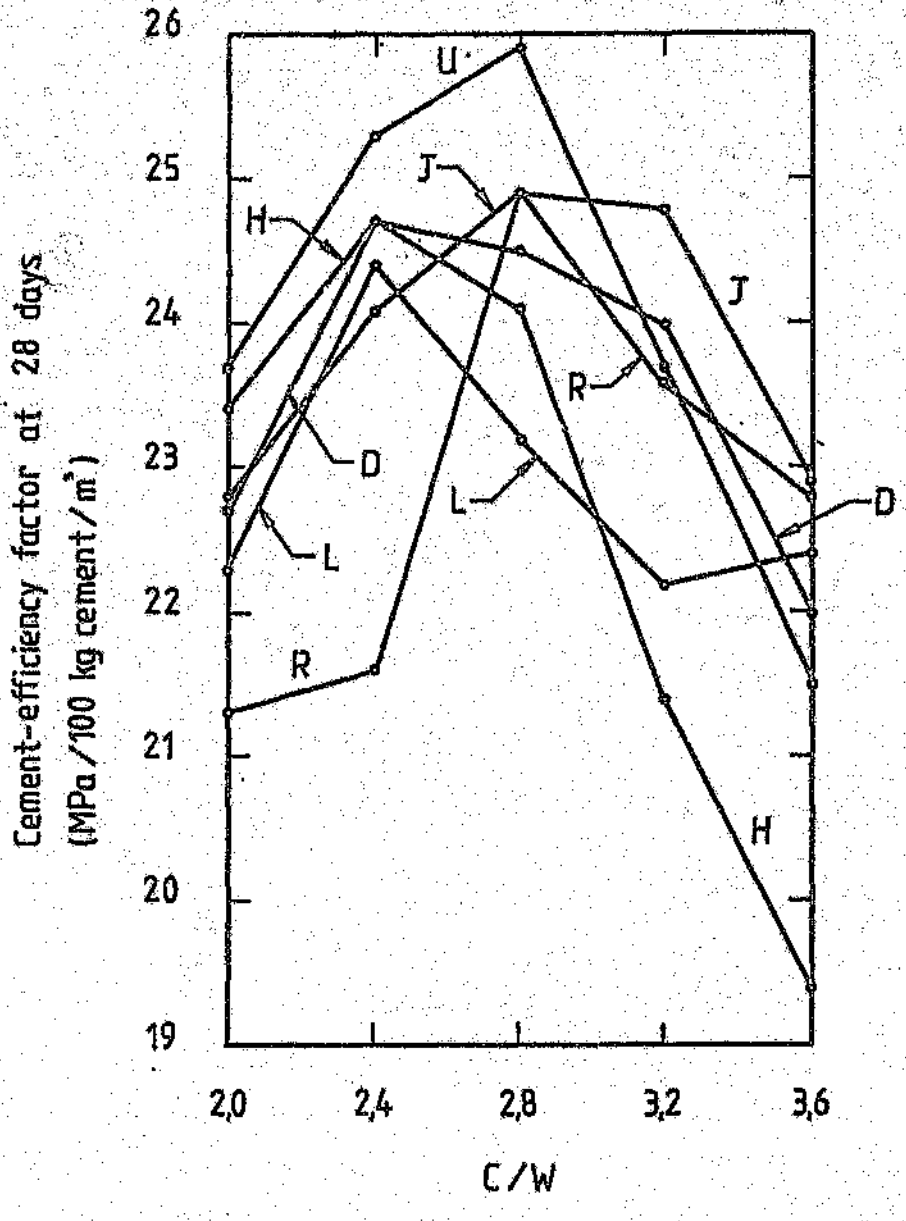


FIGURE 8.21: Cement–efficiency factor at 28 days versus cement:water ratio for concretes made with RHPCs from various sources

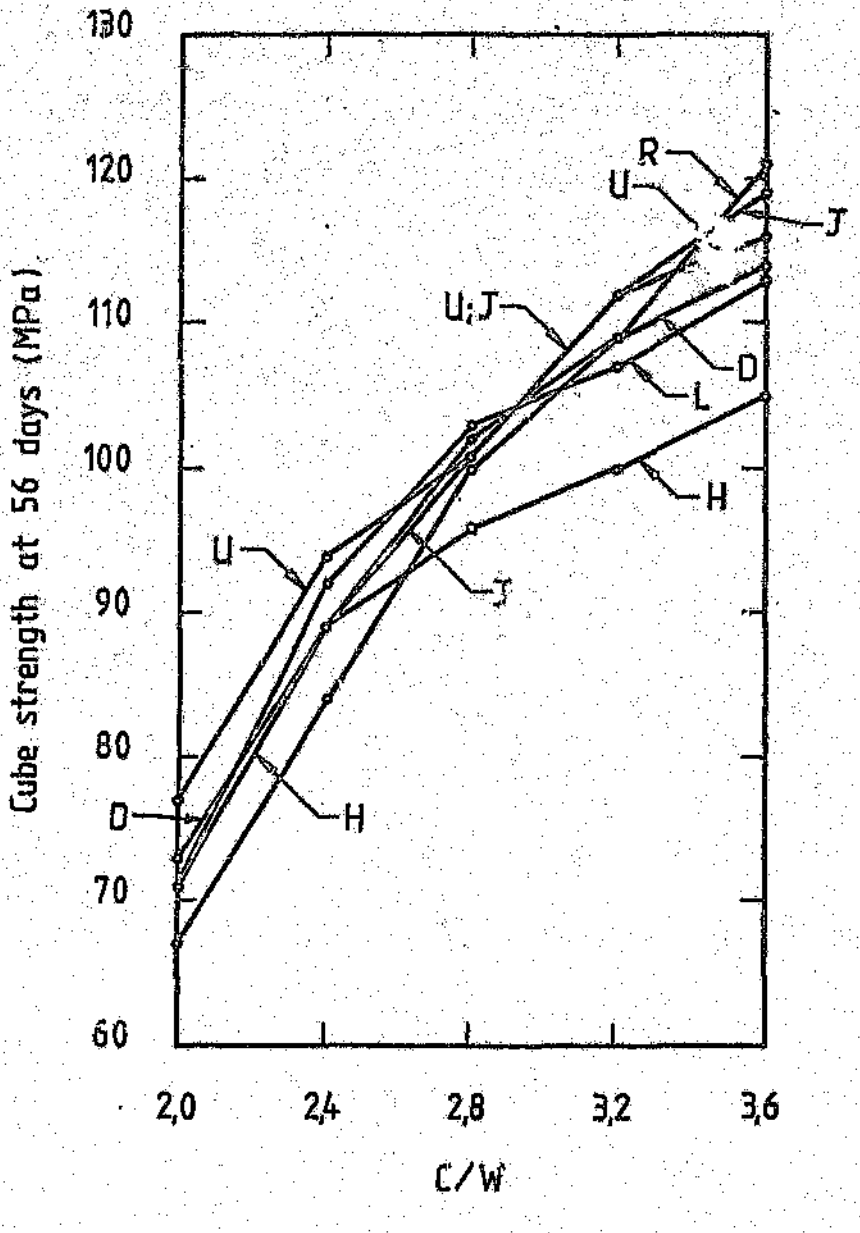


FIGURE 8.22: Cube strength at 56 days versus cement:water ratio for concretes made with RHPCs from various sources (no value for J for C/W < 2,4)

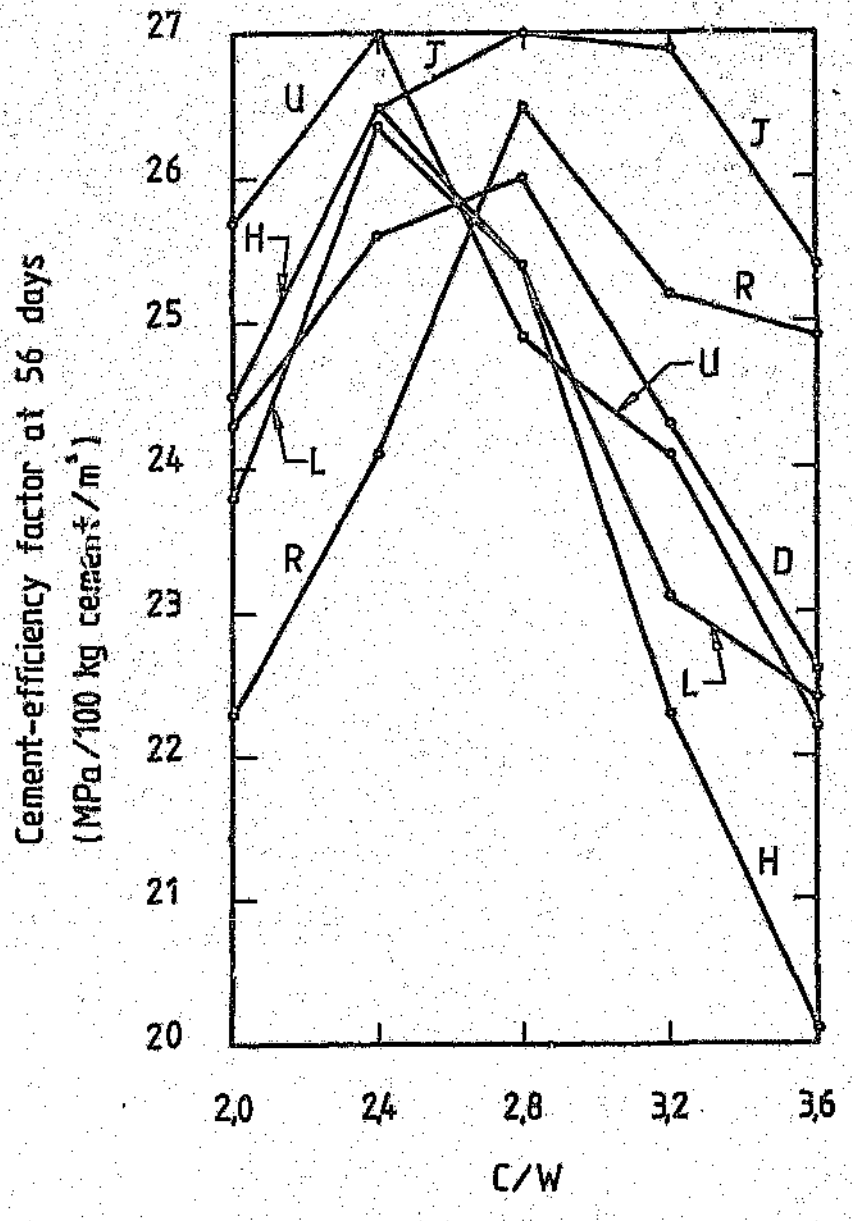


FIGURE 8.23: Cement–efficiency factor at 56 days versus cement:water ratio for concretes made with RHPCs from various sources (no value for J for C/W < 2,4)

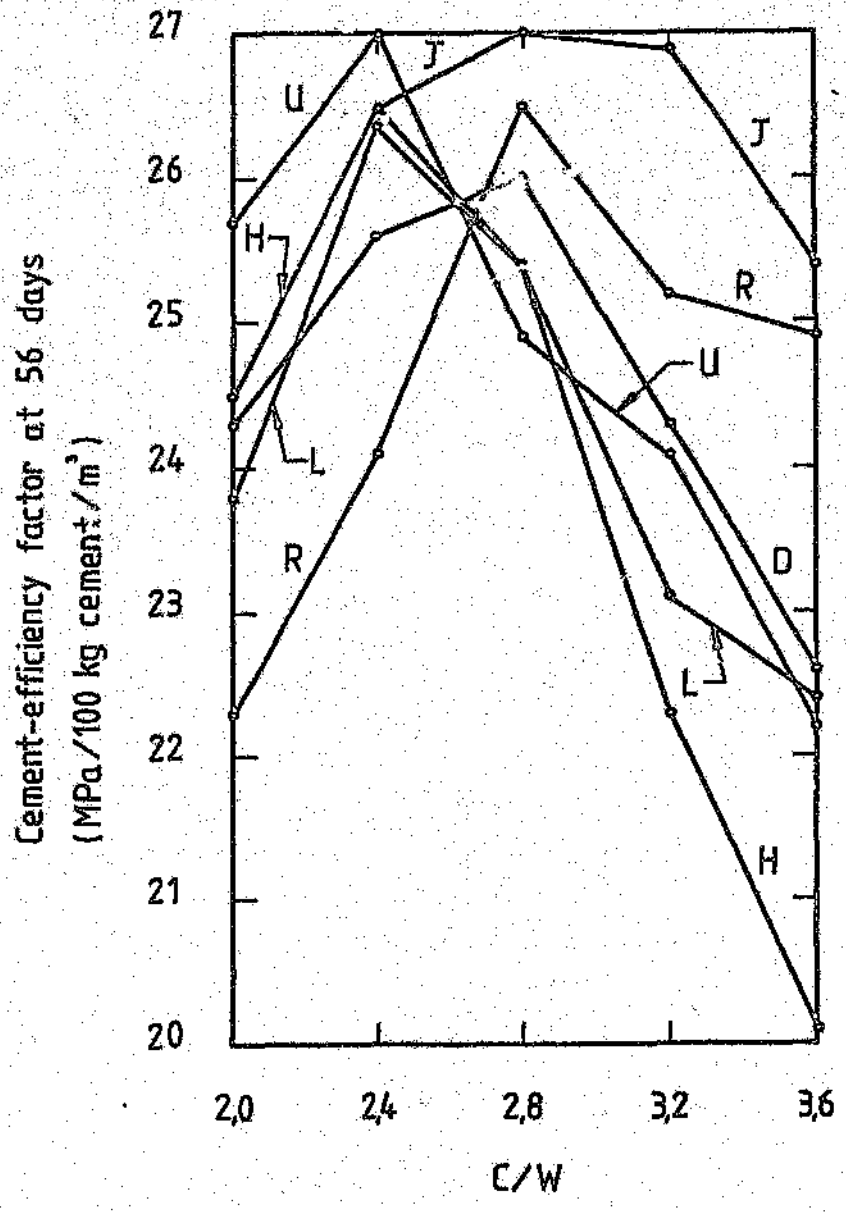


FIGURE 8.23: Cement–efficiency factor at 56 days versus cement:water ratio for concretes made with RHPCs from various sources (no value for J for C/W < 2,4)

8.4.2.4 Observations

8.4.2.4.1 Water requirement of concrete

The effect of cement source on water requirement is shown graphically in Figure 8.17

The figure shows that, with the exception of cement "H", water requirement decreased with increasing C/W, reaching what is probably close to a minimum at about C/W = 3,6. In the case of cement "H", water decreased initially with increasing C/W but reached a minimum at C/W = 2,8 and at C/W = 3,6 was the same as at C/W = 2,0.

Range of water requirements, expressed as a percentage of the median value, was 3,3 % at C/W = 2,0 and 10,9 % at C/W = 3,6.

It has not been possible to isolate any cement characteristics shown in Table 8.5 which correlate with water requirement.

8.4.2.4.2 Strength

7–, 28–, and 56–day strengths are shown plotted against C/W in Figures 8.18, 8.20 and 8.22 respectively.

Range of strength, as a percentage of median value was as follows:

Age (days)	C/W	Range (%)
7	2,0 3,6	15 12
28	2,0 3,6	10 11
56	2,0 3,6	14 14

The cement characteristics given in Table 8.5 do not appear to relate to strength in any quantifiable way. The strength of concrete made with a specific cement is probably the result of a complex combination of factors.

8.4.2.4.3 Cement-efficiency factor

CEF at 7, 28 and 56 days is plotted against C/W in Figures 4.20, 4.22 and 4.24 respectively.

Range of CEF, as a percentage of median value was as follows:

Age (days)	C/W	Range (%)
7	2,0 3,6	18 13
28	2,0 3,6	11 17
56	2,0 3,6	14 23

At 56 days, the most mature age at which tests were done, C/W for maximum CEF ranged between about 2,4 and 3,0. C/W assumed for cement-saturation was 2,5 (see sections 7.2.2 and 7.2.3).

When comparing C/W at maximum CEF with C/W for cement-saturation, it must be borne in mind that the position of maximum CEF may be shifted if water requirement is not constant for the range of C/Ws used. If water requirement decreases with increasing C/W (which was the case in most of the cements investigated), the position of maximum CEF will be shifted to a higher C/W.

There was no correlation between C/W for cement saturation and C/W for maximum CEF for the individual cements but C/Ws for these conditions as a whole were of similar magnitude.

8.4.2.5 Conclusions

The performance of cement in concrete depends on a complex combination of cement characteristics. It has not been possible in this investigation to relate any cement characteristics to performance in concrete.

The large ranges in strengths (about 10 % at 28 days) and CEFs (between 11 % and 17 % at 28 days) do however indicate significant differences in performance of the various cements. It is therefore essential that tests be done with specific cement(s) available for a project. If possible it would be advisable to test a number of cements for suitability when selecting materials for high-strength concrete.

8.4.3 The effect of condensed silica fume on strength of concrete

Condensed silica fume (CSF) is an extremely fine powder: specific surface area is of the order of 20 000 m^2/kg and average particle diameter is about 0,1 µm. Condensed silica fume is also an extremely pure silica in a highly reactive form. The effect of CSF on the strength of concrete is a result of these properties.

The mechanisms by which CSF improves the microstructure of concrete and so enhances strength and durability are described by Mehta and Aitcin[2,10] (see section 2.2.3). Briefly, the very fine particles of CSF segment channels of water flow in fresh concrete and so reduce bleeding and settlement; they also provide numerous nucleation sites for the precipitation of cement hydration products. The homogeneity of the concrete is thus improved by what is known as the "fine-filler effect".

Further densification of spaces between original cement particles occurs when the pozzolanic CSF particles react with calcium hydroxide to form calcium silicate hydrate.

The relative importance, as regards strength, of the fine-filler effect and the pozzolanic reaction was investigated by Mehta.[5,10] He measured strength development rates of superplasticized concrete mixtures containing 10 % addition (by mass of cement) of a CSF, a rice-hull ash, and two types of colloidal carbon. A control mix was made without any addition. In all cases the 1-, 3-, 7-, and 28-d compressive strengths of the mixes with an addition were about 20 % higher than that of the control mix. This was in spite of the fact that colloidal carbon is not pozzolanic. The study indicated that

the strengthening effect associated with pozzolanic reaction was relatively unimportant at ages up to 28 days.[b]

Yogendran et al[2.20] investigated the effects of condensed silica fume on strength of cement pastes. They examined only two water:cement ratios: 0,48, and 0,28 (equivalent to cement:water ratios of 2,1 and 3,6 respectively); "cement" was taken to be the total of portland cement and condensed silica fume. ASTM Type I cement was used throughout. A sulphonated naphthalene formaldehyde superplasticizer was used at a dosage sufficient to provide each mix with the same flow. Flow was measured by filling a small cone placed on a table and then lifting the cone. It was found that for the leaner of the two ratios, substituting condensed silica fume for up to 15 % of the portland cement increased concrete strength at all ages between 3 and 180 days. For the richer of the ratios, substituting condensed silica fume for up to 25 % of the portland cement generally reduced strengths although strength reductions were generally marginal (10 % or less) for substitutions up to 15 %.

Yogendran's work is open to criticism on two counts: it was done on pastes and not concretes; the variable dosage of superplasticizer could favour mixes with the highest substitution levels of condensed silica fume.

References 8.10 and 2.20 above describe work done on concrete and paste respectively. In both references, superplasticizer dosage differed from mix to mix. Because the cost of superplasticizer in a concrete mix forms a substantial fraction of the overall material cost, the results have limited practical value. (See chapter 11.) In the following discussion, the effect of CSF on the strength of concrete is examined on the basis of constant slump and constant superplasticizer dosage.

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[b] The apparently dramatic effect of colloidal carbon is perhaps questionable when it is realized that in the investigation the control mix (ie without additions) did not contain any superplasticizer. The mix which included colloidal carbon contained a high dosage of superplasticizer in order to disperse the carbon. It is possible that some of the beneficial effects associated with the use of colloidal carbon were as a result of the improved dispersion of the cement particles.

Taking a given **concrete** mix as basis, will the addition of a given quantity of CSF be more or less effective, in improving strength, than the addition of the same quantity of portland cement? Alternatively, will the substitution of an amount of CSF for an equal amount of portland cement in a mix increase or decrease the strength of the mix?

Neither question can be answered directly because the use of CSF in lieu of portland cement influences the water requirement of the mix if the slump is to remain constant. The concept of cement–efficiency factor can however be used to allow for changes in water requirement when comparing strength performance.

Figure 8.24 was constructed by drawing smooth best–fit curves through the points on Figure 8.14 for a 5 % substitution of CSF for RHPC. (The points for 10 % substitution are too erratic to permit a meaningful curve to be drawn.)

Let "relative effectiveness" (RE) of CSF be defined as the CSF's notional contribution to strength, relative to the contribution of an equal amount of portland cement at a given age of concrete.

$$\text{Then} \quad RE = \frac{f_b - f_p (1 - B)}{f_p B} \qquad \text{.....(8.5)}$$

where RE = relative effectiveness of CSF
f_b = cube strength of concrete made with a blend of portland cement and CSF
f_p = cube strength of concrete made with portland cement only
B = CSF fraction of blend.

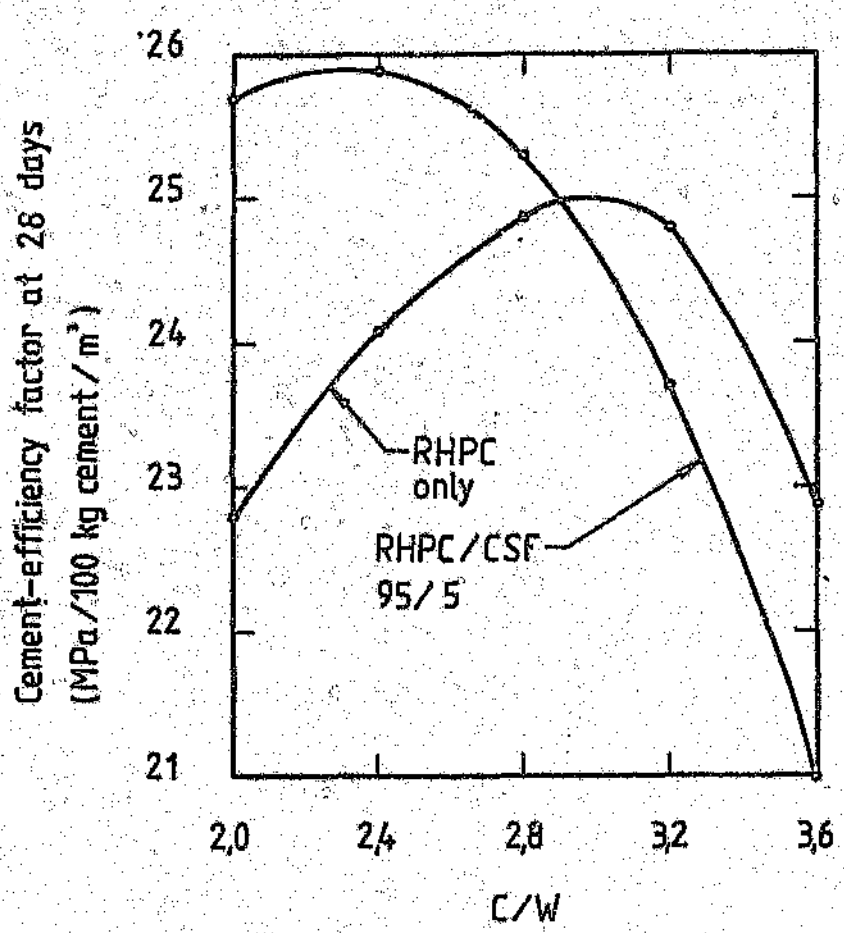


FIGURE 8.24: Cement–efficiency factor versus cement:water ratio for concretes made with RHPC only and a blend of RHPC and condensed silica fume

Values calculated from equation 8.5 and Figure 8.24 are as follows:

C/W	RE for B = 0,05
2,0	3,5
2,4	2,5
2,8	1,3
3,2	0,1
3,6	–0,7

As can be seen, the effectiveness of CSF, relative to the specific rapid-hardening portland cement used for the tests, decreased virtually linearly with increasing C/W. The phenomenon seems to be not unexpected when considering the explanation of the fine-filler effect as described above. At low values of C/W, a gel formed by the hydration of portland cement on its own would have marked inhomogeneities and the fine filler would be able to make a significant improvement. As C/W increases, homogeneity improves and when the point (or zone) of cement saturation is reached the fine filler can effect relatively little improvement.

In the light of the above, it therefore appears that CSF is of little use in improving the **efficiency** of the binder at high values of C/W. But Figures 8.13 and 8.15 show that CSF marginally improves **strength** at such values.

This apparent anomaly can possibly be explained by comparing the water requirement of the two series of concrete mixes considered in Figure 8.14. Water requirement is shown in Figure 8.10. Although the 5 % CSF mix has a higher water requirement than the RHPC-only mix at all values of C/W, the difference at the higher values of C/W is twice the difference at the lower values.

At a given C/W, water content determines the cement content. Because CEF is a function of the reciprocal of cement content, CEF at a given C/W is a function of the reciprocal of water content. The higher water requirement of the 5 % CSF mixes at high values of C/W therefore adversely affects their CEF, relative to that of the RHPC-only mix. In this case, the small difference in strength is insufficient to cancel out the effect of the difference in water requirement.

It should be borne in mind that the above comparison refers to a specific portland cement and that other portland cements could give different values, although the trends could be expected to be similar.

The discussion thus far has dealt with concrete cured in water under laboratory conditions. Site concrete, even if moist-cured, is unlikely to have as much water available for hydration. CSF is a pozzolan and because the pozzolanic reaction is not hydration,[8.11] ie it does not consume water, the substitution of CSF for some of the portland cement will make more water available for the hydration of the remaining portland cement. This could have an indirect beneficial effect on strength of site concrete and supports the case for using CSF blends for high-strength concrete.

8.5 Performance of various aggregate types

The object of this part of the work was to investigate the concrete-making properties of different aggregate types and, if possible, to establish relationships between aggregate properties and their effects on high-strength concrete.

8.5.1 Types of aggregate used

Samples of 26,5-mm stone, 6,7-mm stone, and crusher sand of the following aggregate types were obtained:

Geological type	Quarry
Dolomite	Olifantsfontein
Granite	Jukskei
Andesite	Eikenhof
Dolomite	Daleside
Quartzite	Ferro
Dolerite	Roolkraal

In the case of the Ferro Quarry, samples of washed and unwashed crusher sand were obtained.

8.5.2 Aggregate properties

8.5.2.1 Gradings and densities

Sieve analyses and density determinations were carried out on each sample and the results are shown in Table A8.4.1 of Appendix A8.4.

Particle–size distributions of the fine material in the crusher sands were determined using the hydrometer method[8.12] and results are shown in Table A8.4.2 of Appendix A8.4.

Note that hydrometer determinations do not necessarily correspond exactly to results of sieve analyses. In the hydrometer method, "particle sizes" are diameters of spherical particles having the same settlement characteristics as the actual particles. Sieve analyses do not give particle sizes; they indicate percentages of material which pass through sieves with square openings of standard sizes.

Particle–size distributions of the fine material in the sands are shown graphically in Figure 8.25.

8.5.2.2 Particle shape

Particle shapes were typical of crushed material: chunky, flat and elongated particles were present to different degrees.

8.5.2.3 Packing capacity

The degree to which an aggregate, when compacted in a container, packs to minimize the volume of voids is a property which depends on particle shape and particle–size distribution. For a single–sized aggregate, the closer the particle shape is to spherical the denser the possible packing; flaky and elongated particles will pack less densely than cubical or chunky particles. For particles of given shape the densest possible packing is a function of particle–size distribution. The densest packing will be possible when the smaller particles fill the spaces between the larger particles without pushing them apart.

The packing capacity of an aggregate can be described by the proportion of solids in the compacted state. This is equal to the compacted bulk density divided by the solid density and can be conveniently expressed as a percentage. Values are given in Table A8.4.1 and shown in histogram form in Figure 8.26.

The significance of packing capacity of concrete aggregates is that the higher the packing capacity the lower the volume of voids that has to be filled with paste. High packing capacities tend therefore to be associated with low water requirements. This is true in spite of the fact that aggregate particles do not touch each other in the mix. See section 6.4.

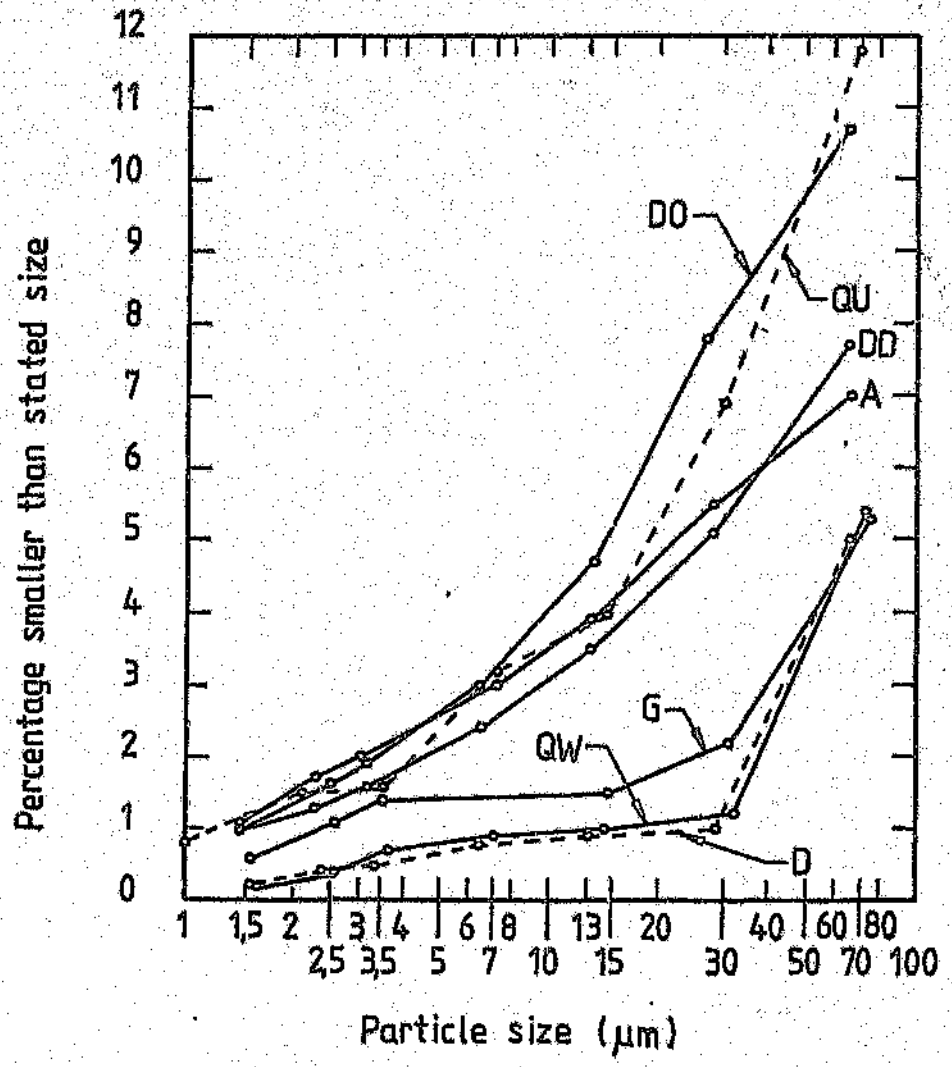


FIGURE 8.25: Fine material in sands according to hydrometer method

DO	Dolomite ex Olifantsfontein
G	Granite
A	Andesite
DD	Dolomite ex Daleside
QW	Quartzite, washed
QU	Quartzite, unwashed
D	Dolerite

8.5.2.4 Aggregate strength

The strength of a concrete aggregate can be measured: directly by dermining the unconfined compressive stress at failure of specimens such as cubes or plane-ended cylinders cut from the parent rock; or indirectly by measuring the extent of crushing of a stone aggregate sample under load.

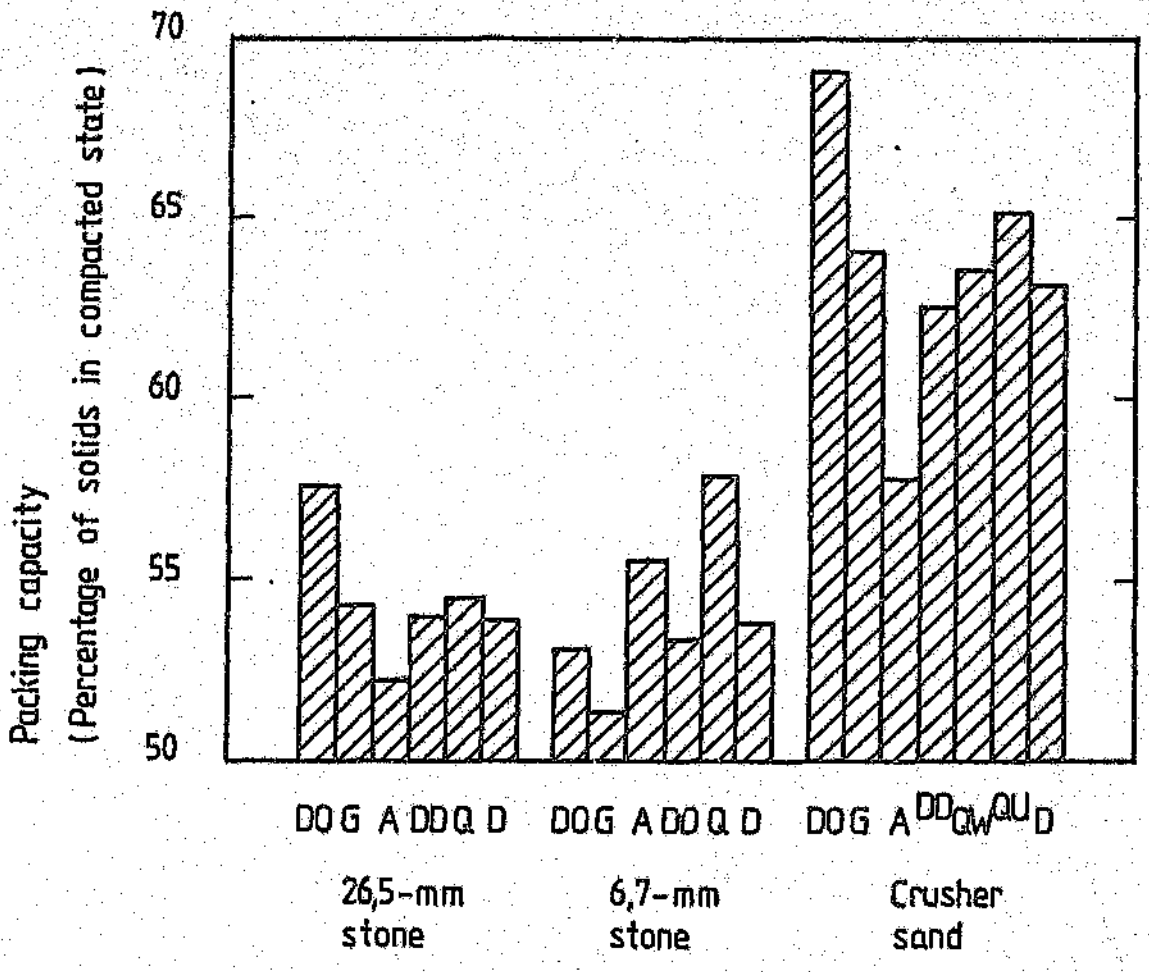


FIGURE 8.26: Packing capacity of various aggregates (see Figure 8.25 for key)

While the direct method is theoretically preferable, it is expensive and impracticable for routine testing. The indirect method on the other hand is relatively simple and economical but indicated strengths are influenced by particle shape: elongated and flaky particles tend to break more easily than chunky or rounded shapes.

Two methods for the indirect approach are prescribed by South African Bureau of Standards: Aggregate crushing value (ACV), Method 841;[8.13] and the Ten per cent fines aggregate crushing test (10 % FACT), Method 842.[8.5]

The ACV test was used for the investigation in preference to the 10 % FACT because it makes provision for aggregate sizes of 26,5 mm and 6,7 mm which were the sizes used for the concrete. While the ACV test is a useful means of comparing aggregates, results obtained with different aggregate sizes may differ.

Results of the ACV tests are shown in Table A8.4.3 of Appendix A8.4 and graphically in Figure 8.27.

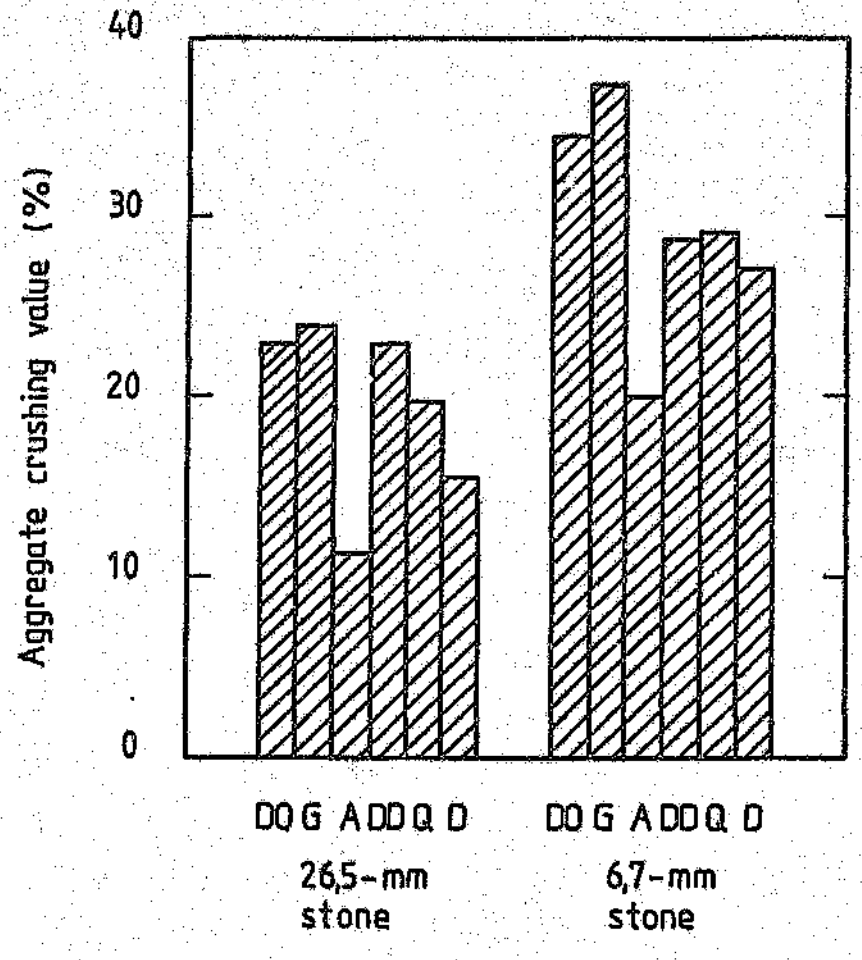


FIGURE 8.27: Aggregate crushing value for various coarse aggregates (see Figure 8.25 for key)

Further information on strength characteristics of some of the aggregates was obtained from Reference 8 [illegible] and is shown in Table A8.4.4 of Appendix A8.4.

8.5.2.5 Elastic modulus of aggregate

Results obtained from tests on specimens cored from solid rock were as follows:

Dolomite ex Daleside: E = 108,5 GPa (an average of 8 tests with a standard deviation of 4,7 GPa)

Dolerite ex Rooikraal: E = 103,3 GPa (an average of 5 tests with a standard deviation of 4,2 GPa)

Other aggregate types used in this part of the investigation were not tested directly but typical values for similar materials were taken from Reference 8.14.

Andesite ex Eikenhof	E =	96 to 105 GPa
Dolomite ex Olifantsfontein	E =	109 to 118 GPa
Granite ex Jukskei	E =	76 to 84 GPa
Quartzite ex Ferro	E =	42 to 98 GPa

8.5.2.6 Blending of large and small stone

In order to get the best possible performance from each aggregate type, a blend of large and small stone was used. The optimum blend for each aggregate is that with the highest packing capacity and consequently the [illegible] content and therefore the lowest water content.

The optimum blend of 26,5–mm and 6,7–mm stone was determined for each aggregate type by measuring compacted bulk density of blends of different proportions. Results are shown in Table A8.4.5 of Appendix A8.4 and Figure 8.28.

The following blend ratios of 26,5–mm and 6,7–mm stone and resulting CBDs were used for concrete mixes (the larger size is given first):

Aggregate	Blend ratio	CBD (kg/m³)
Dolomite ex Olifantsfontein	75:25	1 850
Granite	70:30	1 660
Andesite	75:25	1 790
Dolomite ex Daleside	65:35	1 810
Quartzite	70:30	1 620
Dolerite	65:35	1 900

Note that the CBD shown above is based on the average of the initial test (Table A8.4.1) and the test done for blending.

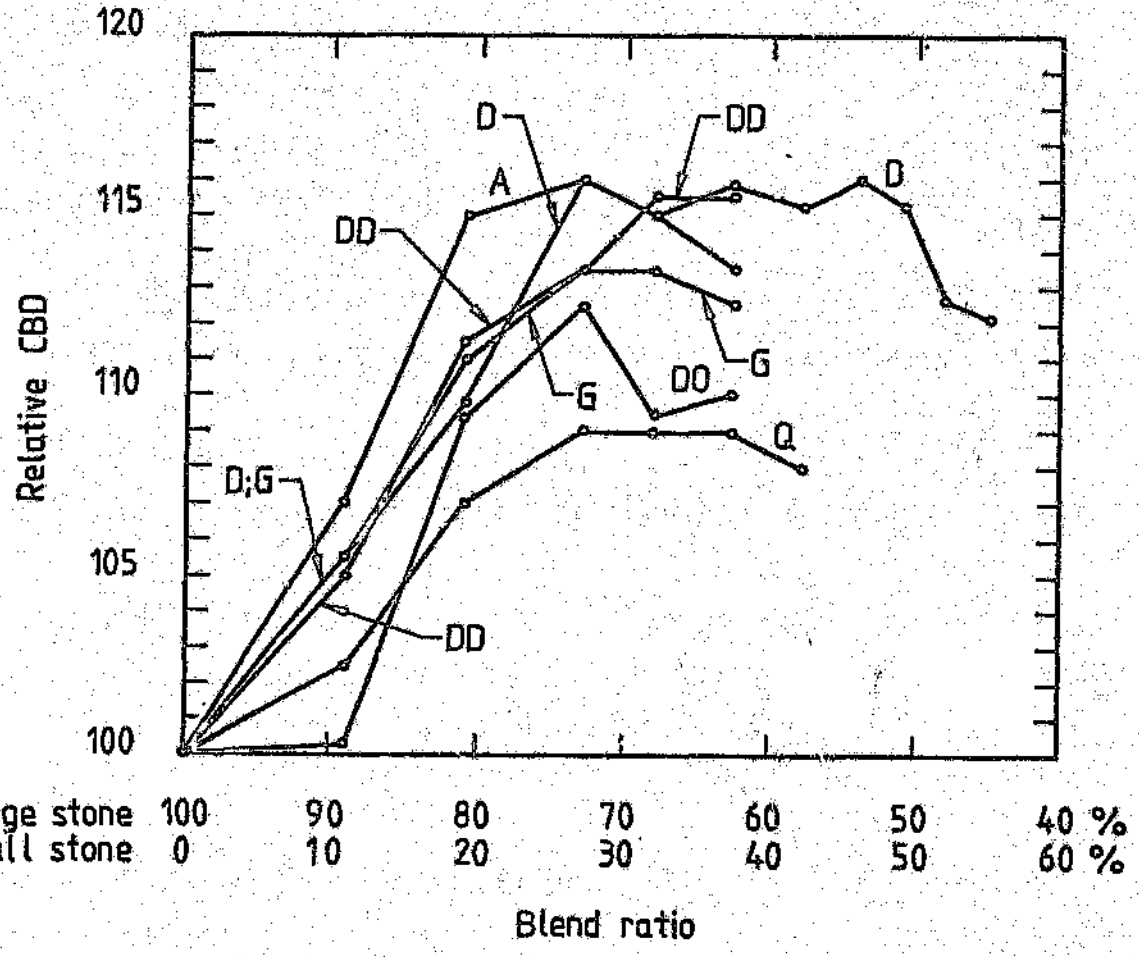


FIGURE 8.28: Relative compacted bulk density of blends of 26,5–mm and 6,7–mm stone (see Figure 8.25 for key)

8.5.3 Aggregate in concrete

8.5.3.1 Mixes

Concrete mixes were made using the various aggregate types at different cement:water ratios with the object of studying the effect of aggregate type on the properties of fresh and hardened concretes. Stone and sand from the same source were used for each mix. The same cement (RHPC type "J") and admixture, sulphonated naphthalene formaldehyde (SNF) type "C", in liquid form dosed at 2 ℓ to 100 kg of cement were used for all mixes. Mixes were proportioned according to the method discussed in section 6.4. Slump was 75 mm plus or minus 20 mm and workability satisfactory in all cases. Cubes were made and tested in accordance with SABS Method 863.[8.17] Mix proportions, and strengths and cement–efficiency factors at 7, 28 and 56 days are shown in Tables A8.4.6 to A8.4.12 of Appendix A8.4. Cube strengths are an average of three cubes unless stated otherwise.

8.5.3.2 Water requirement of concrete

The effect of aggregate type on water requirement is shown graphically in Figure 8.29.

Table A8.4.13 of Appendix A8.4 shows water requirement of concrete at C/W values of 2,0 and 3,6 and packing capacity (solid volume at CBD) of the various sands and stone blends. Water requirement is shown plotted against average packing capacity of stone blend and sand in Figure 8.30. The figure shows some correlation as could be expected. Correlation is higher and the slope of the best–fit line steeper at C/W = 3,6 than at C/W = 2,0, indicating a greater influence of aggregate packing capacity in the richer mixes. This is the opposite of what could be expected because mixes contain about 9 % less aggregate at C/W = 3,6 than at C/W = 2,0. But differences are small and could be affected by the small number of statistical points.

8.5.3.3 Strength

7–, 28–, and 56–day strengths are shown plotted against C/W in Figures 8.31, 8.32 and 8.33 respectively.

Andesite and dolerite, which have the lowest ACVs, produced some of the stronger mixes at all three ages. To investigate whether cube strength correlates with ACV, values are shown in Table A8.4.14 of Appendix A8.4 and graphically in Figure 8.34.

As can be seen, correlation is poor although it is slightly better at C/W = 3,6 than at C/W = 2,0. (A comparison between ACV of 26,5–mm stone and strength gave as poor a correlation.) The slope of the best–fit line at C/W = 3,6 is however almost twice that of the line at C/W = 2,0; this indicates that aggregate strength plays a greater role at higher strengths than at lower strengths.

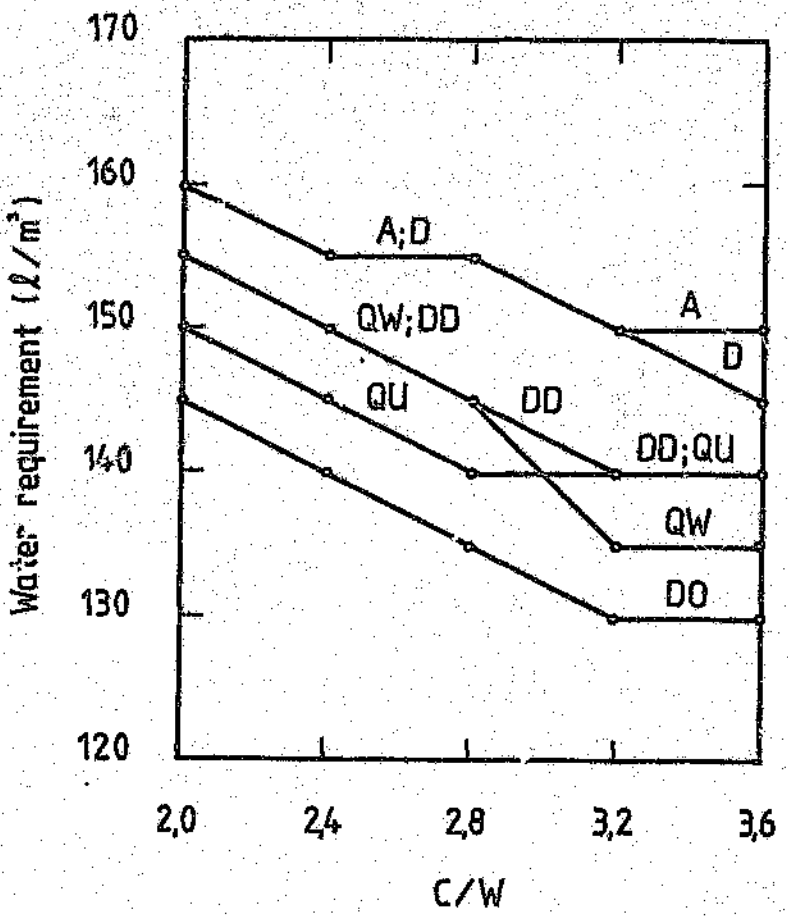


FIGURE 8.29: Water requirement (to nearest 5 ℓ/m³) for 75–mm slump versus cement:water ratio for concretes made with aggregate of various types (see Figure 8.25 for key)

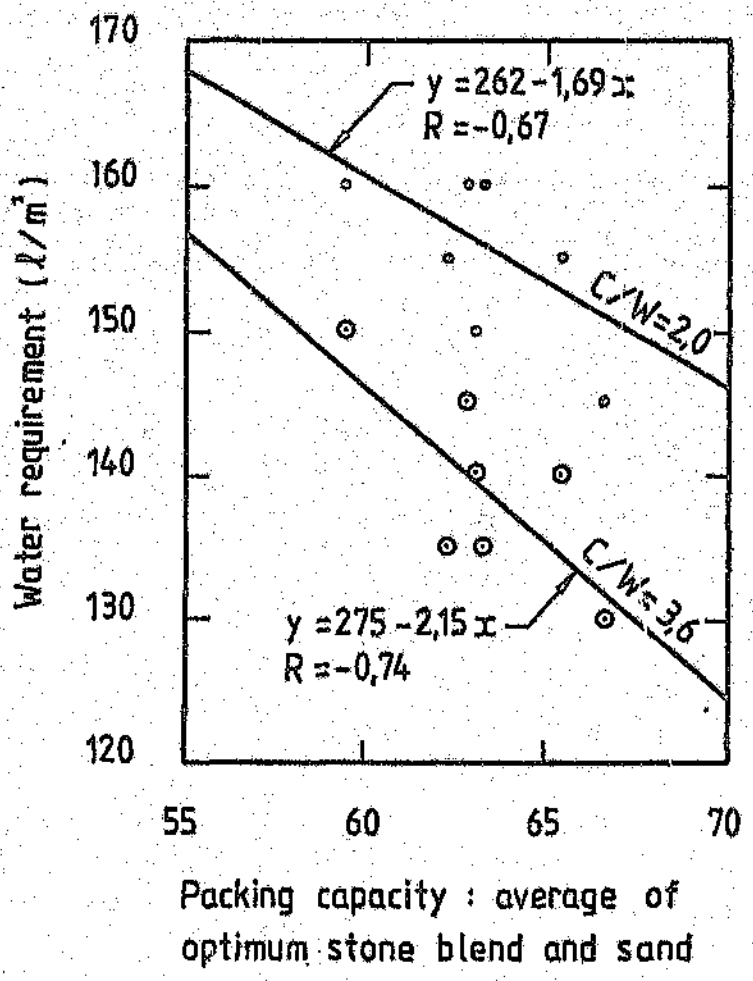


FIGURE 8.30: Water requirement of concrete versus packing capacity of aggregates

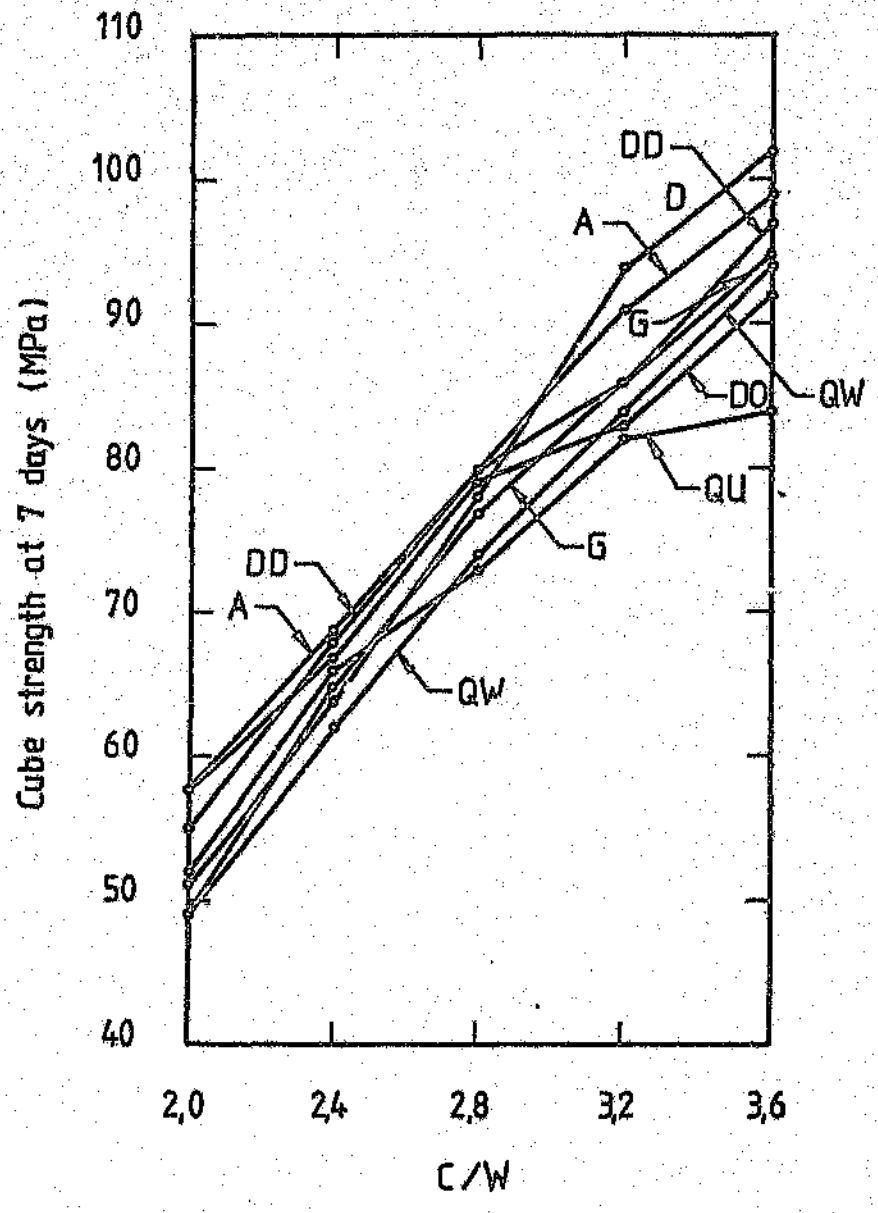


FIGURE 8.31: Cube strength at 7 days versus cement:water ratio for concretes made with aggregate of various types (see Figure 8.25 for key)

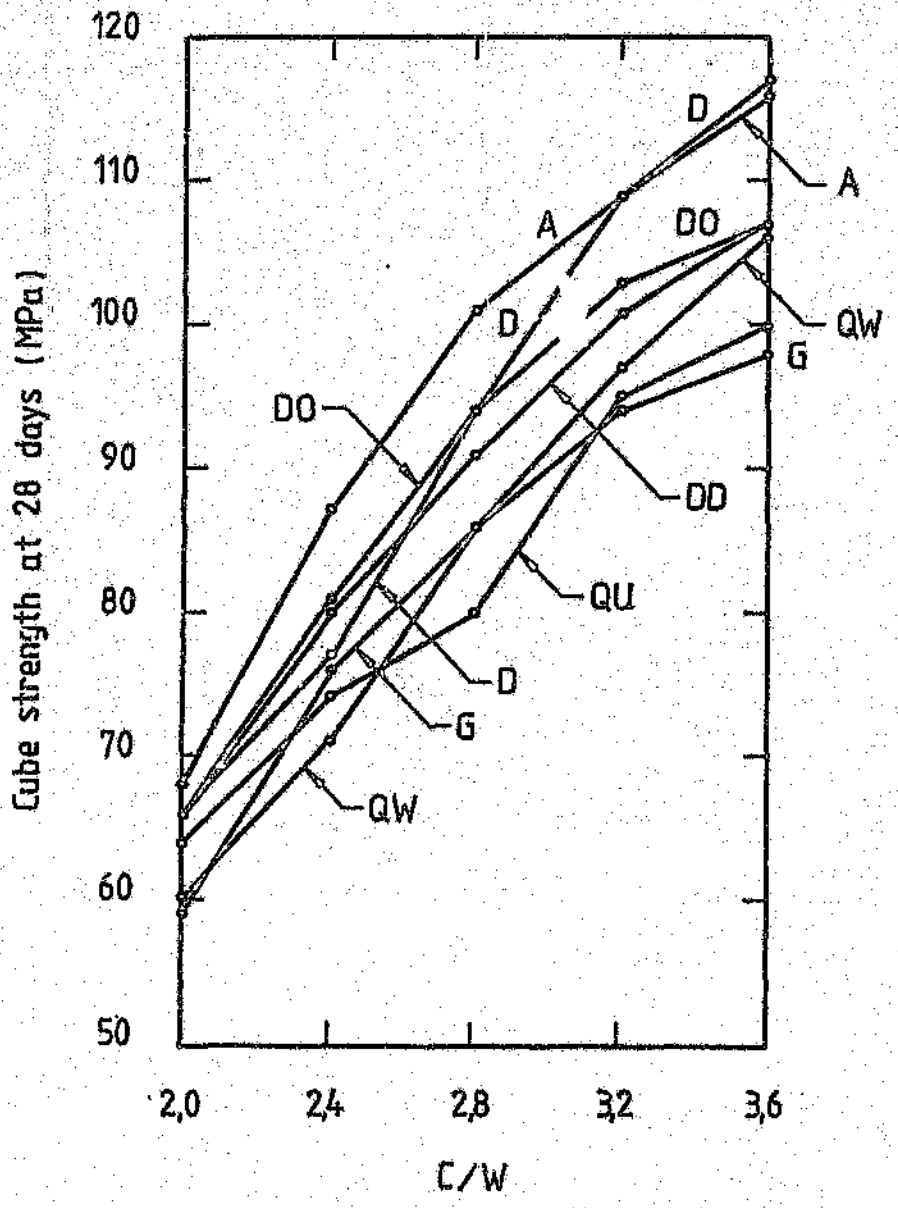


FIGURE 8.32: Cube strength at 28 days versus cement:water ratio for concretes made with aggregates of various types (see Figure 8.25 for key)

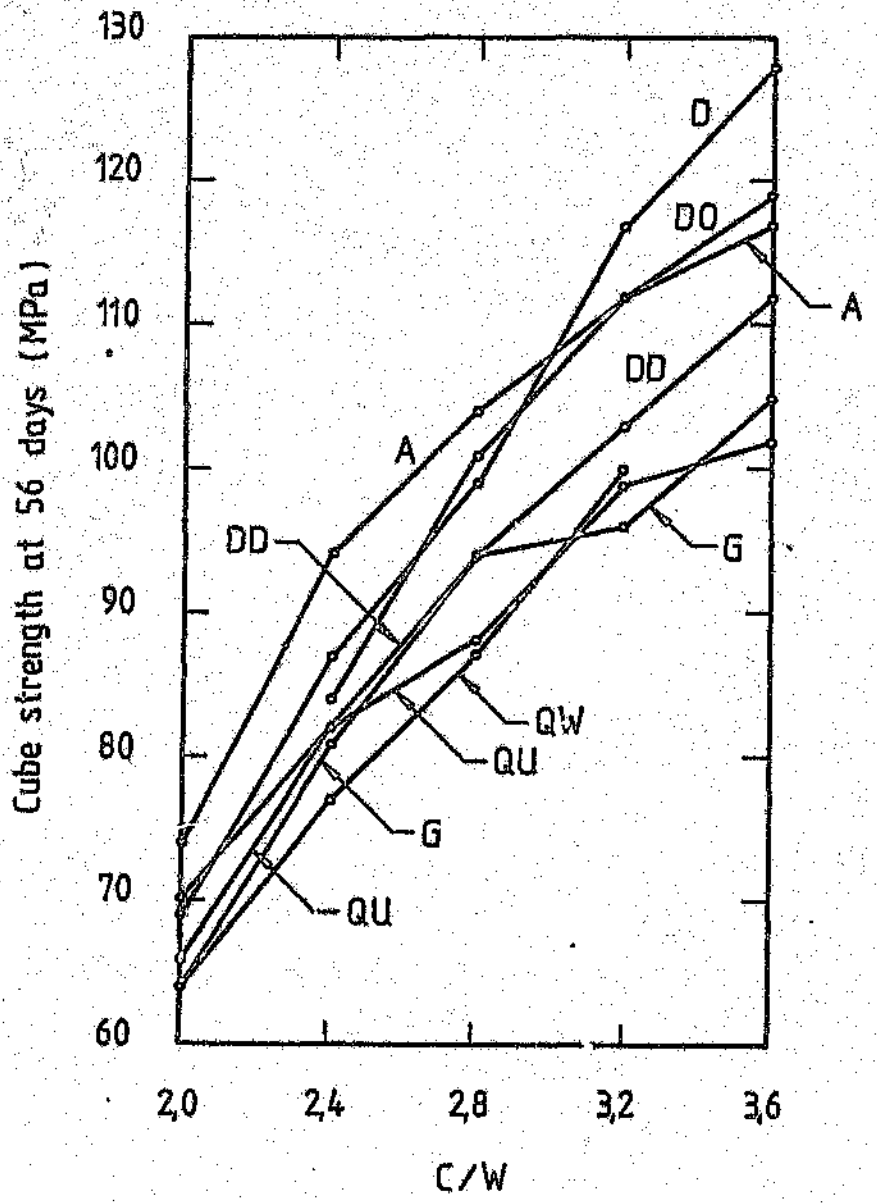


FIGURE 8.33: Cube strength at 56 days versus cement:water ratio for concretes made with aggregates of various types (see Figure 8.25 for key)

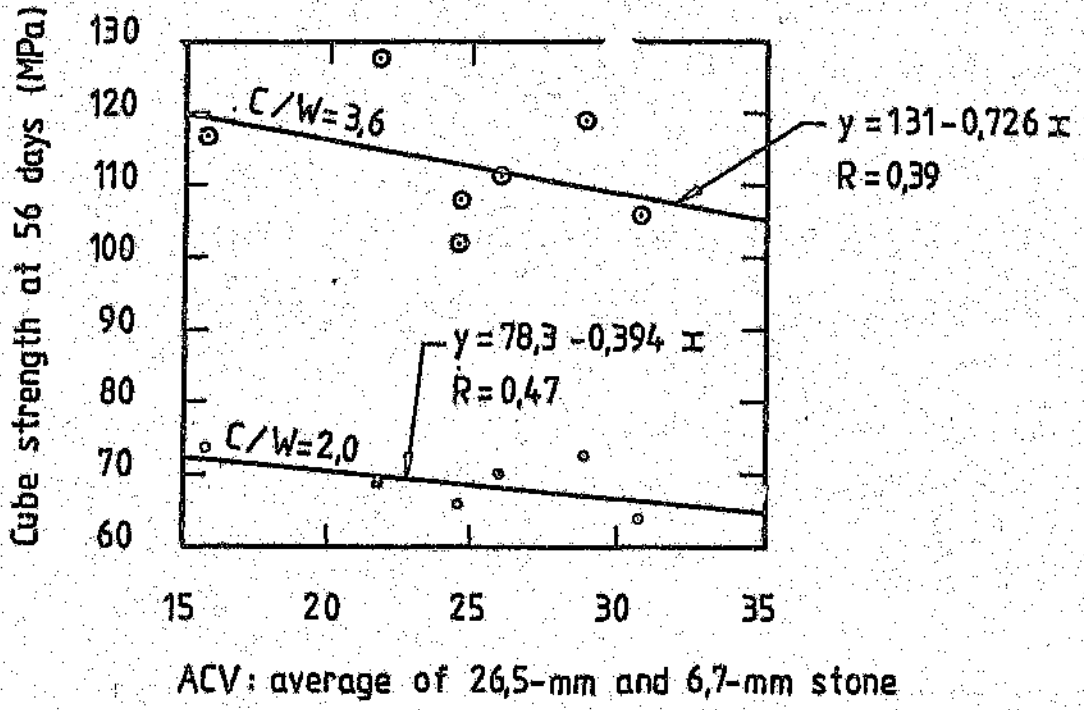


FIGURE 8.34: Cube strength versus aggregate crushing value

Cube strengths also showed virtually no correlation with compressive strengths of rock specimens as shown in Table A8.4.4.

There was no significant correlation between cube strengths and modulus of elasticity of rock but concretes made with the two rock types with the lowest moduli, ie quartzite and granite, were generally the weakest.

Ranges in strength at C/W = 2,0 and 3,6, expressed as a percentage of median value, were as follows:

Age (days)	C/W	Range	
		(MPa)	(%)
7	2,0 3,6	9 18	17 19
28	2,0 3,6	9 19	14 18
56	2,0 3,6	10 26	14 23

It is interesting to note the effect of washing the quartzite crusher sand. At 56 days washed and unwashed sands gave similar strengths at C/W = 2,0 but the washed sand gave the higher strength at C/W = 3,6.

8.5.3.4 Cement-efficiency factor

CEFs at 7, 28 and 56 days are shown in Figures 8.35, 8.36 and 8.37 respectively. Results are rather erratic but the general trend in these figures is for CEF to show a maximum value in the range of 2,4 < C/W < 3,2; dolerite is an exception, achieving its highest values at C/W > 3,2. If fair envelope curves are drawn around the results for 28 days and 56 days, maximum CEF is found at C/W ≈ 2,8 compared with C/W = 2,5 assumed for cement-saturation.

Ranges in CEF at C/W = 2,0 and 3,6, expressed as a percentage of median value, were as follows:

Age (days)	C/W	Range (%)
7	2,0 3,6	29 16
28	2,0 3,6	21 15
56	2,0 3,6	14 23

The best overall performer was dolomite ex Olifantsfontein because of a combination of reasonably high strength and consistently lowest water requirement.

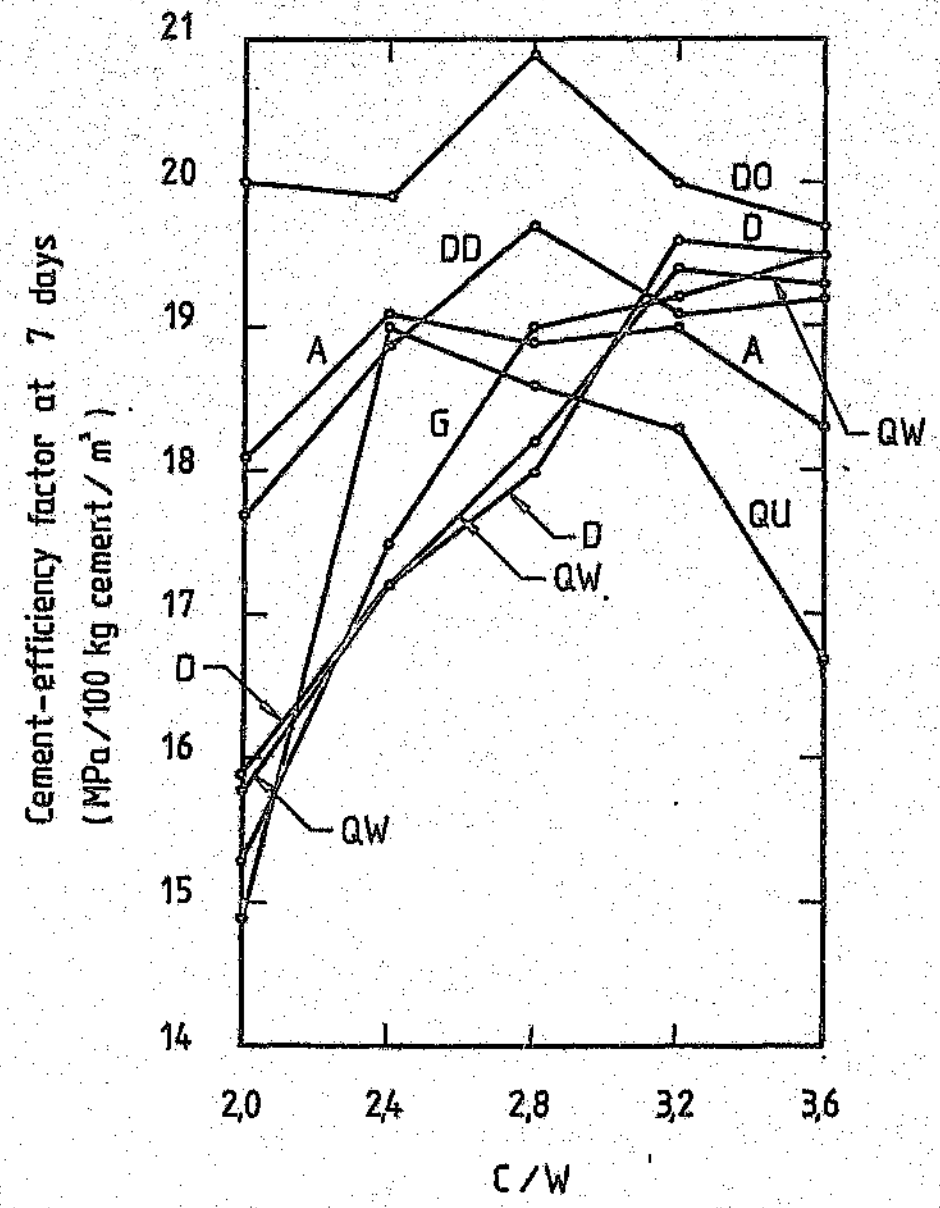


FIGURE 8.35: Cement–efficiency factor at 7 days versus cement:water ratio for concretes made with aggregates of different types (see Figure 8.25 for key)

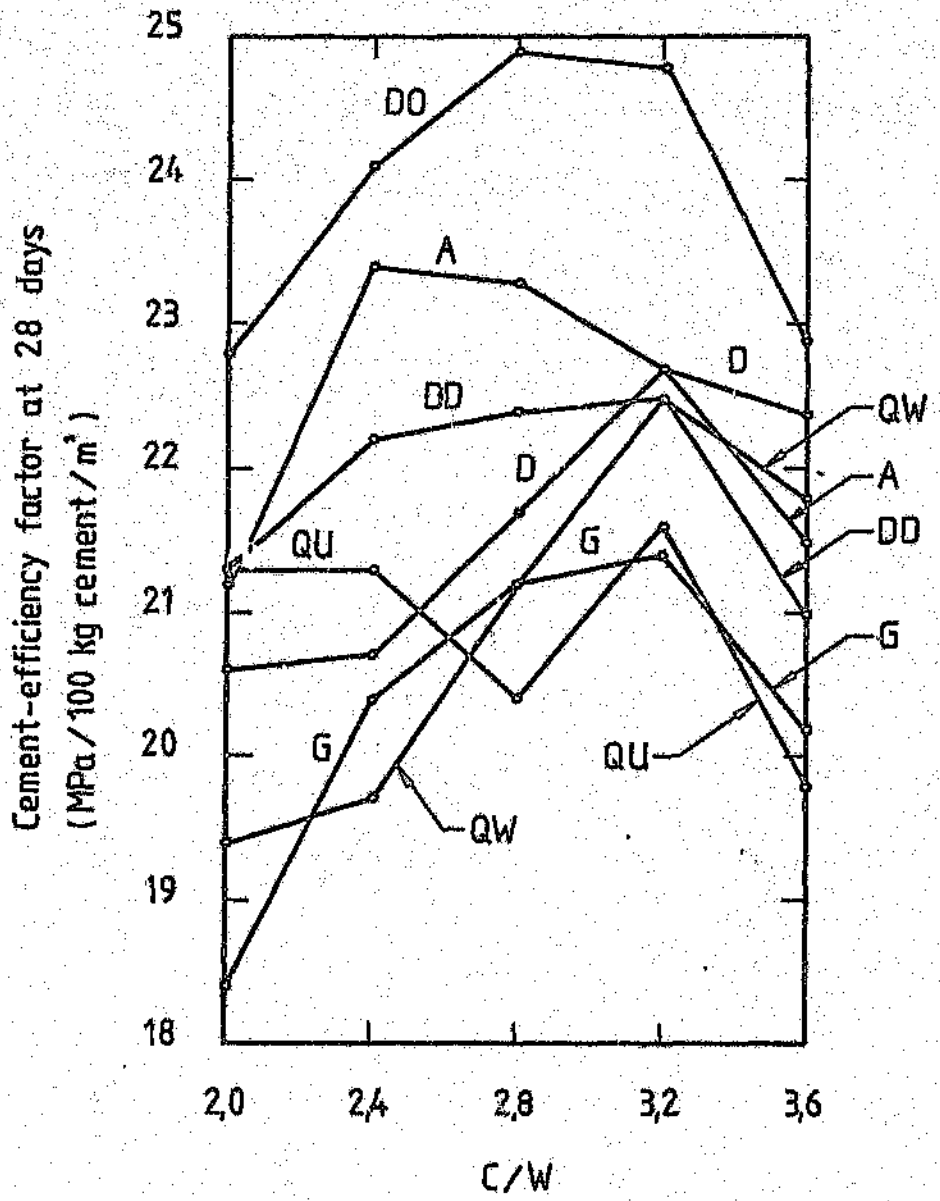


FIGURE 8.36: Cement–efficiency factor at 28 days versus cement:water ratio for concretes made with aggregates of different types (see Figure 8.25 for key)

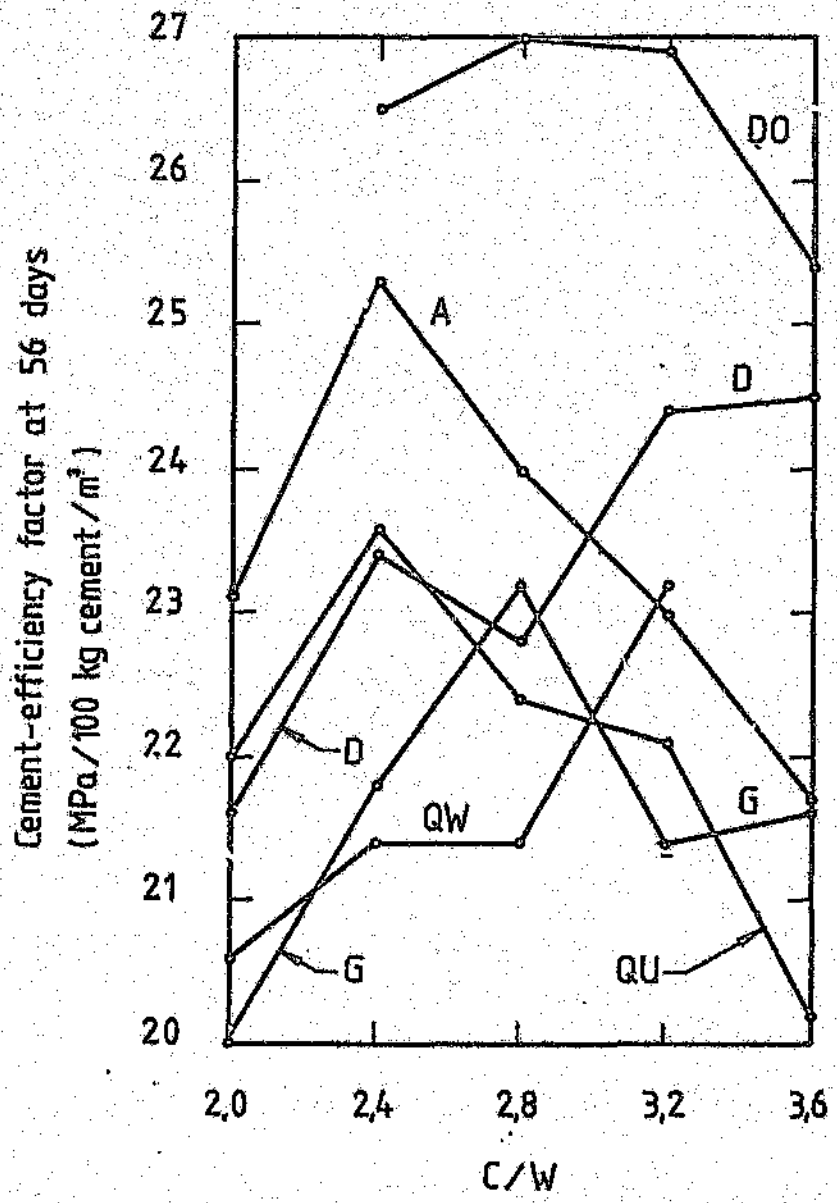


FIGURE 8.37: Cement–efficiency factor at 56 days versus cement:water ratio for concretes made with aggregates of various types (see Figure 8.25 for key)

8.5.4 Conclusions

8.5.4.1 Water requirement

Water requirement is strongly influenced by the packing capacity of the aggregates and can be predicted to within about 5 ℓ/m^3 with the best–fit–line equations at C/W = 2,0 or 3,6. (See Figure 8.30.)

8.5.4.2 Strength of concrete

Aggregate properties that appear to influence concrete strength are:

8.5.4.2.1 Packing capacity

By influencing water requirement and thus total porosity of the hardened concrete, the higher the packing capacity the higher the strength potentially.

8.5.4.2.2 Ultra–fines content

This influences bleeding and plastic settlement. High ultra–fines contents reduce bleeding and potentially increase strength. This effect is often observed in conventional concrete but does not appear to be significant in high–strength concrete, possibly because bleeding is minimal in such rich mixes. There is for example little difference between strengths obtained with washed and unwashed quartzite sands. (See Figure 8.33.) The relatively small effect of ultra–fines at high C/Ws is probably similar to that of condensed silica fume as discussed in section 8.5.3.

8.5.4.2.3 Aggregate strength

There is some relationship between ACV and concrete strength but correlation is not high (eg Figure 8.34). The influence of aggregate strength does however appear to be greater at higher than at lower concrete strengths. (See Figure 8.34.)

Alexander and Davis[8.15] came to a similar conclusion after studying the effect of aggregate properties on the strength of concrete. They measured the strengths of different aggregate types in two ways: unconfined compressive strength (UCS) tests on cores drilled from boulders and 10 % FACT values of aggregate made from the same rock. They found a rough correlation between UCS and 10 % FACT values.

Concrete mixes at two strength levels (60 to 70 MPa and 20 to 40 MPa) were made using the different aggregate types and the ratio of 28-day cube strength to C/W plotted against mean UCS of rock cores. They found some correlation for the higher-strength concretes but none for the lower-strength concretes and concluded that aggregate strength is a factor in concrete strength only for high-strength concretes.

The strength level of Alexander and Davis's higher-strength concrete was similar to that of concretes with C/W = 2 in this investigation.

8.5.4.2.4 Aggregate stiffness

Aggregates with low moduli of elasticity appear to make relatively weak concrete. But because the sample was so small, this could be merely concidental.

8.5.4.3 Selection of aggregates

It does not seem to be possible to predict, without testing, the performance of aggregates with regard to strength. According to Danielsen, it is not fully realized how the different physical and chemical properties of aggregates influence the mechanical properties of concrete.[8.16]

The large percentage differences in strength and CEF between best and worst performers indicate that testing is necessary to find the most suitable aggregate for specific high-strength concretes.

Based on the tests described above, the most promising aggregate types are andesite, dolerite and dolomite. According to reference 8.15, felsite would also be worth testing.

8.6 Splitting-tensile strength and flexural strength

The object of this part of the work was to investigate tensile-related strength properties of high-strength concrete.

8.6.1 Materials used

Aggregates

Aggregates from three sources were used: dolerite ex Rooikraal, andesite ex Eikenhof and dolomite ex Olifantsfontein. Gradings and densities are given in Table A8.5.1 of Appendix A8.5.

Cement

Rapid-hardening portland cement type "R" supplied in a single batch was used for all the tests. A chemical analysis of the cement was not available.

Admixture

A sulphonated naphthalene formaldehyde (SNF) type "C" was used in liquid form.

8.6.2 Concrete mixes

Mixes were proportioned using the method described in section 6.4. Blends of 26,5-mm stone and 6,7-mm stone were used for all mixes. Ratios for these blends were those giving the highest packing capacity for each aggregate type.

Mixes were made at cement:water ratios of 2,0, 2,8 and 3,6 (or near to these) for each aggregate type. Details of mixes are given in Appendix A8.5.

Batch yield volumes were either 10 ℓ, 20 ℓ or 24 ℓ depending on the number of cubes and beams required from each mix.

8.6.3 Strength tests

The following specimens were tested from each mix:

	3 cubes for compressive strength at 28 days;
plus	3 cubes for splitting-tensile strength at 28 days;
and/or	3 beams for flexural strength at 28 days.

Cubes were all 100 mm. Beams were 100 mm x 100 mm x 500 mm.

All specimens were stored overnight in their moulds in a humidity chamber at 95 % relative humidity, demoulded the next day, and then cured in water at 22 °C to 25 °C until being tested wet.

Testing was done according to the following methods:

Compressive strength	:	SABS Method 863[8.17]
Splitting tensile strength	:	Method proposed by Nilsson; line loads applied to opposite faces of a cube[8.18]
Flexural strength	:	SABS Method 864.[8.16]

Strengths were calculated as follows:

$$\text{Compressive strength} = \frac{P}{a^2}$$

$$\text{Splitting tensile strength} = \frac{2P}{\pi a^2}$$

$$\text{Flexural strength} = \frac{P\ell}{bd^2}$$

where P = load at failure
a = size of cube = 100 mm
ℓ = span of beam under test (ie distance between supports) = 300 mm
b = width of beam = 100 mm
d = depth of beam = 100 mm.

8.6.4 Results

Mix proportions are given, together with compressive strength, and splitting-tensile strength and flexural strength where applicable, in Tables A8.2 to A8.14 in Appendix A8.5. Results of strength tests are shown graphically in Figure 8.38; the expressions for splitting tensile strength and flexural strength were derived from reference 2.59 using a factor of 1/0,85 to convert cylinder strength to cube strength. It may be noted

that the formulae are applicable for concrete strengths up to 83 MPa (cylinder).[2.59] This is equivalent to about 100 MPa cube strength.

Photographs of split faces at three different strength levels are shown in Figures 8.39 to 8.41.

8.6.5 Comments

8.6.5.1 Fresh concrete

Slumps differed noticeably from mix to mix. Reasons for this were the extreme sensitivity of slump to water content found in superplasticized mixes – this aspect is discussed in chapter 5 – and the tendency for larger batches of given proportions to have a higher slump. Nevertheless it is believed that water content was in all cases close to the content required for a slump of approximately 75 mm.

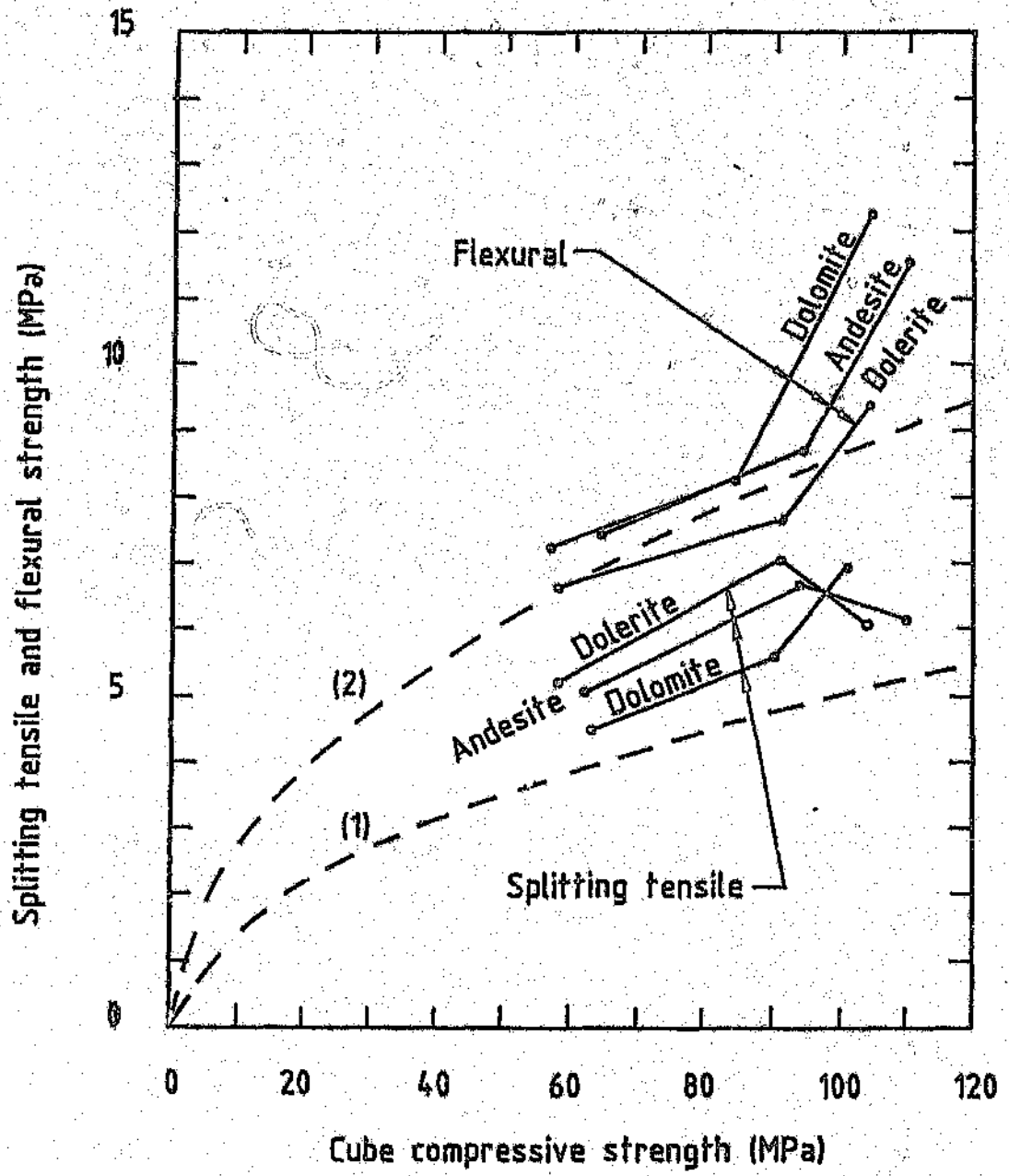


FIGURE 8.38: Splitting-tensile strength and flexural strength versus cube strength for concretes made with aggregates of various types;

(1) T = $0{,}50\sqrt{f_c}$ (reference 2.59)
(2) F = $0{,}87\sqrt{f_c}$ (reference 2.59)
where T = splitting tensile strength
F = flexural strength
f_c = cube compressive strength

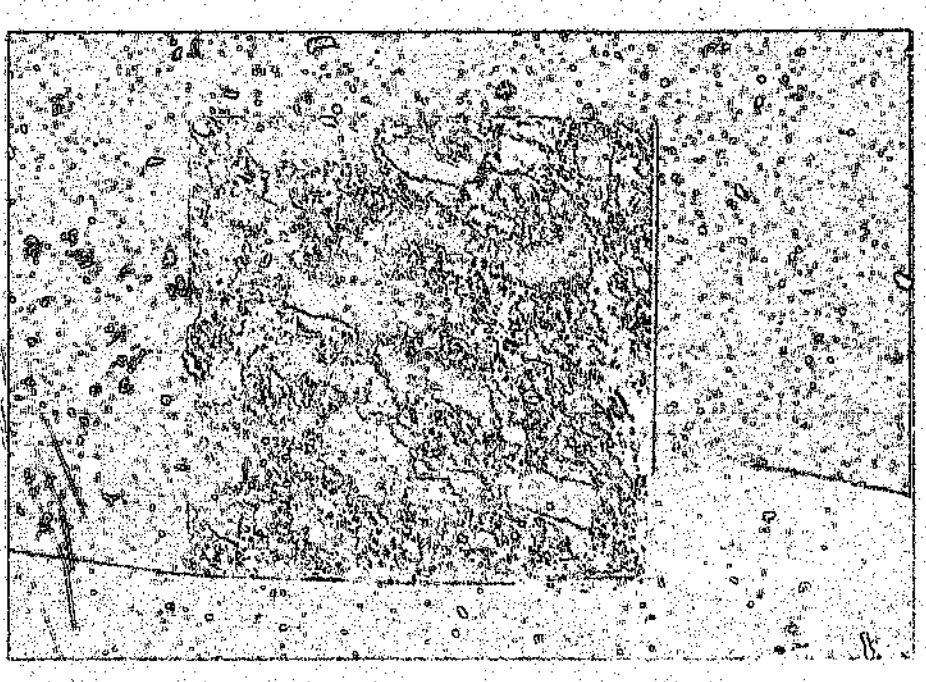


FIGURE 8.39: Photograph of fracture face; splitting–tensile test; dolomite aggregate; cube strength 63 MPa; splitting–tensile strength 4,5 MPa

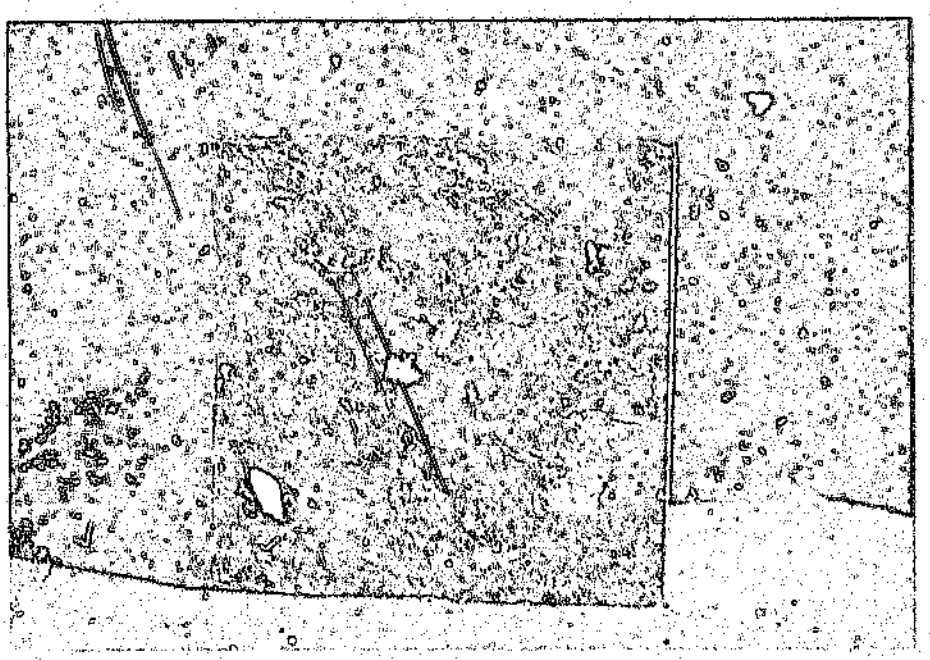


FIGURE 8.40: Photograph of fracture face; splitting–tensile test; dolomite aggregate; cube strength 90 MPa; splitting–tensile strength 5,6 MPa

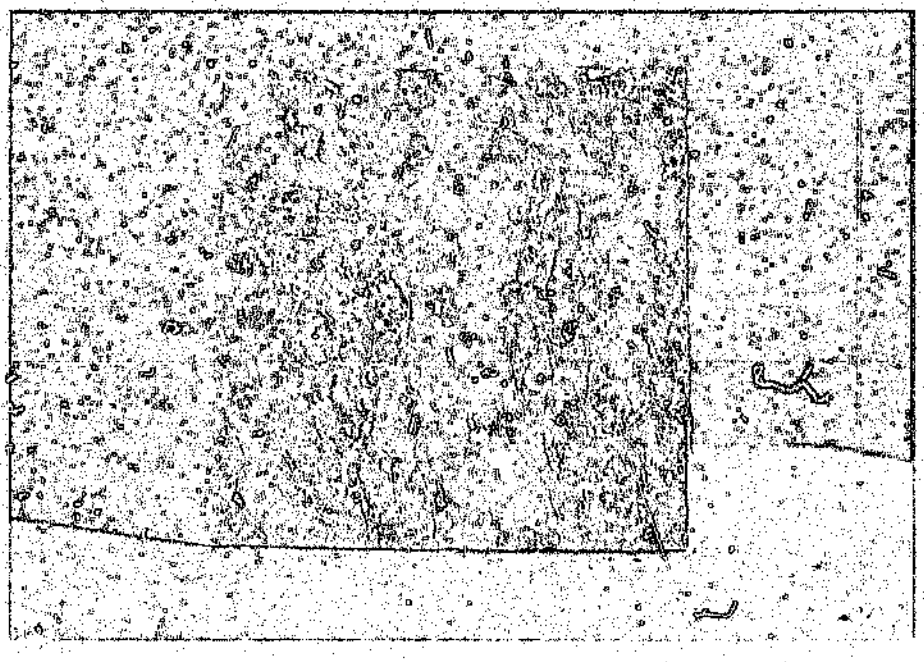


FIGURE 8.41: Photograph of fracture face; splitting-tensile test; dolomite aggregate; cube strength 101 MPa; splitting-tensile strength 7,0 MPa

8.6.5.2 Splitting-tensile strength

Expressed as a percentage of compressive strength, splitting-tensile strength was an average of 7,0 % with individual results ranging between 5,9 % and 9,1 %.

As shown in Figure 8.38, all results were higher than predicted by the expression derived from reference 2.59. Differences between results and the curve according to the formula are however small enough to allow the use of the formula as a conservative estimate of splitting-tensile strength of high-strength concrete.

The variability of strengths of individual specimens within groups of three differed markedly for the two tests: average coefficient of variation (CV) was 7,4 % for splitting-tensile and 2,7 % for compressive strength; CV ranged between 2,6 % and 16,0 % for splitting-tensile strength and between 0,4 % and 6,9 % for compressive strength.

The reason for the significant difference in variability of the two tests is possibly that splitting–tensile strength is dependent on the nature, size and position of significant flaws in or close to the fracture plane; in the compression test, the rigid and very flat surfaces of the platens exert an averaging influence on the behaviour of the specimen.

Some understanding of the mechanism of failure in splitting can be gained by studying Figures 8.39 to 8.41. At the lower strength (Figure 8.39), the split face shows predominantly coarse deviations from the plane due to failure on aggregate–paste interfaces, although some aggregate particles have broken. As strength increases, the split face shows an increasing degree of planeness.

Failure at lower strengths is associated with a high incidence of failure of paste–aggregate interfaces. As concrete strength increases, differences in strength between paste–aggregate interfaces and the aggregate particles themselves decrease and failure occurs more in a plane passing through paste and fine and coarse aggregate particles.

These observations confirm the descriptions of failure surfaces in the literature.[2.46, 2.60, 2.66]

8.6.5.3 Flexural strength

Expressed as a percentage of compressive strength, flexural strength was an average of 10,6 %. with individual results ranging between 9,0 % and 12,8 %.

As shown in Figure 8.38, results were generally similar to values predicted by the formula derived from reference 2.59; two exceptions were concretes made with andesite and dolomite where cube strength exceeded 100 MPa. The formula appears therefore to be a realistic to conservative prediction of flexural strength.

The variability of strengths of individual specimens within groups of three differed markedly for the two tests: average coefficient of variation (CV) was 8,7 % for flexural strength and 2,8 % for compressive strength; CV ranged between 2,1 % and 14,7 %

for flexural strength and between 0,2 % and 6,9 % for compressive strength. The reason for this difference is probably similar to that proposed in the discussion on splitting tensile strength in section 8.6.5.2 above.

Fracture surfaces tended to become more plane as concrete strength increased. The reason for this is believed to be the same as for a similar phenomenon observed in splitting–tensile tests as discussed in section 8.6.5.2 above.

8.6.6 Conclusions

In the tests described, tensile–related concrete strengths were similar or higher than values predicted by formulae developed for concrete with equivalent cube strengths up to about 100 MPa. Nevertheless formulae can be used to give generally conservative predictions of splitting–tensile strength and flexural strength.

In both tests, fracture surfaces tended to become more plane with increasing strength. Plane or nearly plane fracture surfaces are associated with a lack of aggregate interlock and would therefore offer relatively poor load transfer in shear across a crack. High–strength concrete would therefore not be suitable for the construction of floors on the ground where cracking is controlled by sawn joints unless such joints are provided with dowels. Nearly plane fracture surfaces are also relevant to the shear resistance of beams.[2.80, 2.86, 2.87]

The strength and mode of failure of high–strength concrete under tension are sufficiently different from those of conventional concrete to preclude the use of design and construction practice, developed for conventional concrete, for high–strength concrete without appropriate adaptation.

8.7 The effect of specimen aspect ratio on the measured compressive strength of concrete and the influence of high strengths on this effect

It is generally found that for a given concrete, specimens with a greater aspect ratio fail at lower compressive stresses than those with a smaller aspect ratio. For example, cylinders with height equal to twice diameter are weaker than cubes. Some workers

have reported that the relation between cylinder and cube strength depends on the strength of the concrete. Munday and Dhir,[8.20] for example, propose correction factors ranging from 0,7 to 0,9 for concrete strengths ranging from 29 MPa to 60 MPa respectively.

To test whether the effect of slenderness on concrete strength depends on strength level, two series of tests were carried out.

In the first series, prisms of square cross section were cast to different lengths between precast high-strength cubes in 500 x 100 x 100-mm beam moulds. Faces of the precast cubes were lightly coated with petroleum jelly immediately before the prisms were cast. Two sets of prisms were made: at cube strengths of about 25 MPa and 70 MPa. Four prisms were made for each length and strength level.

Materials used for both sets of specimens were ordinary portland cement, 13,2-mm dolerite stone and a weathered Ecca grit sand from Kendal. In addition, a sulphonated naphthalene formaldehyde plasticizer was used in the high-strength mix.

Results were [average of four tests for each value of height over width (H/D)]:

H/D	Average strength (MPa)	Strength relative to strength at H/D = 1
0,5	38,0	1,53
0,75	27,8	1,12
1 (cube)	24,8	1,00
2	20,5	0,83
3	20,8	0,84

H/D	Average strength (MPa)	Strength relative to strength at H/D = 1
0,5	95,4	1,37
0,75	75,4	1,08
1 (cube)	69,7	1,00
2	60,1	0,86
3	59,4	0,85

In the second series, using RHPC, SNF superplasticizer and andesite aggregate, mixes were made up with C/W = 2,0 to 3,6 in increments of 0,2. Nine cubes were made with each mix. (Details of mixes are shown in Appendix A8.3.) At 28 days cubes were crushed, placed on their sides as cast, as follows:

- Three cubes individually according to the standard procedure.
- Three sets each of two cubes stacked one on top of the other.

Results were:

C/W	2,0	2,2	2,4	2,6	2,8	3,0	3,2	3,4	3,6
A	66,4	74,8	83,0	89,3	99,3	102,3	102,7	108,7	112,5
B	54,7	58,8	66,2	75,2	77,8	80,7	84,5	89,8	97,3
B/A	0,82	0,79	0,80	0,84	0,78	0,79	0,82	0,83	0,86

A = average cube strength (MPa) B = average strength of two cubes stacked (MPa)

In the first series relative strengths were virtually the same for 25–MPa and 70–MPa concrete. In the second series ratio B/A ranged from 0,79 to 0,86 for the nine levels of C/W and averaged 0,81.

A comparison of the results of the two series shows that pairs of stacked cubes behaved similarly to specimens of H/D = 2. Stacked cubes can therefore be used to study the effects of specimen aspect ratio on strength.

Results from both series showed that strengths of taller or shorter prisms relative to cube strength showed no trend with increasing concrete strength. In other words, high–strength concrete behaves no differently to conventional concrete in this respect.

This does not agree with the proposals given in reference 8.20. The reason for the difference has possibly to do with the specificity of concrete. Unfortunately no information on mixes is given in the reference.

8.8 Strength of concrete cast in situ

It is recognized that the strength of cast in situ conventional concrete is lower than that of laboratory cast and cured specimens of the same concrete. Criteria for acceptance of concrete based on tests on cores taken from structures are based on strength ratios of in situ to laboratory concrete of between 70 % and 90 %.[8.21, 8.22, 8.23, 8.24]

It is important to the designer of a structure of high–strength concrete to know whether ratios of in situ strength to laboratory strength are the same for high–strength concrete as for conventional concrete.

Haug and Jakobsen [8.24] reported the results of an extensive programme of testing of cast–in situ concrete used in three Norwegian–built offshore platforms. Approximately 1 000 cores were drilled and tested and compared with companion laboratory cast and cured specimens. It was found that strengths of cast–in situ concrete varied between 75 % and 100 % of the characteristic strength of laboratory concrete. Concrete taken from structures built by slipforming was superior to that from structures built by

conventional placing methods, presumably because of the low segregation, good compaction and some revibration of previously placed layers that can be achieved when concrete is placed in the small height increments typical of slipforming.

The conclusion that can be drawn from reference 8.24 is that the performance of high-strength concrete cast in situ, and relative to the strength of laboratory specimens, is similar to that of conventional concrete. Because of the self-desiccating tendency of concretes with very high cement:water ratios, this conclusion would be correct only if the high-strength concrete is cured by making additional water available at the concrete surface.

8.9 Within-batch variation of cube strength

According to SABS Method 863[8.17] **Compressive strength of concrete**, the compressive strength of concrete is the average strength of three cubes from the same batch of concrete; cubes are made, cured and tested according to a standard procedure. Method 863 states that the test is valid if the difference between the highest and lowest result in the group of three does not exceed 15 % of the average. A range of 15 % within a group of three is equivalent to a coefficient of variation of between 7,50 (for symmetrically arranged results) and 8,66 (for results arranged with maximum asymmetry). It is significant that the validity criterion is based on a fraction of the average and not on an absolute difference.

To investigate whether the above criterion for validity is suitable for testing high-strength concrete, 79 sets of cube tests were analysed. Strengths of the sets varied between 50 MPa and 121 MPa.

For each set of three cube results, average strength and the coefficient of variation of the three individual strengths were calculated. Results are shown in Figure 8.42.

It is clear from the figure that coefficient of variation was not related to concrete strength. Average coefficient of variation was 2,7 % and exceeded 5 % in very few sets.

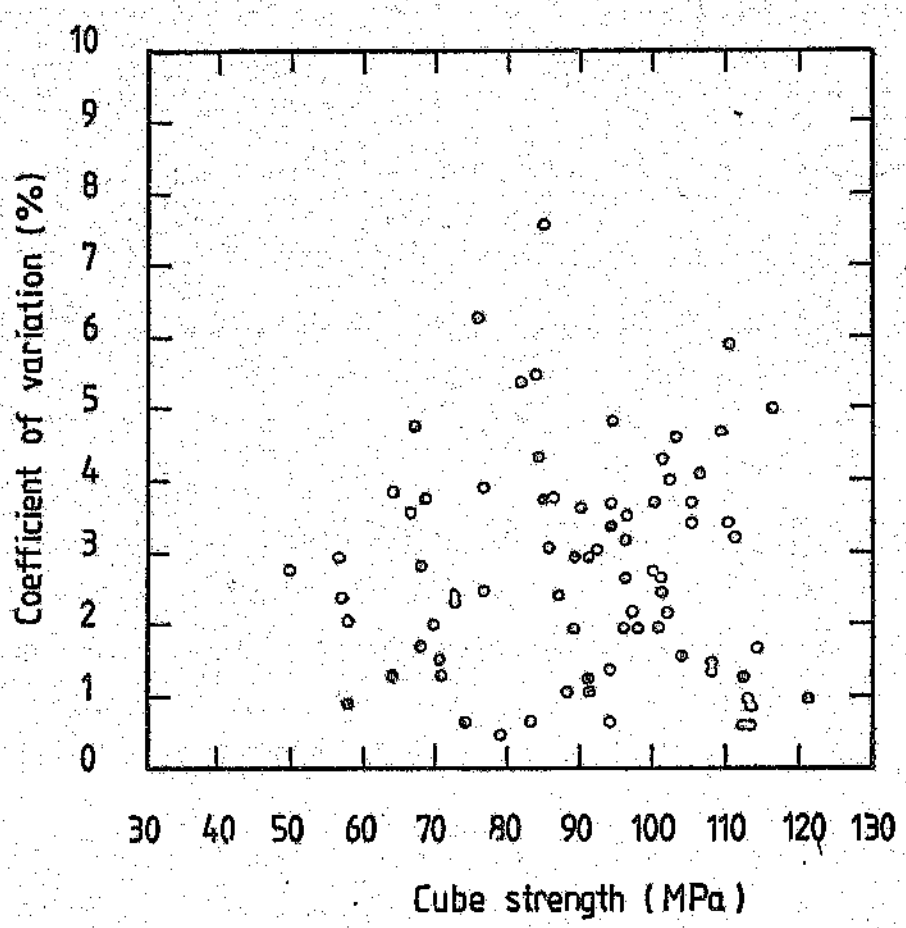


FIGURE 8.42: Coefficient of variation of individual cube strengths within batches of three versus cube strength

It appears therefore that the 15 % range permitted in SABS Method 863 is as appropriate for high–strength concrete as it is for conventional concrete. This has a significant practical implication: the validity criterion used in strength testing of conventional concrete may be used without modification for high–strength concrete.

8.10 Summary and conclusions

This chapter deals with the relative importance of aggregate and hardened cement paste with regard to compressive strength of concrete, and the effect of coarse–

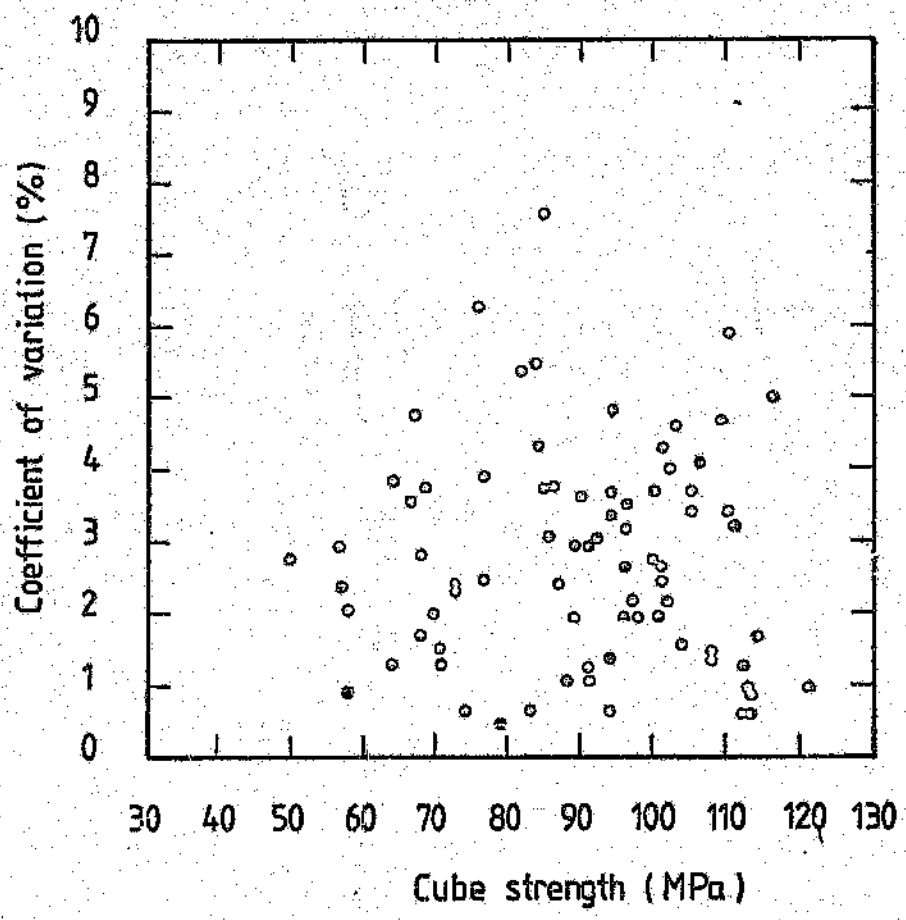


FIGURE 8.42: Coefficient of variation of individual cube strengths within batches of three versus cube strength

It appears therefore that the 15 % range permitted in SABS Method 863 is as appropriate for high–strength concrete as it is for conventional concrete. This has a significant practical implication: the validity criterion used in strength testing of conventional concrete may be used without modification for high–strength concrete.

8.10 Summary and conclusions

This chapter deals with the relative importance of aggregate and hardened cement paste with regard to compressive strength of concrete, and the effect of coarse–

aggregate size on strength and cement efficiency of concrete. The strength and cement–efficiency in concrete performance of local materials is studied by comparing different portland cements; blends of portland cement and different cement extenders; and various aggregate types.

Splitting–tensile strength and flexural strength and their relation to compressive strength are studied using mixes made with different aggregate types.

The testing of high–strength concrete is considered by investigating the effect of specimen aspect ratio on strength; by analysing within–batch variation; and by reporting on a study of strength of in situ concrete.

It was found that aggregate properties play an increasingly important role in influencing strength of concrete as strength increases. The effect of aggregates at C/W = 3,6 (cube strength about 100 MPa) is about one–and–a–half times greater than at C/W = 2,0 (cube strength about 65 MPa). This conclusion is in agreement with the statements found in the literature.

Stone size for optimum strength and cement–efficiency of concrete was investigated by making mixes with different sizes of stone: 26,5 mm; 19 mm; 13,2 mm; and a blend of 26,5 mm and 6,7 mm. Optimum size for strength at 28 days was 26,5 mm or the 26,5/6,7–mm blend. Optimum size for cement–efficiency at 28 days was 26,5 mm for all strengths between about 60 and 110 MPa. These findings are contrary to statements found in the literature which favour smaller stone sizes.

The performances in concrete of rapid–hardening portland cement (RHPC) and blends of this cement with various extenders differed significantly with regard to water requirement, strength and cement–efficiency. Water requirement had a range of 10 to 15 ℓ/m^3 for the various combinations. Cube strength at 28 days, had a range of between 15 and 21 MPa at the C/Ws used. Cement–efficiency factor (CEF) at any given C/W differed by as much as 4,3 MPa/100 kg cementitious material/m^3.

Best strengths at 28 days were achieved with mixes containing a 90/10 blend of RHPC and condensed silica fume (CSF). Best CEF at 28 days was achieved with mixes containing either a 90/10 or a 95/5 blend of RHPC and CSF; plain RHPC mixes however also had high CEF at high values of C/W.

Mixes containing 85/15 and 75/25 blends of RHPC and fly ash and ground granulated blastfurnace slag respectively performed relatively poorly especially with regard to strength.

The work done indicates that the most favourable cementing materials for locally made high-strength concrete are RHPC on its own or RHPC/CSF blends of between 90/10 and 95/5.

Six different RHPCs were compared. Their performances in concrete with regard to water requirement, strength and cement efficiency differed significantly. Water requirement had a range of between 5 and 15 ℓ/m^3. Cube strength at 28 days, for a given C/W, had a range of between 6 and 14 MPa. CEF at 28 days had a range of between 2,4 and 3,7 MPa/100 kg cement/m^3. Although cement from one source gave the highest strengths at virtually all strength levels, no single cement gave the best CEF for the range of C/Ws tested. It was not possible to relate performance to cement characteristics.

Six different aggregate types were compared. Their performance in concrete with regard to water requirement, strength and cement-efficiency differed significantly. Water requirement had a range of 15 to 20 ℓ/m^3. Cube strength at 28 days, for a given C/W, had a range of between 9 and 21 MPa. CEF at 28 days had a range of between 3,1 and 4,5 MPa/100 kg cement/m^3. The most promising aggregate types were andesite, dolerite and dolomite. Quartzite and granite gave the poorest results.

Although there was some correlation between aggregate strength and concrete strength, the degree of correlation was not good enough to make it possible to predict

concrete strength performance or ranking of different aggregates on the basis of aggregate strength.

The overall conclusion after comparing different RHPCs, different cement extenders, and different aggregate types, is that selection of materials for high–strength concrete must be confirmed by testing.

In making the above comparisons, CEF was used as an index of material performance in concrete. It is interesting to compare CEFs achieved in the laboratory with those reported in the literature.

Laboratory CEFs at 28 days, rounded off to whole numbers, were as follows:

Comparison of	CEF (MPa/100 kg cement/m^3)*		
	Lowest	Highest	Lowest by best performer**
Cement extenders	20	26	21
RHPCs	19	26	22
Aggregate types	18	25	23

* Or MPa/100 kg cementitious material/m^3.

** "Best performer" is specific material which produced the highest CEF at any C/W.

Equivalent CEFs reported in the literature[2.34, 2.35, 2.36, 2.37, 2.38, 2.39, 2.40, 2.41] ranged between about 10 and 27 MPa/100 kg cementitious material/m^3.

Compared with CEFs reported in the literature, CEFs achieved with local materials were therefore generally better and highest values were similar.

All mixes made for the investigation contained superplasticizer dosed as recommended by the manufacturer: 2 ℓ of solution to 100 kg cementitious material. This dosage is equivalent to 0,92 % by mass which is slightly less than the 1 % minimum

recommended by El-Zanaty.[2.27] The effect of higher dosages on CEF could be investigated but cost implications would have to be considered because superplasticizer is a relatively expensive ingredient (see chapter 11).

The comparison of CEFs achieved in the laboratory and those reported in the literature indicates that materials suitable for making high-strength concrete are obtainable in South Africa. But careful selection is essential.

Splitting-tensile strength and flexural strength of various concretes were measured and compared with compressive strength. It was found that prediction equations recommended in the literature give values similar to or lower than those achieved.

The effect of high strengths on the testing of concrete is dealt with by studying the influence of specimen aspect ratio on measured strength; by reporting on an extensive investigation into the strength of in situ concrete; and by analysing the within-batch variation of strength specimens tested in the laboratory. The conclusion is that methods of test, developed originally for concretes of much lower strength, are suitable for high-strength concretes.

In summary:
The performance of high-strength concrete is highly specific to the materials used in it and materials must therefore be carefully selected for good performance. High-strength concrete may be tested by conventional methods.

9. MOVEMENT PROPERTIES OF HIGH–STRENGTH CONCRETE

The dimensions of a concrete element are never entirely stable. Dimensional changes occur: initially as the concrete dries out, as a result of sustained loads and changes in loads on the concrete, and as a result of changes in environmental conditions.

Stress–strain behaviour in uniaxial compression is dealt with in the literature survey in chapter 2; see references 2.45 and 2.60 in sections 2.3.2 and 2.4.3 respectively.

In this chapter, three movement–related properties will be discussed. These are modulus of elasticity, creep, and drying shrinkage.

9.1 Modulus of elasticity, creep and shrinkage

9.1.1 Introduction

In attempting to explain the mechanisms involved in the response of hardened concrete to loading within the elastic range, sustained loading, and to partial loss of mixing water, concrete is usually described as a multi–phase or composite material consisting of hardened cement paste and aggregate. This description is however inadequate to characterize concrete made with a very high cement content. As discussed in section 7.2, concrete made with C/W > about 2,5 is cement over–saturated; any cement in excess of about 2,5 times the quantity of mixing water used will remain unhydrated and the paste will not contain capillary pores. (Capillary pores are defined in section 7.2.3.) How significant is the proportion of unhydrated cement in the make–up of hardened concrete? Figure 9.1 shows volume fractions of a theoretical concrete with varying C/W calculated for a constant water content of 150 ℓ/m^3 which is fairly typical of high–strength concretes made with an effective superplasticizer. The figure is based on the assumption that hydration is constrained only by lack of space for the products of hydration.

In the figure, C/W for cement–saturation is taken as 2,5, the value adopted in section 7.2. For 2,0 ≤ C/W ≤ 2,5 with increasing C/W the volume fraction of:

- cement hydrate (ie gel solids and gel pores) increases;
- capillary pores decreases;
- aggregates decreases slightly.

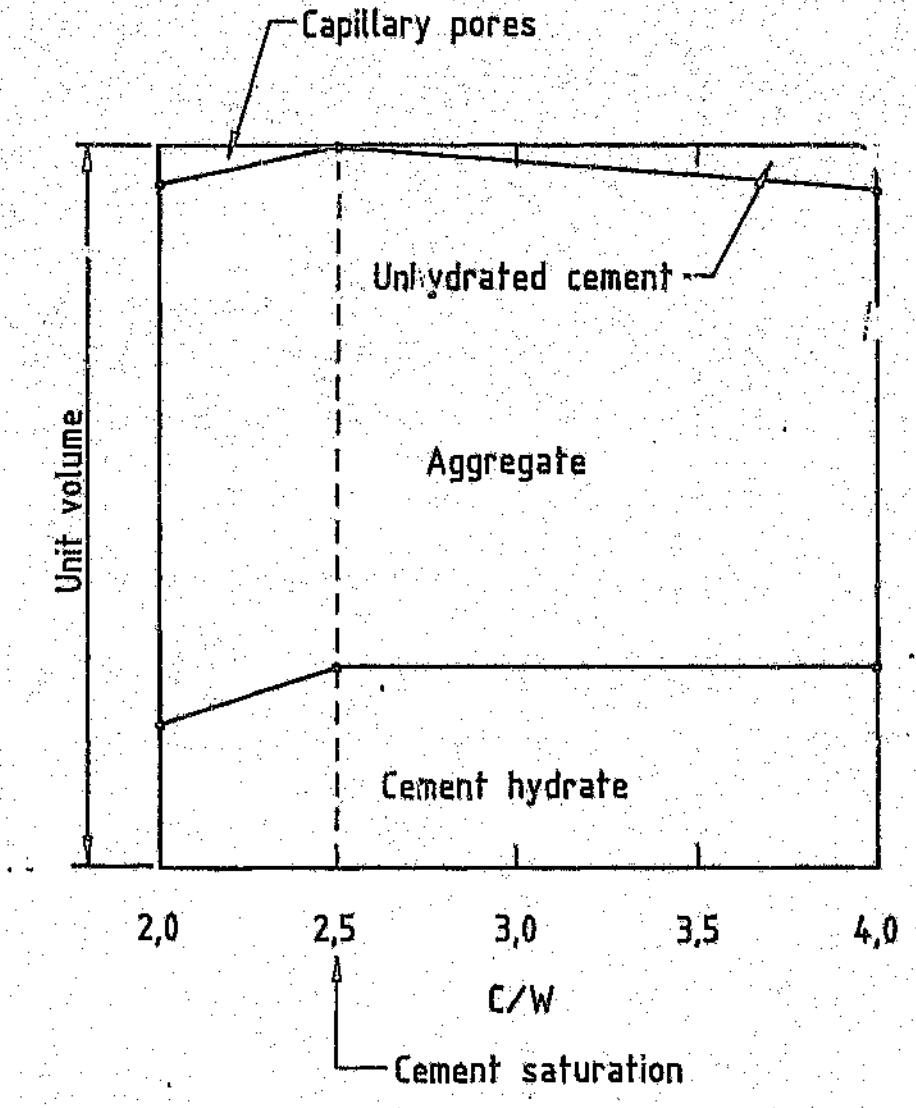


FIGURE 9.1: Volume fractions of phases of hypothetical concrete for $2 \leq C/W \leq 4$; constant volume of mixing water 150 ℓ/m^3

At C/W = 2,5 the mix is cement–saturated and the hardened, fully mature concrete consists only of cement hydrate and aggregate.

For C/W > 2,5, with increasing C/W the volume fraction of:

- hydrate stays constant;
- unhydrated cement increases;
- aggregate decreases.

At C/W = 4,0, for example, volume fractions of hydrate and unhydrated cement are 28 % and 6 % respectively. Unhydrated cement therefore is a relatively small fraction of the concrete at high C/W.

Consider concretes with C/W ≥ 2,5. Figure 9.1 shows that the fraction of pores, including gel pores (provided water content is constant), is constant for all values of C/W. But compressive strength in this zone is not constant.

It appears therefore that the value of C/W for cement–saturation, ie $(C/W)_s$, divides concrete into two categories with fundamentally different properties.

If aggregate and unhydrated cement are together regarded as quasi–aggregate, the unhydrated cement is about 9 % of this. The extent to which the unhydrated cement influences the elastic properties of the quasi–aggregate would depend on how much the elastic properties of aggregate and unhydrated cement differ.

According to the model, for C/W > $(C/W)_s$ concrete properties are as follows:

- Capillary pores are not present in the mature paste.

- Unhydrated cement is present in mature paste and plays the role of fine aggregate which is strongly bound to the surrounding gel.

- An increase in C/W produces an increase in strength by improving the bond between "quasi–aggregate" (true aggregate as well as unhydrated cement) and gel. Strength is not increased by a reduction in porosity.

In fact concrete in which C/W > (C/W)$_a$ appears to be analogous in some ways to steel: the strength of steel depends on structure and boundaries of crystals and not on porosity.

Whether this analogy can be extended to another property, modulus of elasticity, is discussed below.

9.1.2 Modulus of elasticity

Modulus of elasticity determinations were carried out according to BS 1881 Part 5.[9.1]

Figure 9.2 shows results obtained with concretes made with three different aggregate types. Details of mixes are given in Appendix A9.1. Unfortunately only three C/Ws were tested for each aggregate type at each age. The figure does however indicate that it is possible that E could be virtually constant for 2,66 < C/W < 3,5.

To investigate further the elastic properties of concrete a quantity of RHPC was obtained during April 1990. Analysis of the cement, done by the manufacturer, and calculations of compound proportions are given in Appendix A9.2.

To investigate the elastic properties of concrete at various C/Ws made with this cement, a series of mixes was made using andesite aggregate. Intervals between successive C/Ws were made relatively small in order to increase the sensitivity of the investigation.

Proportions of the mixes and the calculated volumetric proportions at maturity are given in Appendix A9.3 and shown graphically in Figure 9.3. Calculations are based on Figures 7.1, 7.2 and 7.3. It should not be inferred that these calculations are exact. As explained in chapter 7, they are merely to indicate the magnitude of porosity.

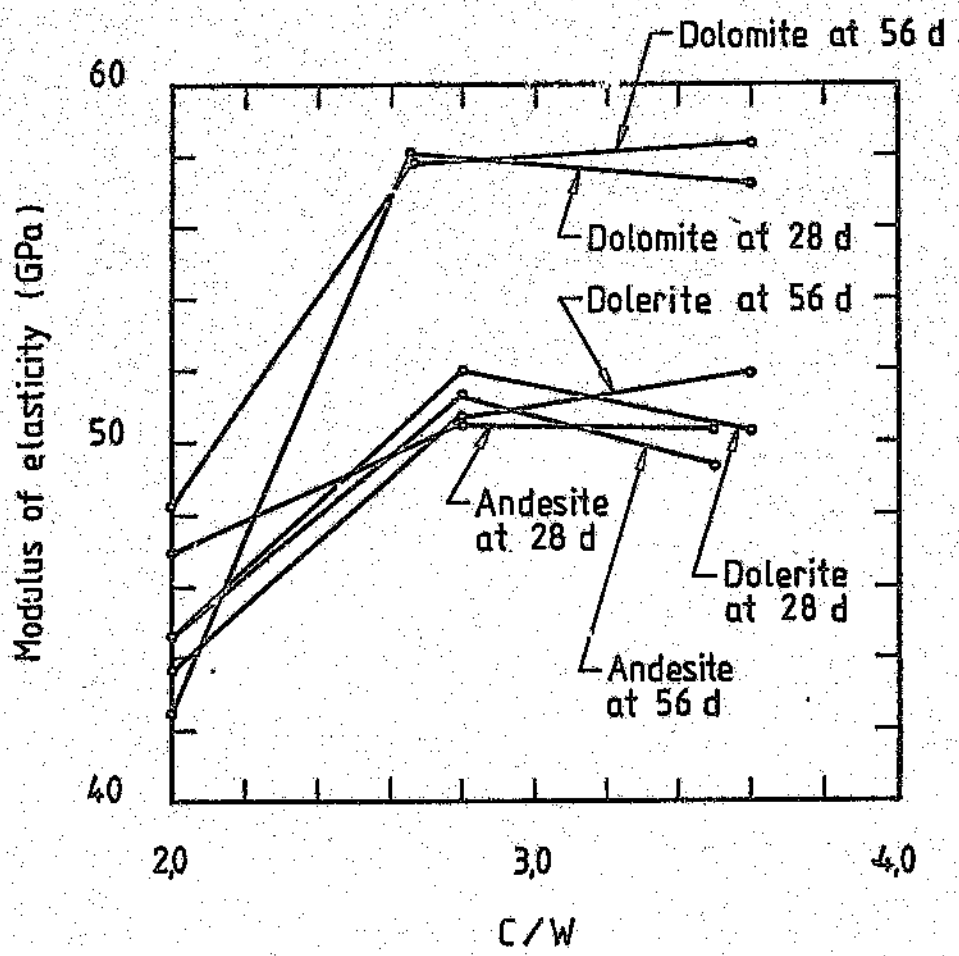


FIGURE 9.2: Modulus of elasticity versus cement:water ratio for concretes made with three different aggregate types and at two different ages

Sets of cubes were crushed at 28 days and modulus of elasticity measured on cylinders at 28 and 56 days. Results are shown in Figure 9.4. The figure also shows predicted modulus of elasticity according to two equations:

$E = 4\,700 \sqrt{f_{cyl}}$ (from reference 2.68)

and $E = 3\,300 \sqrt{f_{cyl}} + 6\,900$ (from references 2.67 and 2.69)

where E = modulus of elasticity at a given age (MPa)

f_{cyl} = cylinder strength at the same age.

To convert to cube strength it was assumed that cylinder strength = cube strength x 0,85.

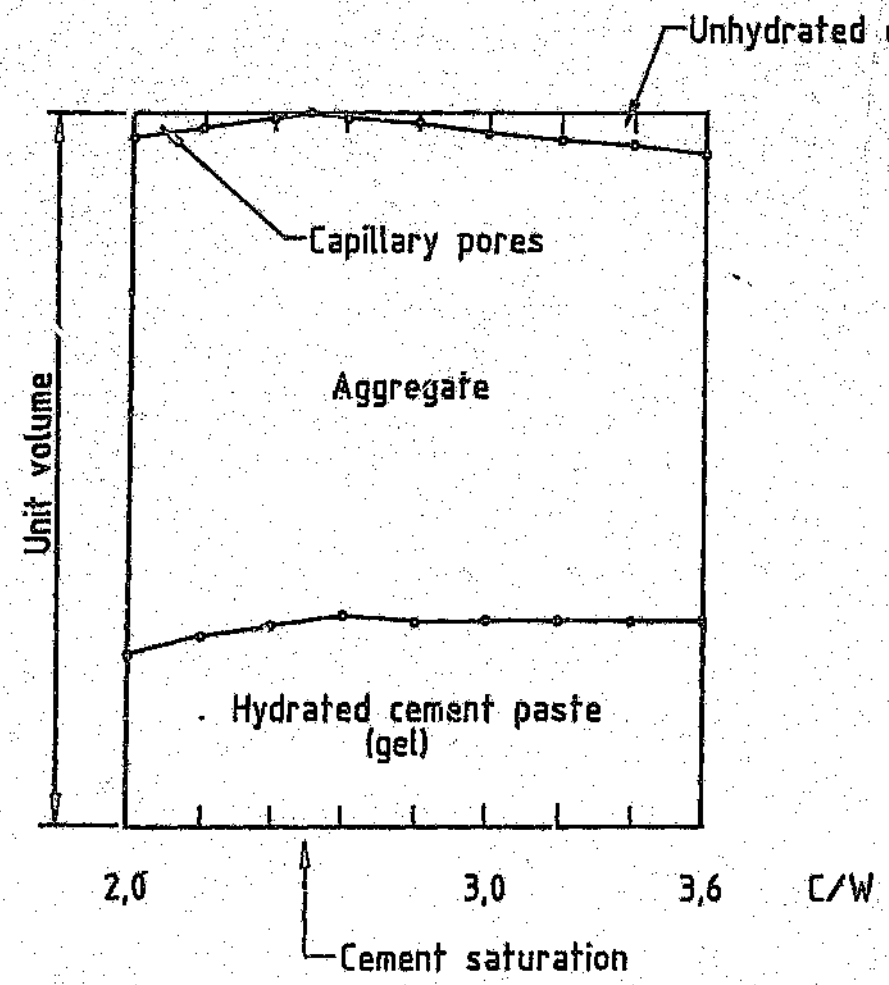


FIGURE 9.3: Calculated volume proportions of actual concrete mixes used for determination of modulus of elasticity as shown in Figure 9.4

From the figure we observe that:

- 28-day strength increased with increasing C/W between C/W = 1,8 and C/W = 3,6. Although the individual points tended to form a somewhat erratic zig-zag pattern, a fair curve drawn through the points shows that rate of strength increase decreased with increasing C/W.

- 28–day modulus of elasticity showed a rather erratic pattern when plotted against C/W.

- 56–day modulus of elasticity showed a more regular pattern when plotted against C/W. If the result at C/W = 2,0 is regarded as somewhat out of phase and ignored, moduli of elasticity increased gradually with increasing C/W between C/W = 1,8 and 2,8. For C/W from 2,8 to 3,6 modulus of elasticity was virtually constant.

- Modulus of elasticity predicted according to reference 2.68 and references 2.67 and 2.69, is too low by about 20 % and 30 % respectively.

Comments are as follows:

The erratic patterns of 28–day results for strength and modulus of elasticity are caused possibly by differing degrees of self desiccation in the various groups of specimens (see section 7.3). These differences could have been the result of slightly different curing regimes at very early ages – for example the length of time between casting and placing specimens in water. With prolonged curing in water these differences would tend to decrease or disappear. (See section 7.3.)

Regarding results for modulus of elasticity we see that the value of C/W at which E changes from an increasing function against C/W to a constant is 2,8. This, of course, is not to say that the curve kinks at exactly this point – it is the C/W at which the particular n... as made. This is in the region of the assumed C/W for cement–saturation, ie 2,5. When one considers the tentative nature of the calculation and the small differences in modulus of elasticity being measured, the difference between 2,5 and 2,8 is small.

What effect could the presence of unhydrated cement have had on modulus of elasticity for C/W greater than cement–saturation? Theoretically, as shown in Appendix A9.3 and Figure 9.3, the volume of unhydrated cement is zero at C/W = 2,5 and increases to about 6 % of the concrete volume (or about 8 % of the volume of

quasi–aggregate) at C/W = 3,6. At such a low concentration the unhydrated cement would have to have elastic properties substantially different from those of the aggregate to make any noticeable difference to the elastic properties of the composite.

The modulus of elasticity of cement clinker was in fact measured on a small prism cut from a lump of clinker and was found to be 62 GPa. This relatively high value, together with the small volume fraction of unhydrated cement, indicates that the effect of unhydrated cement on E is negligible.

Equations given in the literature grossly underestimate modulus of elasticity for the concretes investigated. The equation from reference 2.68 was derived originally for concretes of conventional strength; the equation from references 2.67 and 2.69 for concretes up to 83 MPa cylinder strength (about 100 MPa cube strength. Nevertheless the prediction from reference 2.68 is the more realistic. The study shows that predictor equations should not be used without validation or calibration for specific materials.

In conclusion, when considering the tendencies indicated by Figure 9.3 and 9.4, while taking cognizance of the volumetric characteristics of cement–oversaturated mixes, it may be said that such mixes are analogous to steel: Strength depends on structure and characteristics of interfaces between different phases and not on porosity; modulus of elasticity remains virtually constant for different strengths. This analogy will of course hold only for a series of mixes with the same or nearly similar water content and the same aggregate type.

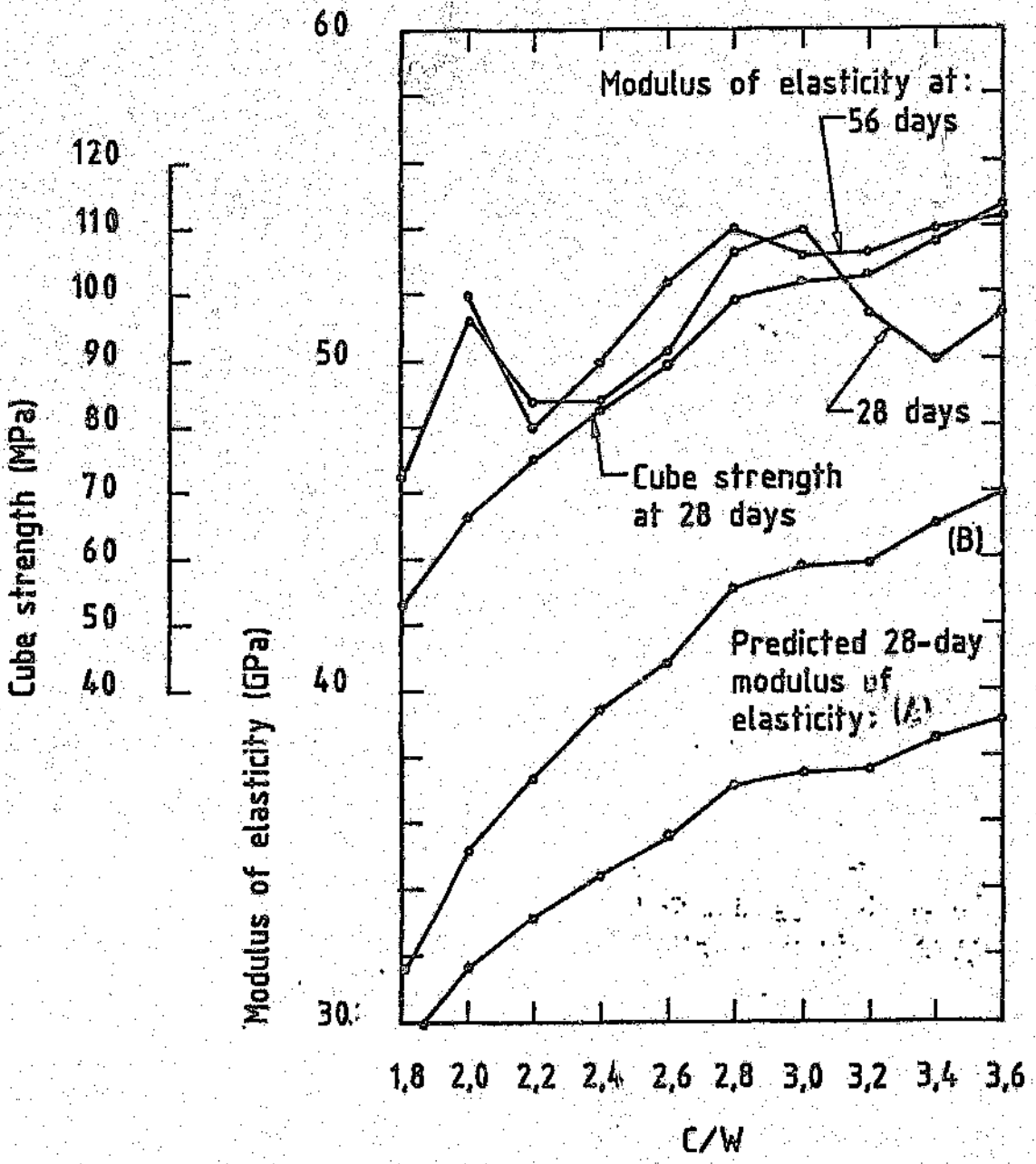


FIGURE 9.4: Modulus of elasticity and cube strength versus cement:water ratio of concretes made with andesite aggregate; predicted modulus of elasticity (A) from reference 2.68, (B) from references 2.67 and 2.69

9.1.2.1 Implications of nearly constant modulus of elasticity for concretes made with C/W greater than C/W for cement saturation

As shown above, as C/W is increased above the value for cement saturation, strength increases while modulus of elasticity stays virtually constant.

In a structural element, higher strengths are exploited by imposing correspondingly higher design stresses.

By definition, strain is equal to stress divided by E.

Therefore if stress is increased while E remains constant, strain will increase. Elements of high–strength concrete with cement–oversaturated paste would consequently exhibit elastic strains higher than those of conventional concrete. Figure 9.4 indicates that this increased strain could be in the order of ten to fifteen per cent. This aspect is discussed in more detail below in section 9.1.4.

9.1.2.2 Estimating the modulus of elasticity of hydrated cement

Using Counto's model[9.2] for elastic stiffness of concrete consisting of a combination of aggregate particles surrounded by hydrated cement, the mixes with $3{,}0 \leq C/W \leq 3{,}6$ in Appendix A9.3 were analysed.

The following assumptions were made:

- The aggregate consists of true aggregate and unhydrated cement (quasi–aggregate as discussed above).

- The effect of the stiffness of unhydrated cement on the stiffness of the quasi–aggregate can be ignored because the volume fraction of unhydrated cement in the quasi–aggregate is so small.

- The modulus of elasticity of the quasi aggregate is 100 GPa. (Reference 9.3 gives a value for andesite of 95 to 105 GPa.)

According to Counto's model[9.2]

$$\frac{1}{E_c} = \frac{1 - \sqrt{V_a}}{E_m} + \frac{1}{[(1 - \sqrt{V_a}) / \sqrt{V_a}]\, E_m + E_a} \quad \text{.....(9.1)}$$

where E_c and E_m are modulus of elasticity of the concrete and matrix respectively; and V_a is the volume fraction of aggregate.

Using the values E_c = 54 GPA
E_a = 100 GPA
V_a = 0,712

the equation was solved for E_m and it was found that E_m = 18 GPa.

Another approach was also tried. Cement clinker was crushed and material passing through a 6,7–mm sieve was used to make a high–strength mortar. Proportions by solid volumes of the mortar were:

water	280 ℓ/m³
cement	280 ℓ/m³
clinker	440 ℓ/m³.

Cement:water ratio was 3,28 and a sulphonated naphthalene formaldehyde superplasticizer was used to improve workability. Measured properties of the mortar were:

7–day cube strength	98 MPa
28–day cube strength	116 MPa
28–day modulus of elasticity	43,5 GPa

By substituting the values

E_c = 43,5 GPa
E_a = 62 GPa
V_a = 0,456 (clinker plus unhydrated cement)

in equation 9.1, modulus of elasticity of hydrated cement paste was calculated to be 32 GPa.

Hirsch[9.4] determined the modulus of elasticity by direct measurement of paste specimens. He found that for a water:cement ratio of 0,4 (which is close to the point of cement–saturation, see chapter 7) the modulus of elasticity of paste at 28 days was 19 GPa.

The value calculated above (ie 18 GPa) therefore appears to be more realistic than the value based on the modulus of elasticity of clinker. A possible reason for the lack of agreement is the uncertainty involved in measuring modulus of elasticity of clinker. When doing the test it was difficult to find an uncracked specimen large enough for the test. Consequently only one specimen was tested. It is also possible that the specimen which was tested had flaws which were not apparent. (The specimen subsequently disintegrated after being kept for some months on a shelf in the office.)

9.1.3 Creep and shrinkage

9.1.3.1 Comparison of three aggregate types

Tests

Six concrete prisms 100 mm x 100 mm x 200 mm were cast with concrete mixes made with three different aggregate types: dolomite, andesite and dolerite; at three different C/Ws: 2,0, 2,66 or 2,8, and 3,5 or 3,6. Details of mixes are given in Appendix A9.1. Of each set of six prisms, three were used for creep tests and three for shrinkage. Prisms were cured in water for 28 days. Creep prisms were loaded at age 28 days into creep frames after their top surfaces as cast had been ground flat. At the same time, companion prisms, for monitoring drying shrinkage of the concrete, were placed on racks in the same room after their ends had been sealed.

Temperature and relative humidity in the test room were controlled for the duration of the test at 23 ± 1 °C and 60 ± 5 % respectively.

Length changes between targets glued to the surface on two opposite faces of each prism were measured by means of a Demec gauge (100–mm gauge length). Measurements were taken at the time of loading firstly when prisms were subject to a small preload stress and secondly when the sustained stress was applied. Thereafter, measurements were taken at 1, 3, 7, 14, 28 and 180 days after loading.

Results

Results of 28–day strength, secant modulus, strains and calculated creep factor, specific creep and creep strain per unit stress:strength ratio are given in Tables A9.4.1 to A9.4.9 of Appendix A9.4. The results of shrinkage and creep tests for the andesite concrete at C/W = 3,5 were discarded because the readings showed no shrinkage at all between 14 and 180 days (Table A9.4.6). This anomaly could have been the result of a loose target or a measuring error.

Results of specific creep and creep factor at 180 days after loading are shown in Figure 9.5.

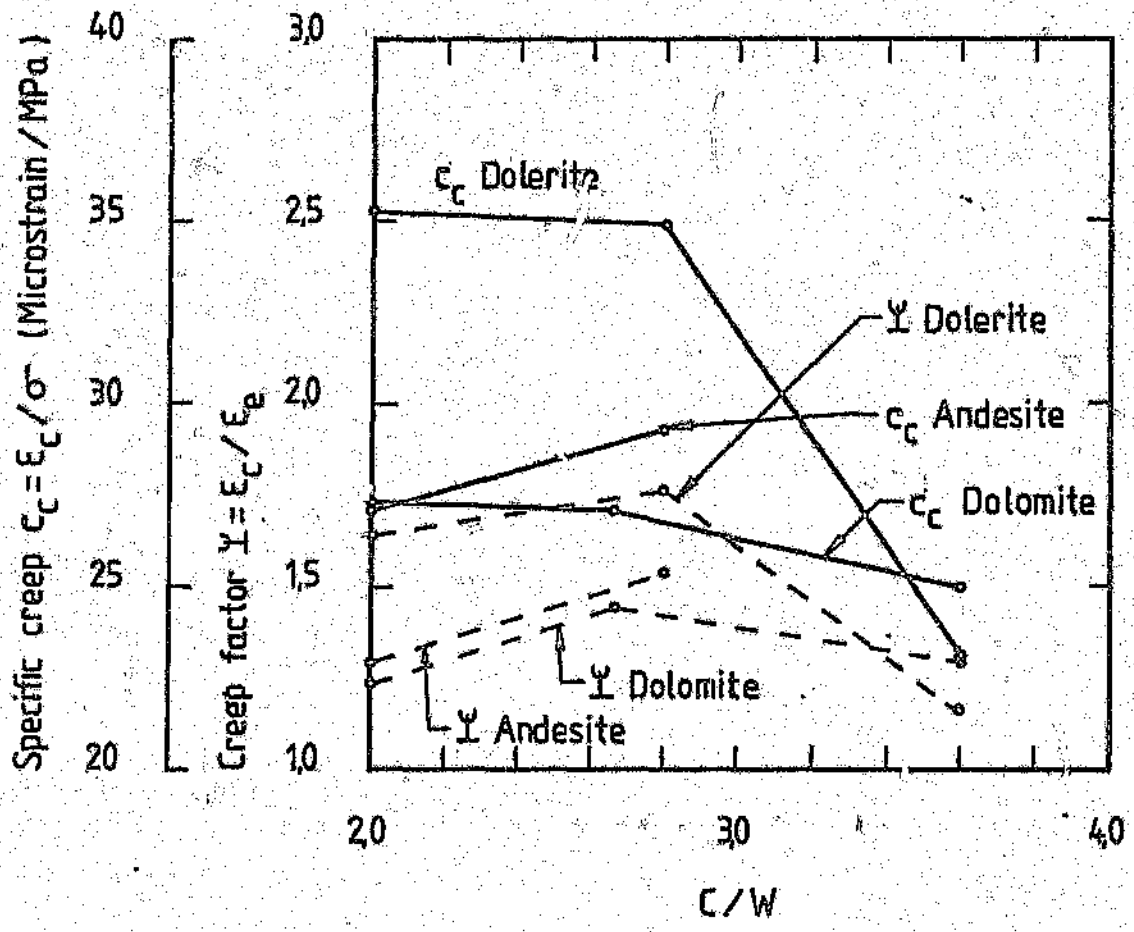


FIGURE 9.5: Specific creep and creep factor at 180 days of loading versus cement:water ratio for concretes made with three different aggregate types

Results of shrinkage at 180 days of drying are shown in Figure 9.6.

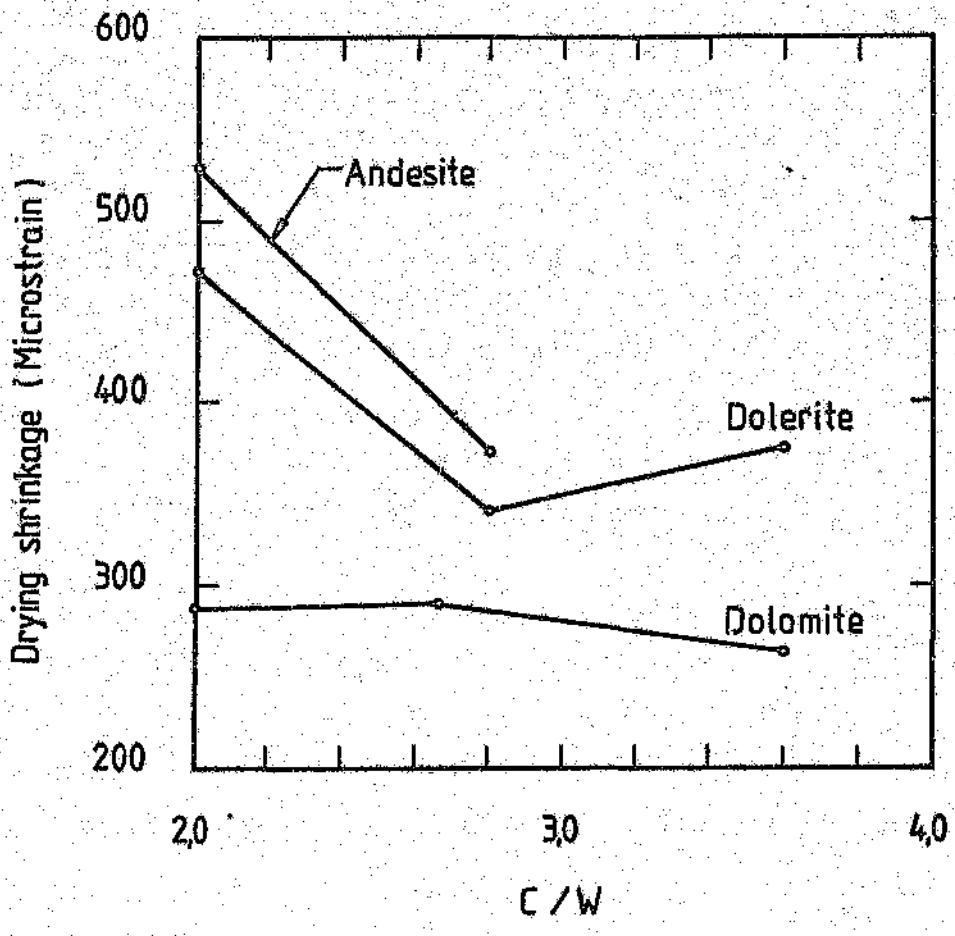


FIGURE 9.6: Shrinkage after 180 days of drying versus cement:water ratio for concretes made with three different aggregate types

Comments

Creep factors at 180 days of loading ranged between approximately 1,15 and 1,75. At C/W = 2,0 and about 2,8, dolerite concretes had the highest values followed by andesite and dolomite but dolerite had the lowest value at C/W about 3,6. All three aggregate types showed the highest creep factor at C/W about 2,8.

Specific creep at 180 days of loading ranged between approximately 23 and 35 microstrain per MPa. At C/W = 2,0 and about 2,8, dolerite concretes had the highest

specific creep by a considerable margin. But at C/W ≈ 3,6 the dolerite concrete had the lowest specific creep by a small margin.

For both creep factor and specific creep the highest value was about 1,5 times the lowest value. This large difference indicates that specific mixes should be tested in order to achieve meaningful predictions of creep in structures.

Creep strain per unit stress:strength ratio was generally lowest for dolomite concrete and generally the highest for dolerite concretes; but at C/W = 3,6 was virtually the same for dolomite and dolerite concretes. Creep strain per unit stress:strength ratio generally tended to increase with increasing C/W but there were too few data to draw any conclusions. This aspect is dealt with in the next section.

Shrinkage at days of drying ranged between about 300 and 500 microstrain. At C/W = 2,0 and about 2,8, andesite concrete had the highest shrinkage followed closely by dolerite. Dolomite concrete had the lowest shrinkage at all values of C/W.

The differences in creep and shrinkage between concretes made with the three aggregate types could be caused by differences in aggregate properties or differences in paste (or water) content, or both.

To see whether paste or water content played an important role, water contents of the nine mixes were compared. (At a given C/W, paste content is determined by water content.) It can be seen that the dolomite mixes had the lowest water content at all values of C/W that were used. If creep and shrinkage were completely dependent on water content, then the ratios of creep:water content and shrinkage:water content would be independent of aggregate type, ie would be the same for all aggregate types at a given value of C/W. Water content, specific creep, shrinkage and ratios of specific creep and shrinkage to water content are shown in Table 9.1.

Table 9.1: Water content, creep and shrinkage of mixes; results are shown in Figures 9.5 and 9.6

Aggregate type	C/W	W Water content (ℓ/m^3)	c_c Specific creep (Microstrain per MPa)	Shrinkage (Microstrain)	$c_c \div W$	Shrinkage $\div$ W
Dolomite	2,0	150	27,3	287	0,182	1,91
	2,66	146	27,1	290	0,186	1,99
	3,6	150	25,0	263	0,167	1,75
Andesite	2,0	160	27,1	529	0,169	3,31
	2,8	150	29,3	373	0,195	2,49
	3,5	161	–	–	–	–
Dolerite	2,0	175	35,3	472	0,202	2,70
	2,8	170	34,9	341	0,205	2,01
	3,6	160	23,1	376	0,144	2,35

In the table, compare the range of specific creep values with the range of specific creep:water content ratios at C/W = 2 by calculating the ratio of highest to lowest value for each:

$$\text{Specific creep} \quad \frac{35,3}{27,1} = 1,30$$

$$\text{Specific creep.water content} \quad \frac{0,202}{0,169} = 1,20$$

Normalizing shrinkage results by dividing by water content therefore reduces the differences but not to a significant extent.

The above comparisons indicate that water content does influence creep and shrinkage but to a limited extent. Aggregate properties probably account for most of the differences between the mixes tested.

This series of tests provided a comparison between the behaviour of concretes made with three different aggregate types. But the increments in C/W used were too large to provide information on creep and shrinkage relative to C/W. A further series of tests was therefore carried out with one aggregate type and much smaller increments in C/W. This work is described in section 9.1.3.2.

9.1.3.2 Influence of cement:water ratio

Tests

A series of mixes, as shown in Appendix A9.3, was made up using andesite aggregates. Six prisms and three cubes were made with each mix. Tests for creep and shrinkage were carried out as described in section 9.1.3.1. except that these tests were started at age 56 days instead of 28 days. It was hoped that this new procedure would eliminate some of the erratic tendencies caused possibly by differences in curing regime at very early ages. Strength tests were done in accordance with SABS Method 863.[6.17]

Readings of strain were taken 0, 15, 28, 56, 83, 160 and 180 days after commencement of the test.

Results

Results of 28-day and 56-day strength, secant modulus, shrinkage and creep strains and calculated specific creep, creep factor and creep strain per unit stress:strength ratio are given in Appendix A9.5.

Drying shrinkage and water content are shown in relation to cement:water ratio in Figure 9.7.

Specific creep and creep factor are shown in relation to cement:water ratio in Figure 9.8.

Creep strain per unit stress:strength ratio is shown in relation to cement:water ratio in Figure 9.9.

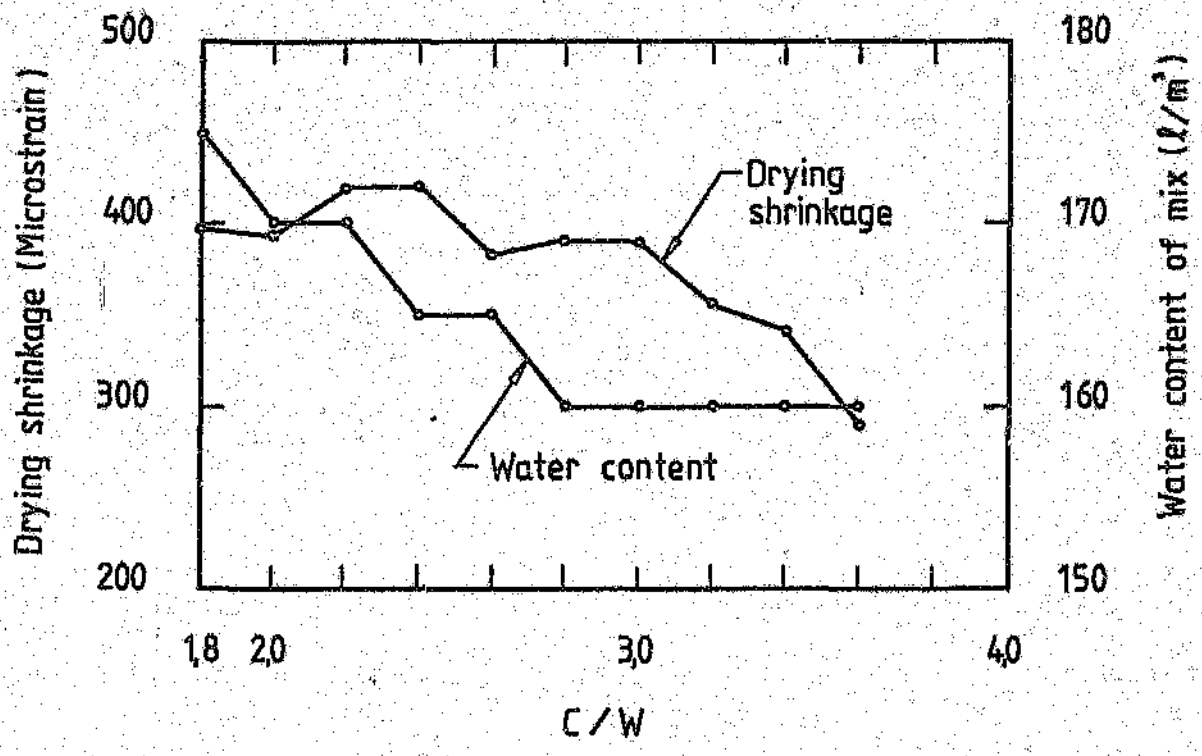


FIGURE 9.7: Drying shrinkage and content of mixing water versus cement:water ratio for concretes made with andesite aggregate

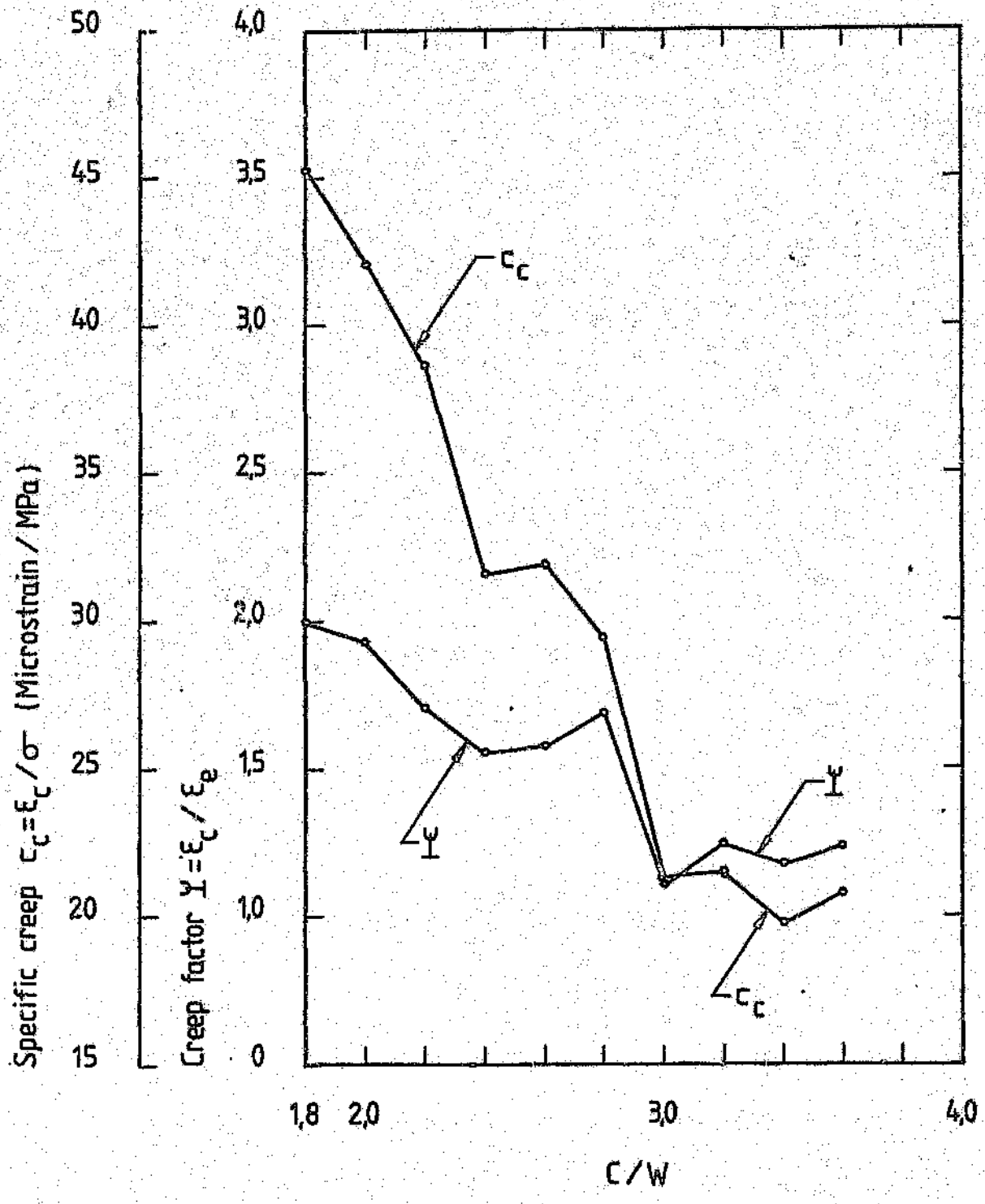


FIGURE 9.8: Specific creep and creep factor versus cement:water ratio for concretes made with andesite aggregates

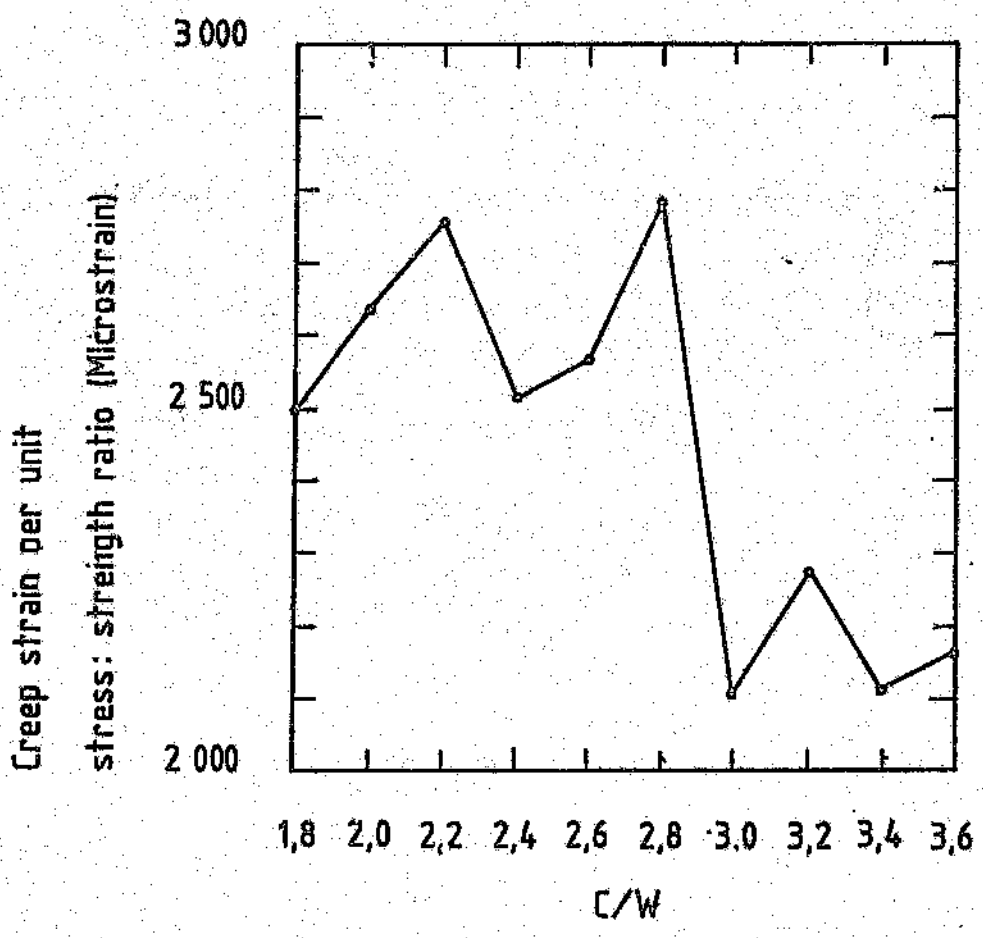


FIGURE 9.9: Creep strain per unit stress:strength ratio versus cement:water ratio for concretes made with andesite aggregate

Observations

Drying shrinkage

Drying shrinkage (see Figure 9.7) showed a distinct overall decrease with increasing cement:water ratio although shrinkage was virtually constant for 1,8 ≤ C/W ≤ 3,0. Individual results were reasonably consistent and deviated by less than 5 % from a smooth curve drawn through the points. Although water content also decreased with increasing cement:water ratio the two trends were not in parallel. In fact, for 2,8 ≤ C/W ≥ 3,6, water content remained constant while shrinkage decreased substantially.

For $1{,}8 \leq C/W \leq 2{,}8$ the roles were reversed: water content decreased while shrinkage remained virtually unchanged.

A proposed explanation of this behaviour is as follows:

In cement–undersaturated mixes (C/W < about 2,5) an increase in cement content effects an increase in gel content and a consequent tendency to greater shrinkage. On the other hand, if water content is reduced as cement:water ratio is increased, shrinkage would tend to decrease. In the concretes investigated, the effects of greater gel content and reduced water content tended to cancel each other out and virtually constant shrinkage resulted.

In cement–oversaturated mixes (C/W > about 2,5) gel content is constant if water content is constant, which was virtually the case in the mixes investigated. On the other hand gel–aggregate interface area becomes smaller as cement:water ratio is increased.

Because shrinkage is the result of water lost from a concrete specimen or element, the rate of water loss, and therefore the amount of water lost in a given time, is a function of the permeability of the concrete. Gel–aggregate interfaces are probably zones of relatively high permeability and if the area of these zones per unit volume of concrete is reduced, a decrease in permeability could be expected.

Total paste content of the mixes considered increased as C/W increased; rate of increase increased slightly with increasing C/W (see Appendix A9.3). Total paste content therefore does not correlate at all with shrinkage.

According to the above hypothesis, the decrease in shrinkage with increasing cement:water ratio was therefore a result of the decrease in permeability.

At C/W = 2,0 the shrinkage of andesite concrete is about 530 and 400 microstrain according to Figures 9.6 and 9.7 respectively. The two mixes were made with

different water contents but the second mix (Figure 9.7) had the higher water content and could therefore had been expected to have the higher shrinkage. A possible reason for the lower shrinkage of the second mix is that it was cured in water for 56 days before being allowed to dry out compared with 28 days of curing for the first mix. The longer curing would have produced a more impermeable concrete that retarded moisture loss and therefore shrinkage.

Creep

As shown in Figure 9.8, specific creep decreased substantially with increasing cement:water ratio for $1{,}8 \leq C/W \leq 3{,}0$ but remained virtually constant for $3{,}0 \leq C/W \leq 3{,}6$.

As explained in chapter 7, the use of a value of 2,5 for C/W for cement–saturation is a rough approximation intended to illustrate the magnitude of porosity of hardened cement paste. If one accepts that $C/W \approx 3$ is still of the right order for cement–saturation, the concretes showed distinctly different characteristics in the cement–undersaturated and cement–oversaturated zones.

In the case of cement–undersaturated concrete, creep is a direct function of the paste fraction of the concrete and an inverse function of strength of the paste. Strength, in turn, is an inverse function of the volume fraction of capillary pores in the paste. As cement:water ratio is increased, the paste fraction increases marginally but strength increases substantially as the fraction of pores is reduced. The effect of the strength increase overrides the effect of increasing paste fraction and specific creep decreases with increasing cement:water ratio. The important point is that creep is influenced by porosity.

In the case of cement–oversaturated concrete, paste volume remains constant if water is kept constant and capillary porosity is zero for all values of cement:water ratio. Strength increases only marginally with increasing cement:water ratio. As discussed previously, this increase in strength is not related to a reduction in porosity but to improved interfacial properties.

Creep remains virtually constant with increasing cement:water ratio in cement–oversaturated mixes because there is no change in porosity. Creep is therefore similar to modulus of elasticity in this respect (see section 9.1.2).

Creep factor, which is determined by specific creep and modulus of elasticity, also shows a pattern similar to that of specific creep with increasing cement:water ratio.

Creep strain per unit stress:strength ratio was rather erratic in relation to cement:water ratio (Figure 9.9); values were generally higher at 1,8 ≤ C/W ≤ 2,8 than at 3,0 ≤ C/W ≤ 3,6. The significance of this is that concretes at the higher range of C/W, ie those with the highest strengths of those considered, would tend to exhibit less creep than concretes at the lower range of C/W if loaded to the same stress:strength ratio. Conversely higher–strength concretes could be loaded sooner than lower strength concretes without an adverse effect (ie an increase) on creep.

Overall paste content increased with increasing C/W for the mixes considered. Because the phenomenon of creep is confined to the hardened cement paste, one would therefore expect creep to increase with increasing C/W. This was not the case – creep tended to decrease with increasing paste content. A possible explanation is that the movement of pore water is inhibited by the dense, highly impermeable nature of the paste. Because creep is associated with the movement of water,[9.5] creep of high–strength pastes is less than that of pastes of conventional strength. This effect outweighs the effect of increased paste volume.

If specific creep of andesite concrete as shown in Figures 9.5 and 9.8 is compared, it can be seen that specific creep is very similar at C/W = 2,8 but significantly different at C/W = 2,0 : 42 and 27 microstrain/MPa for the first and second mixes (Figures 9.6 and 9.8) respectively. Factors which could have played a role in causing this difference are:

- Different RHPCs were used for the first and second mixes.

- The SNF admixture was used in solution form for the first mix and powder form for the second. It is possible that the formulation of the SNF could have been different in the two forms. It is however unlikely that this was a significant factor.

- The specimens in the first test were loaded after 28 days curing in water. In the second test they were loaded after 56 days curing. Concretes loaded at later ages are believed to creep less than similar concretes loaded at earlier ages.

Whatever the relative importance of these factors, the differences emphasise the importance of testing with specific materials and mixes and maintaining curing and loading regimes as close as possible to those expected for a proposed structure.

9.1.4 Design Implications

The designer of a structure requires information on elastic, creep and shrinkage strains to be able to predict structural deformations such as beam deflections and shortening of columns, and loss of prestress where relevant.

A question that comes to mind when designers use high-strength concrete is how the strains shown by this material compare with those of conventional concrete.

To make a comparison, calculations were made, based on the properties of the andesite concrete mixes used to investigate elastic modulus, creep and shrinkage in sections 9.1.2 and 9.1.3 above. Characteristic strengths at 28 days and their corresponding cement:water ratios were derived from Figure 9.4 as described in section 11.2.2. Moduli of elasticity were taken from Figure 9.4. Shrinkage and specific creep were derived from smooth curves drawn through the points on Figures 9.7 and 9.8 respectively. The calculations are tabulated in Appendix A9.6. Strains resulting from shrinkage and an imposed uniform compressive stress of 0,4 times the characteristic 28-day strength are shown graphically in relation to strength in Figure 9.10.

As shown in the figure, shrinkage strain remains virtually constant for characteristic strengths from 40 MPa to 90 MPa but shrinkage of 100–MPa concrete is about 25 % lower.

Elastic strain increases virtually linearly as characteristic strength increases from 40 MPa to 100 MPa. The reason for this is that curves for strength versus cement:water ratio and modulus of elasticity versus cement:water ratio are similar in shape.

As characteristic strength is increased from 40 to 70 MPa, creep strain increases, but at a decreasing rate, and reaches a maximum at 70 MPa. A further increase in strength to 90 MPa results in a decrease in creep strain which remains approximately constant as strength is increased to 100 MPa.

Creep strain is about twice elastic strain at strengths between 40 MPa and 70 MPa, but similar to elastic strain at a strength of 100 MPa.

Total strain, which is what the designer is interested in, increases from about 1 500 microstrain at 40 MPa characteristic strength to about 2 000 microstrain at 70 to 80 MPa and then decreases slightly to about 1 900 microstrain at 90 to 100 MPa. For practical purposes, the relation between total strain and characteristic strength can be regarded as one of linear increase between 40 MPa and 70 MPa and constant between 70 MPa and 100 MPa. It should be noted that the values shown in Figure 9.10 and the above approximation are based on a specific age at loading and duration of loading are therefore not necessarily applicable generally.

Predicted creep and shrinkage strains

Creep strains as predicted in two ways are shown in Appendix A9.6.

The first prediction, from reference 9.6, gives

$$\text{30–year creep} = \frac{\text{stress x creep factor}}{\text{modulus of elasticity at time of loading}}$$

From the reference, creep factor for thin sections is 2,2.

The second prediction, also from reference 9.6, is measured creep divided by 0,6. (The reference assumes that creep at 6 months is 60 % of final creep.)

Assuming that the second prediction is the more reliable because it is based on actual measurements, the first prediction method underestimates creep for characteristic strengths up to 80 MPa and overestimates creep for 90– and 100–MPa characteristic strengths.

Predicted shrinkage strains for thin sections are, according to reference 9.7, about 160 and 390 microstrain at 6 months and 30 years respectively. Measured shrinkage strains at 6 months ranged between 290 and 415 [illegible] far greater than the predicted value of 160 microstrain.

9.1.5 Conclusions

High–strength concretes can be divided into two categories with different movement–related characteristics. These categories are cement–undersaturated mixes and cement–oversaturated mixes.

Not only compressive strength but modulus of elasticity of cement–undersaturated mixes are strongly influenced by the capillary pores present in cement–undersaturated mixes. An increase in cement content produces more hydrate and so reduces capillary porosity. Consequently strength and modulus of elasticity increase while specific creep decreases.

Theoretically, as discussed in chapter 7, cement–oversaturated mixes contain no capillary pores. An increase in cement content can therefore result only in an increase in the volume of unhydrated cement. An increase in cement results in a relatively small increase in strength because of an improvement in interface strength but it does not cause an increase in modulus of elasticity or a decrease in specific creep. In fact these properties remain approximately constant for cement–

oversaturated mixes. If we compare two mixes: one with cement:water ratio at the point of cement--saturation and one with a much higher cement:water ratio; the latter, because of its higher strength can be stressed higher than the former and will consequently exhibit greater elastic and creep strain.

If results obtained in the investigations undertaken are compared with predicted values, it can be seen that tests with specific materials are preferable to generalized predictions.

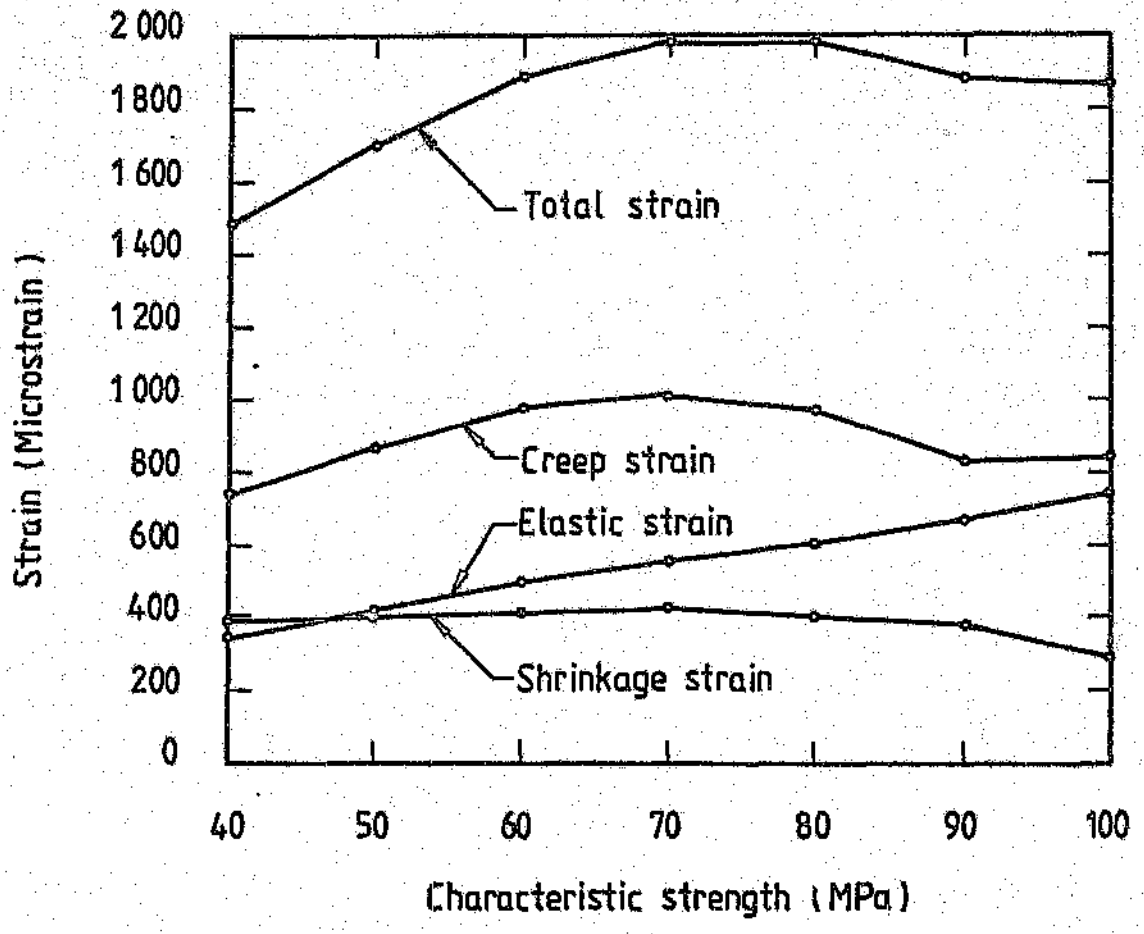


FIGURE 9.10: Shrinkage, elastic, creep and total strain of concrete versus characteristic strength; concrete made with andesite aggregate; imposed stress 0,4 times characteristic strength; concrete cured for 56 days and then loaded; duration of loading 180 days

10. DURABILITY

In this discussion, mechanisms such as impact, overloading and fire, which cause rapid deterioration, are excluded. The mechanisms discussed are those which result in a gradual and often imperceptibly slow deterioration of a concrete element or structure.

Gradual deterioration can be caused by mechanical or chemical means.

10.1 Mechanical deterioration

Gradual mechanical deterioration is caused by abrasive action. Because the mechanism of attack is purely mechanical we could expect strength to be an indicator of abrasion resistance. This supposition is validated by Gjørv et al[2,87] who found the wear resistance of high-strength concrete to be vastly superior to normal-strength concrete.

10.2 Chemical deterioration

It is convenient to examine chemically induced deterioration of concrete in terms of scale. We can consider deterioration of:

- the cement paste;
- the aggregate-paste composite, ie the concrete;
- and the (reinforced) concrete structure.

Paste can deteriorate as a result of an ingress of substances which either:[10.1]

- convert solid components of the paste to water-soluble substances which can be removed by leaching;

- or react with components of the paste to form substances with an increased volume which exert internal expansive pressures on the paste and so cause spalling.

Concrete can deteriorate when:

- the paste is attacked in one of the ways described above;

- aggregate and paste are chemically incompatible and the resulting reactions cause internal swelling and cracking (eg alkali silica reaction);

- crystals form in pores and voids and in so doing exert internal pressures which cause spalling (eg crystallization of salts and freezing of water).

Reinforced concrete can deteriorate when:

- the concrete deteriorates in one of the ways described above;

- reinforcement corrodes and in doing so reduces in cross-sectional area while the corrosion products exert swelling pressures which cause spalling of the concrete.

With the exception of alkali silica reaction, which is usually a result of incompatibility of cement paste and aggregates, all of the above mechanisms of deterioration involve the movement of substances into and through the concrete. These substances may be in the form of a gas, eg air, carbon dioxide, oxygen and water vapour; fluids, usually water; and ions dissolved in water, eg chlorides and sulphates.

Examples are pure water and acid solutions which cause leaching of the paste; sulphate solutions which cause spalling; carbon dioxide which carbonates the concrete and so reduces the pH and destroys the passivating layer on steel reinforcement; and chlorides which promote corrosion of reinforcement.

The transport of substances through concrete can be driven by a difference in pressure, a difference in concentration, or by capillary suction. (It is of course possible that more than one of these mechanisms could operate simultaneously).

Substances move through the concrete in capillary pores, voids caused by incomplete compaction, and cracks. The rate of movement for a given driving force (ie permeability) is determined by the total volume fraction of pores, voids and cracks and the arrangement of these features, especially whether they are continuous or discontinuous.

The theoretical volume fraction of capillary pores is a function of cement:water ratio and volume of mixing water per cubic metre of concrete. At cement:water ratio for cement-saturation (see chapter 7) or higher the volume of capillary pores is theoretically zero. Cement:water ratio of high-strength concrete is typically at or above the value for cement saturation. High-strength concrete is therefore, in theory at any rate, highly impermeable if properly compacted.

A study of the microstructure of high-strength concrete by scanning electron microscope[2.42] confirmed that not only is the paste very dense, but that the interface between aggregate and paste is free of the weak calcium hydroxide zone found in normal-strength concretes.

Microcracking, which could result in an increase in permeability, does not happen in high-strength concrete in the same way as it happens in normal-strength concrete.[2.45] As concrete is loaded, microcracks develop at the interface between coarse aggregate and mortar. For low-strength concrete this microcracking starts at about 30 % of ultimate strength; for high-strength concrete, there is very little cracking at loads below 65 % of ultimate. Under normal service stresses, microcracking of high-strength concrete should therefore be minimal.

It appears therefore that high-strength concrete, if well compacted, should be as impermeable as concrete can be.

This approach is indirectly recognized by various codes of practice in their recommendations for cement:water ratio for highly aggressive conditions. References 10.2, 10.3 and 10.4 are some examples where high cement:water ratios are

recommended (although these ratios are lower than those normally used for high-strength concrete).

Resistance to freezing and thawing appears to be the only durability-related property of high-strength concrete which is in question. Confusing results are given in some references[2.93, 2.96] and ACI Committee 363[2.102] states that research should be done to determine levels of air-entrainment required for high-strength concrete.

10.3 Conclusion

In summary it can be said that high-strength concrete, because of its extremely good impermeability, should have excellent durability. Ingredients must however be selected in order to prevent alkali silica reaction. Durability under freeze-thaw conditions is uncertain but this aspect is of little practical importance in South Africa.

11. ECONOMIC IMPLICATIONS

11.1 Introduction

High–strength concrete costs more per unit volume than conventional concrete. Differences in cost are mainly because of the higher cement contents and the use of expensive superplasticizing admixtures involved in the manufacture of high–strength concrete. Costs of aggregates and quality control also influence total costs. Because aggregate properties play a more significant role in influencing properties of high–strength concrete, it may be necessary to transport selected aggregates over long distances and this would further increase the cost of high–strength concrete. Quality control of high–strength concrete is of critical importance and could cost more than that of conventional concrete.

In many areas and for many uses, the benefits of high–strength concrete more than compensate for the increased unit cost. Basically, high–strength concrete will carry a compression load at less cost than conventional concrete. The actual cost benefits would however be specific to each situation and would depend on current material costs, material performance and current market forces.

According to reference 2.99, economic considerations were behind the development of high–strength concrete for use in buildings in Chicago. References 2.100 and 2.101 show results of cost studies which indicate that high–strength concrete is an economical alternative to concrete of normal strength. These studies are based on prices and concrete properties applicable at the time in the United States of America; they do not necessarily apply to South African conditions. A cost study based on local prices and concretes is described in the next section.

11.2 Cost of columns based on South African conditions

The cost implications of using South African materials are investigated below by designing a hypothetical column with concretes of different strengths and applying current (1991) prices of materials in the Pretoria–Witwatersrand–Vereeniging area.

11.2.1 Design

According to SABS 0100,[11.1] the ultimate axial load for a short column is given by the formula

$$N = 0{,}4\ f_{cu}\ A_c + 0{,}67\ A_{sc}\ f_y \quad \text{.....(11.1)}$$

where

N = ultimate axial load (N)
f_{cu} = characteristic strength of concrete (N/mm^2)
A_c = area of concrete (mm^2)
A_{sc} = area of longitudinal reinforcement (mm^2)
f_y = characteristic strength of compression reinforcement (N/mm^2)

Equation 11.1 was developed for columns of conventional–strength concrete. But as shown in reference 2.80, such equations are also valid for columns of high–strength concrete.

Design load is given by the equation

$$\text{Design load} = \gamma_f\ F_k \quad \text{.....(11.2)}$$

where

F_k = characteristic load
γ_f = safety factor for loads

According to SABS 0100,

γ_f = 1,4 for dead load
γ_f = 1,6 for imposed load.

Consider columns spaced at 5–m centres in both directions. Assumed characteristic loads on one column are:

	Dead load (DL)	=	7 000 kN
	Imposed load (IL)	=	2 500 kN
∴	Design load (N)	=	DL x 1,4 + IL x 1,6
		=	9 800 + 4 000
		=	13 800 kN

Use high–yield steel with f_y = 410 MPa for longitudinal steel.

$$\text{Let } A_{sc} / A_c = p$$

$$\therefore A_{sc} = A_c\, p$$

Substituting in equation 11.1:

$$13{,}8 \times 10^6 = 0{,}4\, f_{cu}\, A_c + 0{,}67\, A_c\, p\, 410$$

$$\therefore \quad A_c = \frac{13{,}8}{0{,}4\, f_{cu} + 273{,}3\, p} \quad (m^2) \quad \text{.....(11.3)}$$

Equation 11.3 may be used to calculate A_c for various combinations of f_{cu} and p.

The following values were used for calculations:

f_{cu} = 30 MPa to 100 MPa in steps of 10 MPa

p = 0,01; 0,02; 0,03; 0,04; 0,05; 0,06.

(SABS 0100 permits a maximum of 0,06 for p.)

Steps in the calculations for each case are:

$$A_c;\ A_c + A_{sc};\ b = \sqrt{A_c + A_{sc}};\ A_{sc}$$

where b = breadth of column which is square in cross section.

11.2.2 Calculations of quantities and costs

Assuming that the height of a column is 2,7 m, quantities per column of concrete, longitudinal reinforcement, and formwork were calculated. The mass of steel in binders was calculated on the following basis:

p	0,01	0,02	0,03	0,04	0,05	0,06
Mass of reinforcement* in binders as % of mass of longitudinal reinforcement	10	9	8	7	6	5

* An approximation based on hypothetical reinforcement details.

Concrete volume was calculated net as far as longitudinal reinforcement was concerned, but the volume occupied by stirrups was ignored.

In order to determine mix proportions on which to base the material costs of concrete mixes, the 28–day cube–strength results shown in Figure 9.4 were analysed. A fair curve was drawn through the results and this curve was regarded as average strength versus C/W. With an assumed margin of 10 MPa, a curve for characteristic strength versus C/W was then constructed below the original curve. The curve for characteristic strength was used to determine C/W for each characteristic strength and values for water content and stone content interpolated from the table in Appendix A9.3. Cement content and sand content for each mix were then calculated. Mix proportions thus determined are shown in Appendix A11.1.

Rates for concrete were based on rates obtained for aggregates, cement and admixture, all delivered central Johannesburg, as follows:

Rapid–hardening portland cement	R0,22/kg
Stone	R0,037/kg
Sand	R0,027/kg
Admixture	R10,33/kg

Calculated material costs of concrete are shown in appendix A11.2

To derive total rates for concrete, the following procedure was followed.

The rate for 30–MPa concrete in place, as quoted by a quantity surveyor at the time the calculation was done, was R220/m^3.[11.2] If material costs per cubic metre are R154,45 (Appendix A11.2) the rate can be built up as follows:

Materials	R154,45	
Labour and plant for mixing, transporting, placing, etc	R42,00	
	R196,45	
With profit at 12 %	R220,02	say R220,00

The rate of each of the other mixes was then calculated and rounded off to the nearest ten cents as shown in Appendix A11.3.

Cost of steel, including cutting, bending and fixing was taken as R1,85/kg; cost of formwork was taken as R24,00/m^2.[11.2]

Calculations of A_c ; $A_c + A_{sc}$; f_{cu}; volume of concrete; mass of reinforcement; costs of concrete, formwork and reinforcement; and building cost per column are shown in Appendices A11.4 to A11.11.

11.2.3 Results

Results are shown graphically in Figure 11.1.

It is clear from the figure that:

- For a given steel:concrete ratio, cost decreases as strength of concrete increases.

- For a given concrete strength, cost increases as steel:concrete ratio increases.

- The economic benefits to be gained as concrete strength is increased in steps of 10 MPa decrease with increasing strength. For example the saving, at $p = 0,01$, by increasing f_{cu} from 90 MPa to 100 MPa is only 3 %.

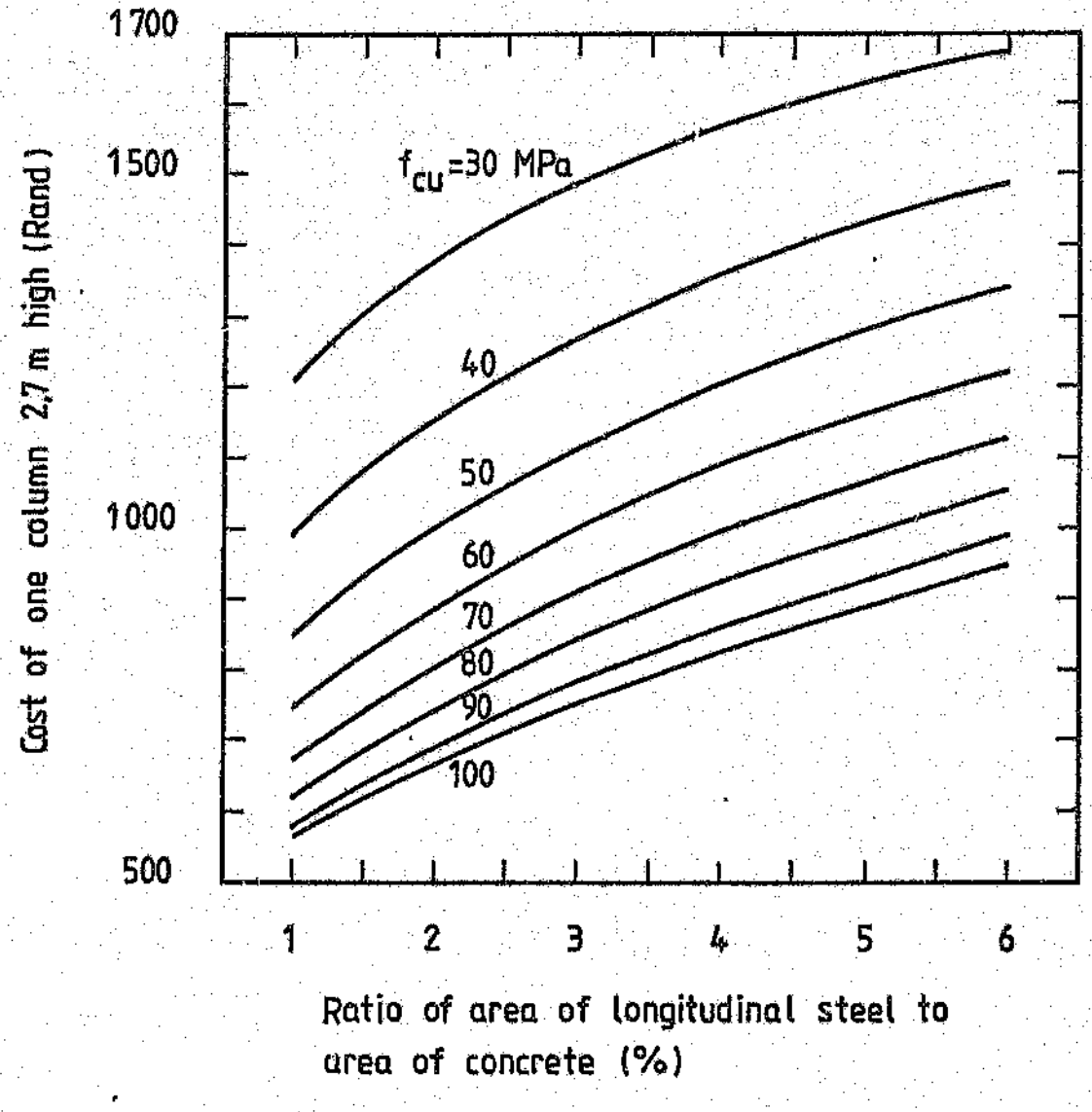


FIGURE 11.1: Building cost of one hypothetical column using local materials; based on February 1991 prices

- The effect on cost of increasing steel:concrete ratio is greater pro rata the higher the concrete strength. For example at f_{cu} = 30 MPa a column with p = 0,06 is 38 % more expensive than a column with p = 0,01; at f_{cu} = 100 MPa the difference is 68 %.

11.2.4 Column size and usable floor space

The effect of concrete strength on overall size of column, and therefore on usable floor space, is interesting. At the time of writing, building costs, in central Johannesburg, for high–quality buildings were R1 650 to R1 700 per square metre.[11.2] Building costs do not include professional fees, costs of land, or finance costs. They do include usable floor space and unusable areas occupied by walls and columns. Rather surprisingly, office rentals are calculated on a floor area which includes interior columns.[11.2] This provision forms part of a standard agreement between property owner and tenant as set out by the South African Property Owners Association. The tenant therefore pays the same rental, all other things being equal, for a given area irrespective of the fraction of the space taken up by interior columns. This then is the contractual aspect. In a free market, however, a tenant confronted with the choice between two spaces, equal in every respect except that they have very different ratios of net area to gross area would prefer the space with the more favourable use of floor space. The owner of such a space would likewise be able to command a rental higher than that for a space with a less favourable use of floor space. The curves in Figure 11.1 do not reflect the influence of column design on usable floor area. (For example, note that the R1 000 cost line intersects curves for 40–, 50–, 60–, 70– and 80–MPa concrete.) What is needed is a means of comparing columns in such a way that a small column will compare favourably with a large column even if building costs of the two columns are equal.

The concept of "Cost–area" is proposed as a means of quantifying the combination of building cost and the floor area taken up by the column. "Cost–area" is defined as the product of building cost and overall cross–sectional area of the column; its units are R x m^2.