HEAT EVOLUTION

MIX DESIGN, WORKABILITY, ~~ADIABATIC TEMPERATURE,~~ AND STRENGTH DEVELOPMENT OF HIGH STRENGTH CONCRETE.

by

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To

my parents Nicos and Thalia.

ABSTRACT.

A literature survey of the properties and uses of high strength concrete, defined for this study as having a strength in excess of 60 N/mm^2, has shown that of prime need is a systematic, reproducible procedure for attaining high strength concrete.

The "Maximum Density Theory", i.e. the requirement that the aggregate occupies as large a relative volume as possible, has been adopted as an approach to optimisation of the mix proportions. However, this does not consider the effect that the aggregate surface area has on the requirement of excess paste for lubrication. To investigate the combined effect of void content and surface area, mixes with lower sand proportions than that required for minimum void content were tested for slump. The optimum sand proportion is the one that produces the highest slump, for a particular cement content. This procedure has been called: "The Modified Maximum Density Theory".

Having thus optimised the cement and aggregate contents, partial cement replacement by mineral admixtures, at low water-cement ratios, has been investigated in order to assess:

a) their contribution to long term strengths,

b) their contribution to reducing the heat evolution of concrete mixes,

and c) their effect on the workability of concrete.

Condensed silica fume (at replacement levels of up to 15%) produced higher compressive strengths than ordinary Portland cement. Ground granulated blast furnace slag (at replacement levels of up to 30%) can be used without decreasing the 28-day strength. Replacement by 20% pulverised fuel ash resulted in a 15% decrease in the 28-day strength and equal strength to ordinary Portland cement concrete at ages beyond 56-days.

Temperature measurements during hydration, under adiabatic conditions, have however shown that these replacement levels do not lower the temperature rise at a water-binder ratio of 0.26. The higher levels required for significant temperature reduction will also cause a significant reduction in the strength. To offset this ground granulated blast furnace slag (58%) and pulverised fuel ash (36%) in combination with 10% condensed silica fume

were used. These combinations reduced the temperature rise by more than 10^0C while the reduction in the 28-day compressive strength was less than 15%.

Partial cement replacement by pulverised fuel ash and ground granulated blast furnace slag improved the workability and therefore allowed a reduction in the superplasticiser dosage required for a given slump. The use of condensed silica fume reduces the workability at low superplasticiser dosages, but it has a water-reducing effect above a certain superplasticiser dosage.

Results from these studies have been used to formulate guidelines for the proportioning of materials for producing high strength concrete.

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CHAPTER 1.
INTRODUCTION.

High strength concrete is often considered a relatively new material. However, its development has been gradual over many years, and as this development has continued, its definition has changed. In the 1950s, concrete with a characteristic compressive strength of 40 N/mm^2 was considered high strength in the USA. In the 1960s, concretes with 40 and 50 N/mm^2 characteristic strengths were used commercially. In the early 1970s, 60 N/mm^2 concrete was being produced[(1)]. However, in recent years, engineers, particularly in North America, have been faced with increasing demands for improved efficiency and reduced concrete construction costs from developers and governmental agencies. As a result, they are now designing larger structures using higher strength concrete at higher stress levels.

The higher strengths have been possible as a result of recent developments in material technology[(2)]. The properties of the cement have improved during the years, especially concerning the early strength development. This has led to higher quality requirements in the codes. For instance, the 3-day mortar strength of cement required in the British Standard Specification for Portland cements has increased from 15 N/mm^2 (1971) to 23 N/mm^2 (1978) and 25 N/mm^2 (1989)[(3)]. According to an FIP report[(4)], the introduction of condensed silica fume (CSF) and superplasticisers during the 1970s also made it relatively easy to produce ready mixed concrete with strength levels in the range of 100 N/mm^2.

1.1 DEFINITION OF HIGH STRENGTH CONCRETE.

High strength concrete refers to concrete which has a uniaxial compressive strength greater than that which is ordinarily obtained in a region, and therefore its definition varies on a geographical basis. For example, in regions where concrete with a compressive strength of 60 N/mm^2 is already being produced commercially, high strength concrete might be in the range of 80 to 100 N/mm^2 compressive strength. However, in regions where the upper limit on commercially available material is currently 35 N/mm^2 concrete, 60 N/mm^2 concrete is considered high strength concrete.

Further complications in defining high strength concrete arise from:

1. *the different sizes and shapes of specimens used for compression testing.* According to the FIP/CEB state-of-the-art report on high strength concrete[2], the characteristic compressive strength is measured on:
   (a) 100 mm cubes, 100 x 200 mm or 100 x 300 mm cylinders in Norway,
   (b) 160 x 320 mm cylinders in France,
   (c) 6 x 12 in. (152.4 x 305.8 mm) or 4 x 8 in. (101.6 x 203.2 mm) cylinders in USA, and,
   (d) 200 mm cubes, and more frequently 150 mm cubes, in Germany.

   For comparison between compressive cube strength and compressive cylinder strength (height/diameter = 2), a multiplication factor of 0.8 to the cube strength is often applied for normal strength concretes[5]. For high strength concrete, however, there exists no universal conversion factor. While the Norwegian Standard NS 3473: "Concrete structures, Design Rules"[6], recommends a reduction of 10 N/mm^2 to the cube strength, the CEB-FIP Model Code 1990[5] recommends a reduction of 15 N/mm^2. Also, Smeplass[7] has shown that the cylinder-cube ratio is not only a function of the strength, but also of the mix design parameters. For this reason the compressive cylinder strengths of structures referred to in this thesis have not been converted to compressive cube strengths. A more detailed description of studies concerning the conversion factor between different specimens is given in Appendix 1.

2. *the age of testing.* Whether specimens are tested at 28, 56, or 90-days, any of which may be more appropriate than the others for a particular job, can make a significant difference to the compressive strength.

In view of the above, it is not surprising that researchers and practising engineers have not yet agreed on a consistent definition of high strength concrete. High strength, normal weight concrete has been defined by the Chicago Committee on High-Rise Buildings (1977)[8], Shah (1979)[9] and ACI Committee 363 (1984)[1] as concrete having a compressive strength of at least 6,000 psi (41.4 N/mm^2) at 28-days. Shah[9] justified this definition with the following two arguments:

1. With the conventional methods of production and materials of construction, the bulk of concrete a ready-mix supplier delivers is in the range of 3,000 to 6,000 psi (20.7 to 41.4 N/mm^2). To produce concrete above 6,000 psi (41.4 N/mm^2), for normal weight aggregates, more stringent quality control, use of admixtures (water-reducers, pulverised fly ash, etc.), and careful selection of the blends of cement and the type and size of aggregates are essential. Thus, to distinguish concrete of above 6,000 psi (41.4 N/mm^2) compressive strength, it may be termed high strength.

2. The current design practice is based, among other things, on experiments made with concrete of compressive strength in the range of 3,000 to 6,000 psi (20.7 to 41.4 N/mm^2). Additional considerations, modifications of the empirical equations, and new tests may be necessary before a satisfactory procedure for the design of structures made with concrete of compressive strength significantly higher than 6,000 psi (41.4 N/mm^2) is developed. Thus, concrete of compressive strength higher than 6,000 psi (41.4 N/mm^2) can be considered in a separate class.

More recently, the FIP/CEB state-of-the-art report (1990)[2] has defined high strength concrete as concretes with a minimum compressive cylinder strength of about 60 N/mm^2, i.e. strengths higher than the present existing limits in national codes.

Shah[10] defined high strength for lightweight concrete as having a compressive strength of over 4,000 psi (27.6 N/mm^2), whereas Albinger[11] set the lower limit at 5,000 psi (34.5 N/mm^2).

The CEB/FIP state-of-the-art report[2] suggested that the practical upper limit for the compressive strength of concretes with ordinary aggregates is 130 N/mm^2, whereas, Saucier[12] stated that the eventual ceiling on concrete strength is virtually unlimited. He reported, however, that very high compressive strengths will only be achieved by changing production methods. In 1980, he stated that 5,000 to 10,000 psi (34.5 to 70.0 N/mm^2) concrete could be produced nearly anywhere in the United States by using conventional production techniques, by properly selecting materials and by maintaining good quality control. He claimed then that while it was possible to produce concrete with a compressive strength of up to 15,000 psi (103.4 N/mm^2) by utilizing more expensive materials and improved production techniques, concrete compressive strengths over 15,000 psi (103.4 N/mm^2) would require "exotic" procedures and materials.

In this research programme, high strength concrete has been defined as concrete with a minimum compressive cube strength of 60 N/mm^2 and made using only readily available concrete making materials and conventional production techniques.

1.2 DEVELOPMENT OF INTEREST IN HIGH STRENGTH CONCRETE.

It follows from the preceding discussion on the definition of high strength concrete that it is not new. Nasser[13] prepared a short bibliography of mainly U.S. reports in 1967 and publications from other countries[14,15,16] were appearing at around the same time. Although the potential of high strength concrete was recognised before 1970 it was not used to any significant extent. A strong interest developed in America when the economic advantages of using high strength concrete in the columns of high-rise buildings were recognised[17] and guidelines for its production were published. From 1975 onwards many reports have illustrated the advantages and uses of high strength concrete in America[1], Japan[18,19,20], Scandinavia[21,22], Russia[23] and the U.K.[24]. Most of the recent major publications have come mainly from America and include a report on high strength concrete in Chicago high-rise buildings[8], ACI Special Publications SP-87[25] and SP-121[26], ACI Committee 363 report[1], a book by Peterman and Carrasquillo[27] and an extensive bibliography[28].

High strength concrete has also been in focus in both research and practical utilization in

Norway for a number of years, in particular due to the potential cost-savings of its use in offshore concrete structures. This was the background for the initiative taken by the Norwegian Concrete Association in arranging the "Utilization of High Strength Concrete" symposium(29), which was held in Stavanger, Norway, June 1987.

While the characteristic strength record has risen to 14,000 psi (115 N/mm^2) in North America for the construction of Seattle's Two Union Square Building(30), Lacroix(31) stated that for about one century the mechanical strength of concrete in France has hardly varied. Current buildings utilize 25 to 30 N/mm^2 concrete strengths, and only exceptional structures utilize more than 35 N/mm^2 concrete strengths. This is despite the fact that concrete with characteristic strengths of 50 to 60 N/mm^2 can be achieved on site over all French territory without excessive material hauling distances.

Although the main interest for high strength concrete has been abroad, the British Cement Association, in 1988, stepped up its efforts to increase its popularity in this country. The initial stage was a comprehensive state-of-the-art report, and an accompanying bibliography, both by Parrott(32,33). This collated and reviewed 131 recent publications. The second stage involved the casting and testing of columns at a ready mix concrete plant(34). The third stage investigated the practicality of high strength concrete on site in central London, and seemed to show that production and utilization of high strength concrete under conventional procedures is feasible(35).

1.3 METHODS OF PRODUCING HIGH STRENGTH CONCRETE.

Several exotic methods for producing high strength concrete have been studied, such as (1) modification with polymers(36), (2) fibre reinforcement(37,38), (3) slurry mixing (preblending water and cement at high speed for efficient hydration)(39), (4) sulphur-infiltrated concrete(40), (5) compaction by pressure(14,41), (6) compaction by pressure combined with vibration(42), (7) autoclave curing(43), and (8) mix proportioning using active or artificial aggregates(14,15). One study(39) advocated revibration 2-1/2 hours after initial vibration as a means for achieving higher strengths. Structural design which accounts for additional concrete strength resulting from triaxial compression or concrete confinement is also possible[14].

However, cost-effective production of high strength concrete in construction today is achieved by carefully selecting, controlling, and combining cement, chemical and mineral admixtures, aggregates, and water, and using conventional mixing and placing techniques, but with high quality control[27]. Freedman[44] stated that in order to achieve higher strength concretes the concrete producer must optimise the cement characteristics, aggregate quality, paste proportioning, aggregate-paste interaction, mixing, consolidation, and curing procedures.

Production of high strength concrete, therefore, requires materials of the highest quality and their optimum proportioning[45]. Mix proportions have often been selected empirically by extensive laboratory testing since there are no accepted procedures[46] such as the Building Research Establishment and ACI methods of proportioning normal concrete mixtures[47,48]. The requirement of a low water-cement ratio has led to the use of high cement contents (up to 550 kg/m^3). Mineral admixtures like pulverised fuel ash (PFA) and ground granulated blast furnace slag (GGBFS), despite not producing exceptionally high 28-day strengths, are often cheaper than ordinary Portland cement (OPC) and this factor in combination with possible improvement in workability and long-term strength, and moderation of the heat evolution of the cement-rich mixes tend to encourage their use[32].

The use of high cement contents and superplasticisers can lead to extremely cohesive or sticky fresh mixes[1], and many researchers[2,16,49,50] have advocated that the slump test therefore gives the wrong impression of the workability. Also, the placement of high strength concrete becomes a critical matter in view of the necessity to minimise crack-like voids against coarse aggregates and reinforcements[51]. In order to be able to understand how fresh high strength concrete will behave differently to traditional concrete and to be able to optimise the mix design according to the requirements given by the methods of placement and compaction, there is a need for a fundamental knowledge of how to measure the workability, for example by expressing it in terms of classical rheological models such as those of Bingham or Newton[2].

1.4 RESEARCH PROGRAMME.

In the thesis, a brief review of the structural applications, advantages and disadvantages of high strength concrete is first presented. The main literature review then deals with the selection and proportioning of materials for the production of high strength concrete. Various methods that have at one time been suggested for proportioning concrete ingredients are also described.

The experimental programme is then described, the main divisions of which are:

1. Attaining a systematic procedure for optimising the mix proportions of concrete with strength in excess of 60 N/mm^2. Only ordinary concrete making materials and conventional production techniques were used.

2. Assessing the rheological properties of high strength concrete. This included:
   (a) reviewing the applicability of the tests included in the British Standards in measuring the "workability" of concrete.
   (b) Determining the effect of partial cement replacement by PFA, GGBFS, and condensed silica fume (CSF), on the workability and workability loss, as defined by the slump test.
   (c) Assessing the efficiency of a superplasticiser in reducing the water-binder ratio while retaining the slump at 150 mm.
   (d) Attempting to express the workability of high strength concrete according to the Bingham model, with tests carried out with Tattersall's two point test.

3. Assessing the effectiveness of mineral admixtures, i.e. PFA, GGBFS and CSF, in reducing the adiabatic temperature rise.

4. Measuring the effects of water-cement ratio, aggregate type and size, sand grading, and partial cement replacement by PFA, GGBFS and CSF on the strength

development of high strength concrete.

The thesis ends with conclusions and recommendations for further research.

CHAPTER 2.
STRUCTURAL APPLICATIONS, ADVANTAGES AND DISADVANTAGES OF HIGH STRENGTH CONCRETE.

2.1 INTRODUCTION.

There are definite advantages, both technical and economical, in using high strength concrete in structures today. However, many proponents of high strength concrete also indicate that it has certain disadvantages. Few of these are regarded as insurmountable but they can dilute or, in some cases, negate the advantages of high strength concrete[32].

Instances of applications showing the technical and economical advantages of high strength concrete are outlined below. Some of the structures have concrete strength less than 60 N/mm^2, but these are included to show the development of the concrete strength in structures through the years. How some of the disadvantages may be overcome is discussed later in the chapter.

2.2 STRUCTURAL APPLICATIONS OF HIGH STRENGTH CONCRETE.

The main structural applications of high strength concrete have been in:

a) high-rise buildings,

b) bridges,

and, c) offshore platforms.

There have been some applications of high strength concrete in other areas, such as piles, highway pavements, and the surfacing of the stilling basin of a dam.

2.2.1 High-rise buildings.

The economic advantages of high strength concrete are most readily realized when the concrete is used in the columns of high-rise buildings as has been the case in North America for many years[1]. A listing of the more significant buildings and their locations is given in Table 2.1.

The advantages of using high strength concrete in these buildings include:

1. *The ability to reduce the amount of costly reinforcing steel without sacrificing strength.* The strength increase from 6,000 psi (41.4 N/mm^2) to 8,000 psi (55.2 N/mm^2) of the concrete used for the construction of the **Helmsley Palace Hotel**[1] resulted in a 10 per cent reduction of the reinforcing steel used in the columns of the lower five levels. Further economies were realized by minimising changes in column sizes and reducing column reinforcement on the upper floors. In the **225 W. Wacker Drive** building[52], the first five storeys and the basement columns contained 14,000 psi (96.5 N/mm^2) concrete. Varying the strengths from 14,000 to 4,500 psi (96.5 to 31.0 N/mm^2) at the appropriate floor levels to obtain 1 per cent reinforcing in the columns produced the most economical means of satisfying the structural requirements.

2. *The number of storeys can be increased while maintaining an acceptable column size at the lower floors.* The lower floors of the **Helmsley Palace Hotel**[1] were designated for ballroom and restaurant functions and, therefore, required large column spacing. The common limitations of 6,000 psi (41.4 N/mm^2) concrete would have made the columns prohibitively large and uneconomical. The owners of the **Texas Commerce Tower**[53] indicated that they would have a maximum exterior column size of 4 x 4 ft (1.22 x 1.22 m) at the third floor from a leasing standpoint. At the time of its design, the maximum concrete strength utilized in building construction in Houston was 6,000 psi (41.4 N/mm^2), due to the problems of obtaining higher strengths with the river gravel, common in Houston. A 75-storey structure utilizing this concrete would have resulted in extremely large

BUILDING	LOCATION	YEAR[1]	No. OF STOREYS	MAXIMUM DESIGN STRENGTH		REFERENCES.
				psi	N/mm^2	
Lake Point Tower	Chicago	1965	70	7,500	52.7	54,55
One Shell Plaza	Houston	1970	52	6,000	41.4[2]	54,55
Midcontinental Plaza	Chicago	1972	50	9,000	62.1	8
Frontiers Towers	Chicago	1973	55	9,000	62.1	8
Royal Bank Plaza	Toronto	1975	43	8,800	60.7	2
Water Tower Place	Chicago	1975	79	9,000	62.1	8,54,55
River Plaza	Chicago	1976	56	9,000	62.1[3]	2,8
Richmond - Adelaide Centre	Toronto	1978	33	8,800	60.7	2
Helmsley Palace Hotel	New York	1978	53	8,000	55.2	1
One Union Square Building	Seattle	1979	38	7,500	52.7	57
Larimer Place Condominiums	Denver	1980	31	8,000	55.2	2
City Center Project	Minneapolis	1981	52	8,000	55.2	2
Texas Commerce Tower	Houston	1981	75	7,500	52.7	53,56,58
S.E. Financial Center	Miami	1982	53	7,000	48.3	2
Petrocanada Building	Calgary	1982	34	7,250	50.0	2
Skyline Tower Building	Bellevue-Seattle	1982	26	7,000	49.2	57
Chicago Mercantile Exchange	Chicago	1982	40	9,000	62.1[4]	2
Pacific Park Plaza	Emeryville CA	1983	30	6,500	44.8	2
Columbia Center	Seattle	1983	76	9,500	66.0	57
First Republic Bank Plaza	Dallas	1983	72	10,000	69.0	59,60
World Trade Center	Tacoma-Seattle	1984		8,000	56.2	57
Century Square Building	Seattle	1984	29	10,000	70.3	57
900 N. Mich. Annex	Chicago	1986	15	14,000	96.5	2
Scotia Plaza	Toronto	1987	68		70	61
Grande Arche de la Defense	Paris	1988			65	2,62
311 South Wacker Tower	Chicago	1989	79	12,000	82.7	63
Two Union Square Building	Seattle	1989	58	14,000	96.5[5]	30
Pacific First Center	Seattle	1989	44	14,000	96.5[5]	2,64
Gateway Tower	Seattle	1989	62		94	2
One Peachtree Center	Atlanta	1990	60	12,000	82.7	65
225 W. Wacker Drive	Chicago	1990	31	14,000	96.5[6]	52

1. Year in which high strength concrete was cast.
2. Lightweight concrete.
3. Two experimental columns with 11,000 psi (75.8 N/mm^2) concrete were included.
4. Two experimental columns with 14,000 psi (96.5 N/mm^2) concrete were included.
5. The design requirement for an extremely high modulus of elasticity of 7.2×10^6 psi (49,644 N/mm^2), indirectly required a strength of 19,000 psi (131 N/mm^2).
6. One experimental column with 17,000 psi (117.2 N/mm^2) concrete was included.

Table 2.1: Buildings with high strength concrete.

columns. Hence, research was conducted on using high strength concrete to reduce column size. Investigation indicated that if the aggregate was changed from river gravel to an imported limestone aggregate, 7,500 psi (51.7 N/mm^2) could be produced.

3. *The reduction of the dimensions of columns and beams results in a decrease of the dead weight of the structure, which can substantially lessen the design requirements for the building's foundation.* The **One Shell Plaza** building[54], was originally conceived as a 35-storey building because of foundation difficulties. However, a 50-storey building was desirable for a viable investment. A preliminary analysis showed that if high strength lightweight concrete could be used, a 52-storey building could be built for the original estimated unit cost of a 35-storey building. The designer called for concrete with a density of 115 lb/ft^3 (1,840 kg/m^3) and 28-day compressive strength of 6,000 psi (41.4 N/mm^2). Extensive evaluation of the materials in the Houston area was undertaken before the building was constructed. This included tests to evaluate the creep and shrinkage properties. Architectural design limitations for the **311 South Wacker Drive Tower**[63] precluded a composite steel frame because high lateral forces on a tall structure would have created excessive foundation costs. The solution was an all reinforced building with shear wall frame interaction using concrete with design strengths up to 12,000 psi (82.7 N/mm^2).

4. *The additional stiffness provided by high strength concrete with its high modulus of elasticity reduces the amount of sway in the upper storeys of buildings.* The **Columbia Center**[57] is a steel frame building except for three large steel/concrete composite columns which carry most of the self weight, along with the wind and seismic forces. The main concern was to maximise the modulus of elasticity of the concrete in order to reduce the amount of sway in the upper storeys of the building. Concrete with a compressive strength of 9,500 psi (65.5 N/mm^2) was used. High strength concrete was specified for the **First Republic Bank Plaza**[59] to provide a high stiffness for the full height of the structure, which had a high height-to-width ratio of 7.24. The rigidity of steel was calculated to be five to

seven times that of concrete per unit volume, but the cost was 20 to 30 times the cost per unit volume of cast-in-place reinforced concrete. The high strength concrete with a high modulus of elasticity provided six times as much stiffness per dollar in a column as steel. The design strength specified for the **Two Union Square Building**[30] was "only" 14,000 psi (96.5 N/mm^2), but the design requirement for an extremely high modulus of elasticity of 7.2 x 10^6 psi (49,644 N/mm^2), indirectly required a strength of 19,000 psi (131.0 N/mm^2). This is about twice the modulus of a conventional, hardened concrete, and was required to meet the occupant-comfort criterion for the completed building.

5. *The relatively high early concrete strengths allow early stripping of the formwork, thus accelerating the construction.* A custom-made, self-climbing, three-storey steel formwork assembly, was used for the construction of the **Texas Commerce Tower**[53]. High strength concrete allowed early stripping of the formwork allowing the construction to proceed at the rate of two floors per week which at least equals that for structural steel construction. The **311 South Wacker Tower**[63] used ultra high strengths combined with a post-tensioned floor design to speed its construction.

6. *Mouldable to architectural advantage over steel.* The architectural features of the **Texas Commerce Tower** building[53] dictated a granite facade which tends to favour the composite, concrete/steel, design rather than an all-steel structure. Major architectural changes in the vertical elements of the **311 South Wacker Tower**[63] would have been required if a steel, instead of a reinforced concrete, frame was to be used.

Although the advantages in using concretes with characteristic compressive strengths of as high as 14,000 psi (96.5 N/mm^2) are today well recognised, the development of the strength has progressed gradually, as can be seen from Table 2.1. The development of mix designs for concretes with greater compressive strengths than those which have ordinarily been obtained in a region has been accompanied by a lot of independent regional research. For example, in:

(i) *Chicago area.*

According to the Chicago Committee on High-Rise Buildings[8], the highest concrete strength available in the Chicago area in 1962 was 5,000 psi (34.5 N/mm^2). The increase from 5,000 to 6,000 psi (34.5 to 41.4 N/mm^2) concrete for the lower columns of 1000 Lake Shore Plaza did not require much research. The amount of cement was increased, and pulverised Fuel ash and a water-reducer were added to give the increased strength. This new concrete was delivered to another high-rise project in the Chicago area, The Outer Drive East Condominiums.

The next step, 7,500 psi (51.7 N/mm^2) concrete for the Lake Point Tower project (1965) required a carefully planned research and development programme which optimised all the variables in materials affecting this product. Also, new quality control procedures had to be established for the production, delivery, handling and placing of the concrete. Other projects such as 1130 South Michigan Avenue, also received the benefit of the 7,500 psi (51.7 N/mm^2) concrete.

The next step in the increasing demand for higher strengths was 9,000 psi (62.1 N/mm^2) concrete for the Midcontinental Plaza Building (1972). By this time, 7,500 psi (51.7 N/mm^2) concrete had been accepted by the industry and was being used in columns and shear walls when logical for architectural, engineering or economic considerations. Other projects built in the next few years after 1972 such as 200 W. Monroe, Frontier Towers, Water Tower Place, and Marriot Chicago, received the benefit of the 9,000 psi (62.1 N/mm^2) concrete.

During the spring of 1976, 11,000 psi (75.8 N/mm^2) concrete was delivered by Material Service Corporation to two instrumented columns of the River Plaza Project as a full size test, to investigate the properties of this product during its early life.

More recently, in 1989, 17,000 psi (117.2 N/mm^2) concrete was delivered on four different occasions to the Construction Technology Laboratories for investigation of shear strength, bond, and structural properties testing[52]. An experimental column with 17,000 psi (117.2 N/mm^2) concrete was also cast in the 225 W. Wacker Drive. Moreno[52] stated that:

> "As of 1989, this strength is not yet commercially available; 26 years of research and development have culminated in technology that can produce and deliver these high strength concretes in ready-mixed concrete trucks."

(ii) *Texas.*

While 11,000 psi (75.8 N/mm^2) concrete had been experimentally used in Chicago in 1976, the first use of 10,000 psi (69.0 N/mm^2) concrete in Texas was in 1983, for the construction of the First Republic Bank Plaza[59]. Prior to its construction, the question was asked if concrete could be produced in Dallas that would meet the specified strength of 10,000 psi (69.0 N/mm^2). The response was that a definite "yes" would require an extensive laboratory investigation. It was fortunate that there was approximately a two-year lead time. During the 18 following months a total of 100 laboratory trial batches were made, 70 m^3 of plant-batched concrete was produced for field testing and approximately 4,000 specimens were made.

Following this, Peterman and Carrasquillo[27] realised that if engineers in Texas were to take advantage of high strength concrete, as had been done in Chicago, they had to be given reason to be confident that high strength concrete could be produced and used safely, economically, and efficiently. According to them, the much needed first step was to establish, in a form useful for practising engineers in Texas, guidelines for the selection of materials and their proportions for producing 6,000 to 9,000 psi (41.4 to 62.1 N/mm^2) concrete. To achieve this 2,500 concrete specimens, from over 200 different batches of concrete, were tested. They advocated, because of the variability in physical properties and availability of concrete making materials, a similar testing programme for each separate region intending to use high strength concrete.

Table 2.2 shows how the availability of concrete making materials in various regions has

	Water Tower Place[38], (Chicago)		River Plaza[38], (Chicago)		Texas Commerce Tower[38], (Houston)		First Republic Bank Plaza[39], (Dallas)			
							MIX 1.		MIX 2.	
	lb/yd^3	kg/m^3	lb/yd^3	kg/m^3	lb/yd^3	kg/m^3	lb/yd^3	kg/m^3	lb/yd^3	kg/m^3
CEMENT	846	501.7	850	504.1	640	379.5	606	359.4	602	357.0
PFA	100 Class F	59.3 Class F	100 Class F	59.3 Class F	160 Class C	94.9 Class C	253 Class C	150.0 Class C	251 Class C	148.8 Class C
GGBFS										
CSF										
TOTAL BINDER	946	561.0	950	563.4	800	474.4	859	509.4	853	505.8
COARSE AGGREGATE Max. size: Type:	1800 5/8 in. Stone	1067.4 15.9 mm Stone	1730 1/2 in. Stone	1025.9 12.7 mm Stone	1925 1/2 in. Limestone	1141.5 12.7 mm Limestone	1772 3/8 in. Limestone	1050.8 9.5 mm Limestone	1994 1 in. Limestone	1182.4 25.4 mm Limestone
SAND	1025	607.8	1040	616.7	980	581.1	1151	682.5	1017	603.1
WATER	300	177.9	330	195.7	300	177.9	279	165.4	250	148.3
ADMIXTURE	25.4 fl. oz./yd^3	552 mL/m^3	43.0 fl. oz./yd^3	934 mL/m^3	24 fl. oz./yd^3	521 mL/m^3	34.3/85.9[1] fl. oz./yd^3	745/1866[1] mL/m^3	34.2/85.3[1] fl. oz./yd^3	743/1853[1] mL/m^3
WATER-BINDER RATIO	0.32[2]	0.32[2]	0.35[2]	0.35[2]	0.38[2]	0.38[2]	0.33	0.33	0.29	0.29
SLUMP	4.5 in.	115 mm	4.5 in.	115 mm	4.0 in.	100 mm	10 in.	255 mm	9-1/4 in.	235 mm
Air content (%)			1.5	1.5			1.3		0.8	

	Scotia Plaza[41], (Toronto).						Grande Arche de la defense[2], (Paris).		Pacific First Center[2,44], (Seattle).	
	INITIAL	MIX 1.	MIX 2.	MIX 3.	MIX 4.	MIX 5.	MIX 1.	MIX 2.		
	kg/m^3	kg/m^3	kg/m^3	kg/m^3	kg/m^3	kg/m^3	kg/m^3	kg/m^3	lb/yd^3	kg/m^3
CEMENT	315	422	384	338	293	248	425	425	900	533.7
PFA									100 Type F	59.3 Type F
GGBFS	135		47	94	142	189				
CSF	36	36	36	37	37	37		30	68	40.3
TOTAL BINDER	486	458	467	469	472	474	425	455	1068	633.3
COARSE AGGREGATE Max. size: Type:	1130 10 mm Limestone	1044 10 mm Limestone	1094 10 mm Limestone	1099 10 mm Limestone	1105 10 mm Limestone	1112 10 mm Limestone	1044 20 mm Gravel	1033 20 mm Gravel	1802 3/8 in. Gravel	1068.6 9.5 mm Gravel
SAND	745	688	701	704	708	713	730	705	1050	622.7
WATER	145	175	185	182	174	169	170-5	170-190	220	130.5
ADMIXTURE	6000/835[3] mL/m^3	?	?	?	?	?	17,000 mL/m^3	18,000 mL/m^3	250/60 fl. oz./m^3	4210/1010 mL/m^3
WATER-BINDER RATIO	0.38	0.38	0.40	0.39	0.37	0.36	0.40-0.41	0.37-0.42	0.22	0.22
SLUMP	175 mm	140 mm	185 mm	195 mm	220 mm	280 mm	225 mm	235 mm		250 mm
Air content		1.8	1.4	1.2	0.9	0.5				

1 Type A/Type F = Water-reducer/Supeplasticiser.
2 Water-binder ratios have not been given, and they were therefore calculated from the total binder and water contents. No allowance was made for absorption of water by the aggregate.
3 Superplasticiser/Retarder.
4 Retarder dosages varied from 0 to 3 litres.

Table 2.2: Details of high strength concrete mixes used in buildings.
Part A: Mix designs.

	Water Tower Place.				River Plaza.					
	MOIST 6 X 12 in. cylinders		AIR 6 X 12 in. cylinders		YARD. 6 X 12 in. cylinders.		JOBSITE. 6 X 12 in. cylinders.		CORES. 4 X 8 in. cylinders.	
	psi	N/mm²	psi	N/mm²	psi	N/mm²	psi	N/mm²	psi	N/mm²
12 hours strength										
24 hours strength										
1-day strength										
2-day strength										
3-day strength										
7-day strength	7,640	52.7			7,790	53.7	7,310	50.4	6,750	46.5
28-day strength	9,400	64.8	9,150	63.1	10,340	71.3	9,410	64.9	8,100	55.9
56-day strength	10,580	73.0			11,240	77.5	10,500	72.4	10,690	73.7
91-day strength			9,420	65.0	11,630	80.2	11,410	78.7	10,460	72.1
180-day strength			9,210	63.5	13,220	91.2				
365-day strength			9,700	66.9						

	Texas Commerce Tower.				First Republic Bank Plaza.			
	Lab 6 X 12 in. cylinders		Field 6 X 12 in. cylinders		MIX 1. 6 X 12 in. cylinders.		MIX 2. 6 X 12 in. cylinders.	
	psi	N/mm²	psi	N/mm²	psi	N/mm²	psi	N/mm²
12 hours strength								
24 hours strength								
1-day strength					1,900	13.1	1,560	10.8
2-day strength								
3-day strength					7,099	48.9	6,669	46.0
7-day strength	7,400	51.0	6,465	44.6	9,228	63.6	9,277	64.0
28-day strength	9,700	66.9	8,146	56.2	11,522	79.4	11,236	77.5
56-day strength	10,475	72.2	9,005	62.1	12,376	85.3	12,448	85.8
91-day strength					12,901	89.0	13,401	92.4
180-day strength								
365-day strength								

	Scotia Plaza.					Grande Arche de la Defense		Pacific First Center	
	MIX 1.	MIX 2.	MIX 3.	MIX 4.	MIX 5.	MIX 1.	MIX 2.		
	150 x 300 mm cylinders.					160 x 320 mm cylinders.		4 x 8 in. cylinders.	
	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	psi	N/mm²
12 hours strength	32.3	25.2	19.9	14.5	6.7				
24 hours strength	35.1	31.2	24.7	20.7	13.5				
1-day strength	40.8	40.6	34.0	28.3	19.5				
2-day strength	46.0	48.9	44.7	40.8	32.4				
3-day strength									
7-day strength	61.9	71.6	70.0	68.8	62.5				
28-day strength	74.5	88.0	87.8	87.3	82.9	59.0	65.4		115
56-day strength	78.8	91.4	92.5	91.6	84.5			17,980	123.4
91-day strength	80.6	93.8	93.7	91.6	86.2	67.0	72.7		
180-day strength									
365-day strength									135

Table 2.2: Details of high strength concrete mixes used in buildings.
Part B: Compressive strengths.

influenced the mix designs. Mineral admixtures that have been used are Class F (low calcium) PFA in Chicago, Class C (high calcium) PFA in Houston and Dallas, GGBFS in combination with CSF in Toronto, Class F (low calcium) PFA in combination with CSF in Seattle, and CSF in Paris. Coarse aggregates that have been used are crushed limestone and gravel, with maximum sizes from 3/8 in. (9.5 mm) to 1 in. (25.4 mm).

2.2.2 Bridges.

Without doubt, the largest use of high strength concrete in North America, has been in columns of buildings. However, despite the limited published information of bridge structures utilizing high strength concrete, long span bridges are another area where the qualities of high strength concrete are proving themselves economically attractive. Several examples of bridges in North America, Europe, and in Japan are listed in Table 2.3(2).

For long spans, the weight of the structure can be a signifigant portion of the total weight carried by the bridge. According to Carpenter(66), high strength concrete's comparatively greater compressive strength per unit weight and unit volume allows increases in span capability, reduction of the girder depth, lighter and more slender bridge piers, and, therefore, reduces the load that has to be carried by the foundations. In addition, the increased stiffness of high strength concrete is advantageous when deflections or stability govern the bridge design. The increased tensile strength of high strength concrete is helpful in service load design in prestressed concrete. Anderson(67) had suggested that the low creep of high strength concrete should be taken into account when considering prestress losses. Since most of the prestress loss is attributable to creep and shrinkage, the losses for high strength concrete members should be less than for lower-strength concrete members.

2.2.3 Offshore concrete platforms.

Concrete has been used successfully in marine structures since the early part of this

Bridge	Location	Year	Maximum span m	Maximum design concrete strength N/mm²
Willows Bridge	Toronto	1967	48	41
Houston Ship Chanal	Texas	1981	229	41
San Diego to Coronado	California	1969	43	41 L*
Linn Cove Viaduct N	Carolina	1979	54	41
Pasco-Kennewick intercity	Washington	1978	299	41
Coweman River Bridges	Washington		45	48
Huntington to Proctorville	W. Va. to Ohio	1984	274	55
Nitta Highway Bridge	Japan	1968	30	59
Kaminoshima Highway Bridge	Japan	1970	86	59
Tower Road Bridge	Washington	1981	49	62
Fukamitsu Highway Bridge	Japan	1974	26	69
Ootanabe Railway Bridge	Japan	1973	24	79
Akkagawa Railway Bridge	Japan	1976	46	79
Kylesku Bridge	Scotland		79	53
Deutzer Bridge	W.Germany	1978	185	69*
Parrot Ferry Bridge	California	1979	195	43*
Pont de Tricastin	France		142,4	30*
Ottmarsheim	France	1979	172	30*
Selbjørn Bridge	Norway	1977	212	40
Pont du Pertuiset	France	1988	110 m	65
Pont de Joigny	France	1988		60
Arc sur la Rance	France	1989		60
Giske	Norway	1989	52	55
Sandhornøya	Norway	1989	154	55*
Boknasundet	Norway	1990	190	60*
Helgelandsbrua	Norway	1990	425	65

* Lightweight concrete

Table 2.3: Bridges with high strength concrete[2].

century. It was first introduced on a large scale to the oil industry in 1971 when Phillips chose concrete for the Ekofisk oil production platform in the North Sea. It was located in water 230 ft (70 m) deep and contained 104,640 ft^3 (80,000 m^3) of normal weight concrete with a specified 28-day compressive strength of 6,000 psi (45 N/mm^2)[68]. In 1971, characteristic compressive strengths of 45 to 50 N/mm^2 were considered a near upper limit for site-produced concrete. Since then a total of 21 major structures containing over 2.6 million yd^3 (2 million m^3) of concrete have been ordered for the North Sea oil fields, (Table 2.4)[69], and specified strengths have risen to 10,000 psi (69.0 N/mm^2)[70].

These very large structures are designed and built according to rigorous specifications and challenge the frontiers of developments in many technological fields including design processes, construction methods and materials technology. The critical demands due to high strength, durability, and constructibility requirements, the magnitude of the structures, and the rate of construction have enforced the development of special cements and the use of pozzolans, hydraulically processed sand, and refined superplasticisers and other chemical admixtures to achieve satisfactory concrete mix designs[70]. Some of the mix designs that have been used are shown in Table 2.5[70]. Figure 2.1[70] shows the specified and obtained and characteristic cube strength for the cell wall slipforming operations carried out by the Norwegian Contractors from 1972 to 1986. The figure demonstrates the consistent uniformity of production as well as the increase, particularly in recent years, in specified and obtained strength. The Norwegian concrete industry is preparing for the use of even higher concrete strengths in both onshore and offshore work in the near future.

In parallel to further development and optimization of the "traditional" Condeep platform built so far, the next generation of platforms has been under development for quite some years. Concepts have been developed for concrete in floating and sub-sea installations. But emphasis has up to now been put on the development of very large structures capable of operating in water depths in excess of 300 m and on very soft soil conditions. One significant step in this context is the Gullfaks C platform, (Figure 2.2)[68], that has been constructed for 216 m of water and for soft normally consolidated clay. The 22 m deep concrete skirts or skirt piles have been designed to carry the loads down to firmer and more competent soils. Figure 2.3[68] shows two deepwater platform concepts that have

PLATFORM	DESIGNER/ CONTRACTOR	CONSTRUCTION SITE	DELIVERED	CONCRETE VOLUME m³	WATER DEPTH
Ekofisk 1	Doris/Selmer-Høyer	Stavanger	1973	80.000	70 m
Beryl A	Norwegian Contractors	Stavanger	1975	52.000	118 m
Brent B	Norwegian Contractors	Stavanger	1975	64.000	140 m
Frigg CDP-1	Howard Doris/ Norwegian Contractors	Åndalsnes	1975	60.000	104 m
Brent D	Norwegian Contractors	Stavanger	1976	68.000	140 m
Frigg MP-2	Howard Doris/Skånska Doris	Strømstad	1976	60.000	94 m
Frigg TP-1	Sea Tank/McAlpine	Ardyne Point	1976	49.000	104 m
Statfjord A	Norwegian Contractors	Stavanger	1977	87.000	145 m
Frigg TCP-2	Norwegian Contractors	Åndalsnes	1977	50.000	104 m
Dunlin A	Andoc	Rotterdam	1977	90.000	153 m
Brent C	Sea Tank/McAlpine	Ardyne Point	1978	105.000	141 m
Cormorant A	Sea Tank/McAlpine	Ardyne Point	1978	120.000	149 m
Ninian Central	Howard Doris	Loch Kishorn	1978	140.000	136 m
Statfjord B	Norwegian Contractors	Stavanger	1981	140.000	145 m
Statfjord C	Norwegian Contractors	Stavanger	1984	130.000	145 m
Gullfaks A	Norwegian Contractors	Stavanger	1986	125.000	135 m
Gullfaks B	Norwegian Contractors	Stavanger	1987	100.000	141 m
Oseberg A	Norwegian Contractors	Stavanger	1988	120.000	109 m
Gullfaks C	Norwegian Contractors	Stavanger/Vats	1989	240.000	216 m
* Ekofisk PB	Doris/Peconor Ekofisk J/V	Rotterdam/ Ålfjorden	1989	106.000	75 m
* Sleipner A	Norwegian Contractors	Stavanger	1992	77.000	82 m
* Under construction		TOTAL VOLUME		2.063.000	

Table 2.4: Concrete platforms in the North Sea[69].

Condeep Beryl A (1973 - 75)	
Cement (SP 30)	430 kg
Sand, Årdal 0 - 10 mm	900kg
Coarse aggregate, Årdal 10 - 32 mm	900kg
Water	175 ltr.
Betokem LP	4 ltr.
Slump	120 mm
w/c ratio	0.41
Obtained mean 28 day compressive strength	55 N / mm²

Condeep Statfjord C (1981)	
Cement (SP 30 - 4A)	380kg
Sand, Tytlandsvik 0 - 5 mm	1030 kg
Coarse aggregate, Tytlandsvik (5- 20)	845 kg
Water	160 ltr.
Betokem PA (B)	4 ltr.
Slump	220 mm
w/c ratio	0,42
Obtained mean 28 day compressive strength	67 N/ mm²

Condeep SP Gullfaks C (1986)	
Cement (SP30 - 4A Mod.)	430 kg
Silica	20kg
Sand, Tytlandsvik (5- 20 mm)	860 kg
Water	165 ltr.
Betokem PA (B)	6 ltr.
Slump	240 mm
w/c ratio	0,38
Obtained mean 28 day compressive strength	83 N/mm²

Table 2.5: Concrete mix designs for oil platforms; milestone developments[70].

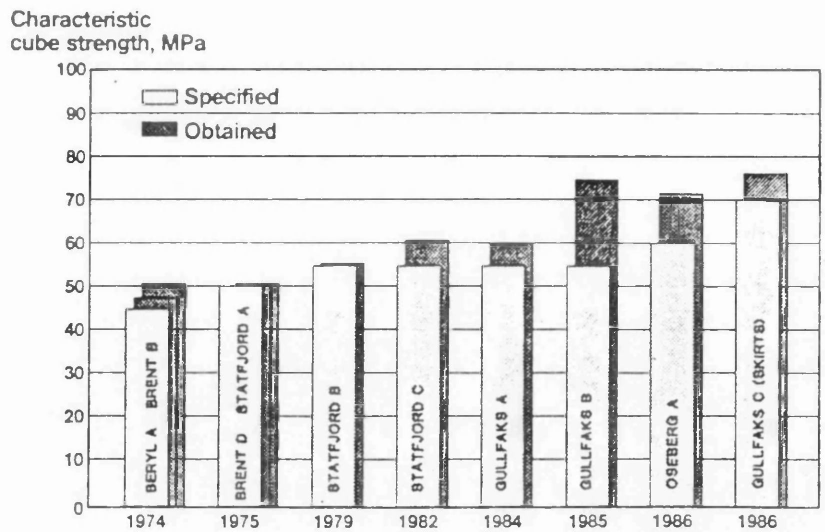


Figure 2.1: Concrete quality for the cell wall slipforming operations(70).

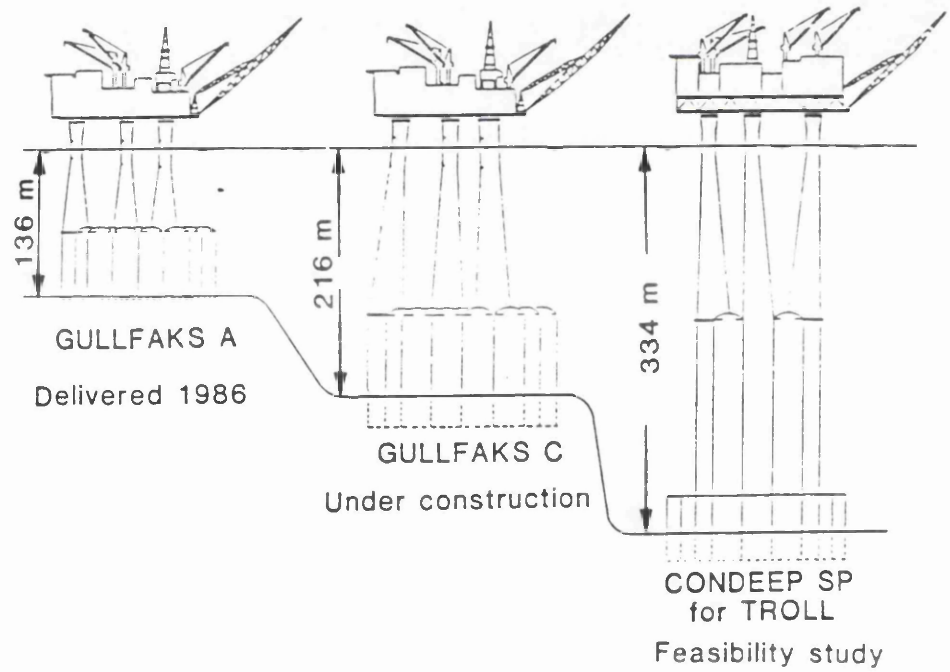


Figure 2.2: Development of offshore concrete platforms(68).

been considered for several fields in the deepwater areas of the Norwegian continental shelf, such as the Troll field in around 340 m of water.

A high topside payload capacity during tow (e.g., 43,000 tons for Gullfaks A) and the possibility of oil storage within the platform have been considered as the primary merits of a concrete platform as compared to the traditional competitor, the steel jacket. These features have made the concrete platforms attractive and competitive for fields like Statfjord and Gullfaks[(68)].

The major advantage of high strength concrete in platform design is its potential for weight reduction. This applies to both floating and fixed platforms. To illustrate the effects of increasing the concrete strength for a real platform, one of the candidate structures for the Troll East Field development, the Condeep T300 shown in Figure 2.3, has been considered. According to Jakobsen et al[(68)], this platform would have required 225,000 m^3 of concrete with characteristic compressive strength of 60 N/mm^2. Increasing the characteristic strength by 20 per cent and making some other, minor modifications to the design reduced the necessary concrete volume to 170,000 m^3, the construction time by eight months and therefore gave considerable cost saving. These comparisons were made by largely retaining the same structural shape and configuration for the two material qualities. Jakobsen et al[(68)] stated:

> "In his search for the "optimum" platforms the contractor has at his service: a unique construction material with high potentials for further development, construction techniques that have recently undergone vast improvements thus offering new flexibility in structural form, and advanced analytical capabilities that have been developed over years. The engineer thus has a firm and well founded basis for further development and optimization. If human fantasy and innovativeness are in keeping with this basis, new, tailor made and highly competitive concrete platforms will emerge in the years to come."

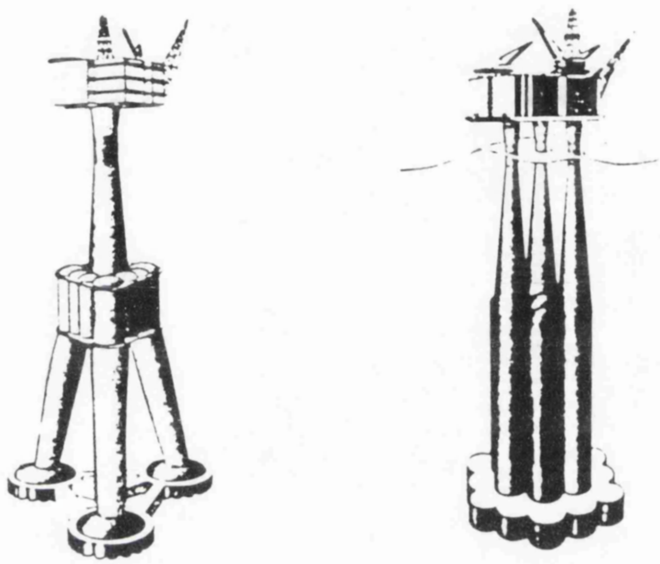


Figure 2.3: Two deepwater platform concepts that have been considered for several fields in the deepwater areas of the Norwegian continental shelf(68).

2.2.4 Other areas of application.

High strength concrete piles have been used for the Selmer-Sande building in Oslo[71]. In total, about 250 piles, 275 mm square, were driven in clay down to the rock at 25 to 35 m depth. Concrete with characteristic strength of 75 N/mm^2 was used instead of the standard strength of 55 N/mm^2, increasing the capacity of each pile from 1600 kN to 2400 kN, which consequently led to a reduction in the required number of piles. This again led to less pressure on the clay, therefore, less building up of pore pressure with better stability for later excavations and less ground settlements due to normalization of pore pressure. This again leads to less negative friction forces on piles with consequently increased direct load bearing capacity.

High strength concrete was used for slipforming two individual cylindrical silos at Porsgrunn, Norway[71]. Each silo has a diameter of 27 m, a height of 28 m and a wall thickness of 270 mm. The silos were for storing fertilizer (calcium nitrate) and therefore a high resistance to chemical attack was required. The mix design included condensed silica fume for pore filling between the cement particles, superplasticisers to maintain workability at low water content and entrained air. The net result was a concrete of very low permeability.

Copen[72] has indicated that the use of 70 N/mm^2 concrete in thin arch dams would usually result in greater economy through reduced volume of concrete. High strength concrete would tend to reduce deflections and may improve strength of construction joints and permit earlier removal of formwork.

In recent years, highway authorities in many countries have been spending an increasing proportion of their available resources on maintenance or rehabilitation of existing highway systems rather than on new construction programmes. In the Scandinavian countries, the extensive use of studded tyres poses a special threat to the highway pavements. Most of the concrete used for highway pavements and bridge decks so far has been with compressive strength of no more than 40 N/mm^2. Gjorv et al[73] carried out work on the abrasion resistance of pavements subjected to heavy traffic from studded tyres, and they found that increasing the concrete strength from 50 to 100 N/mm^2 reduced the abrasion by roughly 50 per cent. At 150 N/mm^2 the abrasion was comparable to that of high quality massive granite. They concluded that the abrasion resistance of the 150 N/mm^2 concrete as compared to an asphalt highway pavement represents an increase in the service life of the pavement by a factor of approximately ten. Subsequently, high strength concrete was used for a number of Norwegian highway pavements, e.g., the Valerenga Tunnel in Oslo (total area paved = 15,000 m^2), a new part of the E-18 and E-6 highways (total area paved = 110,000 m^2, that required 22,000 m^3 of concrete), and a repair job in Smestad Tunnel in Oslo (grooves with a width of 800 mm and 35 mm depth were milled and filled with high strength concrete)(74).

The increased abrasion resistance of higher compressive strength concrete has also been exploited in the re-surfacing of the stilling basin of Kinzua Dam, in 1983(75). Inspection of the 96 N/mm^2 concrete overlay, a year after it had been placed, suggested that it would have a much longer service life than normal strength concrete or the previously used fibre reinforced concrete overlay.

2.3 DISADVANTAGES OF HIGH STRENGTH CONCRETE.

Most of the disadvantages of using high strength concrete listed by engineers result from a lack of research and available information on its behaviour under actual field conditions(27). Possible drawbacks in using high strength concrete and how they have been alleviated in practice are now discussed.

2.3.1 Material selection and concrete mix proportioning.

High strength concrete is a state-of-the-art material, and like most such materials, it commands a premium price(32). Production that consistently meets requirements for workability and strength development places more stringent requirements on material selection than for lower strength concretes(1). For example, as mentioned earlier, to achieve the 7,500 psi (51.7 N/mm^2) concrete design strength for the construction of Texas Commerce Tower[58], it was necessary for the aggregate to be changed from river gravel to an imported limestone.

High strength concrete mix proportioning is a more critical process than the design of normal strength concrete mixtures and, therefore, many trial batches are often required to generate the data that enables the researcher to identify optimum mix proportions[1], (c.f. page 27). Usually, specially selected pozzolanic admixtures are employed and the attainment of a low water-binder ratio by the use of chemical admixtures is considered essential. The utilization of a second cementitious material, in powder or in a slurry form as is sometimes the case with condensed silica fume, and of chemical admixtures requires the batching plant to have the necessary facilities for the handling of these materials.

During the construction of the Willows Bridge[76] it was not possible to consistently produce 5,000 psi (35.2 N/mm^2) concrete after 24 hours of steam curing. Although the solution could have been the use of a high-early-strength (Type III) cement, the batching plant had no facilities for the handling of a second type of cement. Because of this, and as the initial mix proportioning work, carried out under standard curing conditions, had demonstrated the superior strength of the concrete made with limestone aggregate, this had to be substituted for the original gravel.

Even if the facilities for handling an extra cementitious material are available, Pioneer Concrete Ltd. has found that the use of condensed silica fume slowed down the production process in mass output[35].

2.3.2 Quality control.

In some instances, the benefits of high strength concrete more than compensate for the increased costs of raw materials; in others they do not. In New York (1983), the book price for 3,000 psi (20.7 N/mm^2) concrete was \$45 per yd^3; the price was \$140 per yd^3 for 14,000 psi (96.5 N/mm^2) concrete[52]. The prices, as quoted by Pioneer Concrete Ltd.

in October 1991[77], were £25 and £42 per m^3 for 20 and 80 N/mm^2 concrete, respectively. The increase is only 1.7 times in price for an increase of 4.0 times in load-carrying capacity for the concretes quoted for London, and 3.1 times in price for an increase of 4.7 times in load-carrying capacity for the concretes quoted for New York. This clearly demonstrates the economy of using high strength concrete in multistorey building columns. It has, however, been suggested that 30 storeys is the minimum height for a building for which high strength concrete is beneficial. This is because there is an additional factor, besides the selection of quality materials, that will influence costs; this is the cost of the increased testing, quality control, and inspection that the use of high strength concrete requires[1]. The quality and consistency of the concrete is crucial, and the following additional steps have been found necessary.

(i) The supplier has to have quality control personnel at the site to control both the scheduling of trucks and the consistency of the concrete at the time it is delivered - *(Royal Bank Project[1], The First Republic Bank Plaza[60])*.

(ii) The engineering testing agency has to employ a full-time technician to carry out sampling and testing on site - *(Royal Bank Project[1], The First Republic Bank Plaza[60], Scotia Plaza[61])*.

(iii) Periodic visits to the ready-mix batching plant are required to check the storage and handling of materials, and the batching - *(Scotia Plaza[61])*.

(iv) For strengths higher than 70 N/mm^2, the capping compound of sulfur or other material, usually added to the end of a cylinder for testing purposes, must be replaced by a grinding procedure - *(Seattle's Two Union Square building[30])*.

(v) Testing at later ages than 28-days requires early "trigger points" to detect potential strength problems. This can be done by knowing the level of strength the concrete should obtain at early ages in order to reach the design strength at the testing age, or by subjecting specimens to the autogenously cured accelerated test given in, for example, the ASTM C 684-81[78], and BS 1881: Part 3[79] - *(Columbia Center[57],* *Scotia Plaza*[61]).

2.3.3 Workability.

The placement of high strength concrete is a critical matter as its performance demands a dense, void-free mass with full contact with reinforcing steel. Without uniform placement, structural integrity may be compromised[1]. However, as high cement contents, in the range of 500 to 600 kg/m^3, are utilized in combination with superplasticisers, the mix workability in terms of "stickiness" may be adversely affected[1]. Saucier[12] called for improved vibration or compaction procedures to overcome this: "What consolidation improvements are required to overcome this? More powerful vibrators? All frequency vibrators? Ultrasonic vibration?". Specified slumps should therefore provide a workable mixture, easy to vibrate, and mobile enough to pass through closely placed reinforcement[1]. However, it is considered by many researchers[2,16,49,50] that the slump may give the wrong impression of the workability of high strength concrete. The inapplicability of slump tests in describing workability has been shown by, for example:

(i) Kakizaki et al[50], who investigated the behaviour of high strength concrete during vibration using the apparatus shown in Figure 2.4. They defined the "flowability value", given in seconds, as the time taken for enough concrete to flow out of cylinder A and subsequently fill the outer ring C. They concluded that the relationship between slump and concrete "flowability" was affected by the use of mineral admixtures; required slumps of superplasticised concrete containing CSF or PFA, to give the same "flowability" as a Portland cement concrete with a slump of 180 mm, were 210 and 60 mm respectively.

(ii) Helland[80], who investigated the workability of three mixes, with mix proportions as shown in Table 2.6, using Tattersall's two point apparatus. This measures the torque (T) produced on an impeller rotating in fresh concrete at various speeds (N). The impeller torque (T) and speed (N) are found to be related by the linear equation:

$$T = g + hN$$

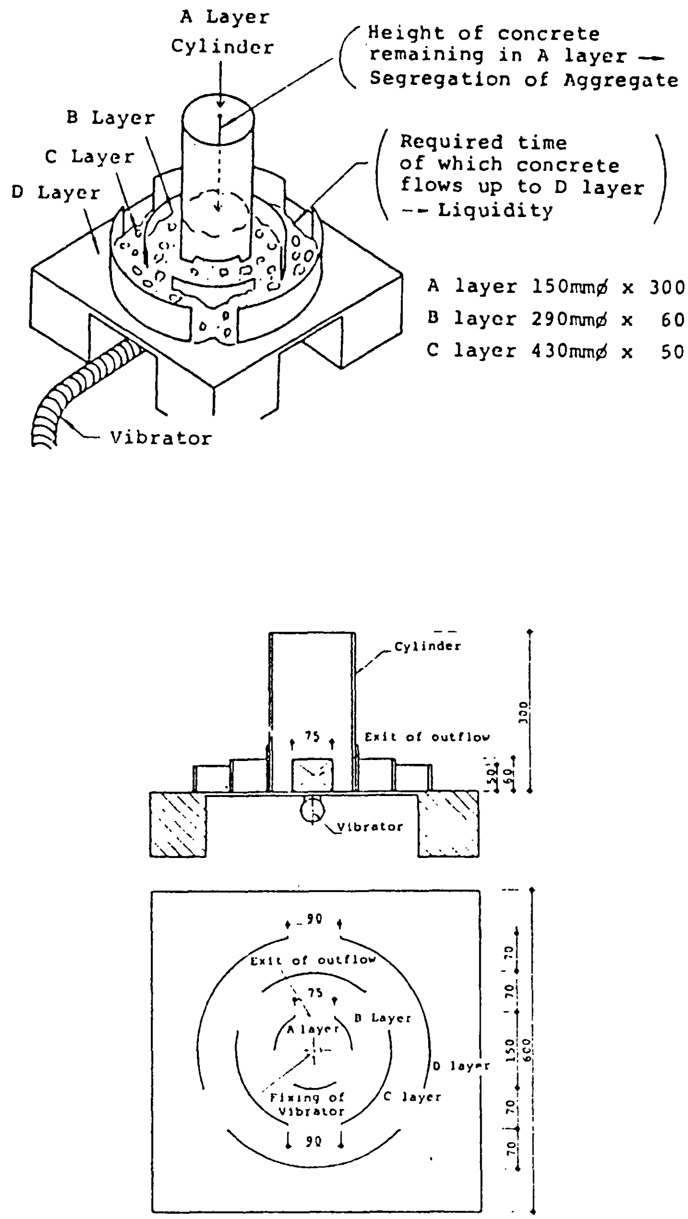


Figure 2.4: Apparatus for measuring the "flowability" of concrete during vibration[50].

Mix	Cement and silica fume (kg)	Total water content (ltr)	Superplasticiser N (ltr)	Melment L 10 (ltr)	Air entraining agent (ltr)	Air content (%)
E	313	203	–	–	–	1.4
F	340	118	16	–	–	2.9
G	293	112	–	15	0.2	7.5

Table 2.6: Concrete mix compositions investigated with the "two point workability apparatus"[80].

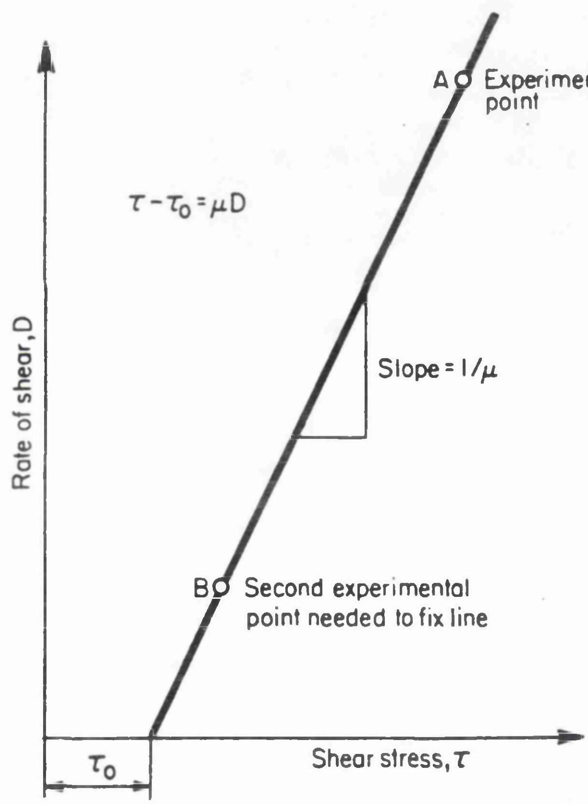


Figure 2.5: The Bingham model[81].

The resemblance of this equation to the rheological equation of Bingham flow, shown schematically in Figure 2.5[81], is often noted. Even with similar "g" values (and slump), (Figure 2.6), mixes E and F behaved quite differently on site. Mix E responded well to the poker vibrator, while mix F, with its extremely high dosage of superplasticisers, reacted badly. He attributed this behaviour to the greater internal resistance of Mix F at the higher rates of shear. This problem was solved by adding an air entraining admixture (Mix G) which decreased the plastic viscosity (h) value. He concluded from these experiments that the slump value reflects the situation when the rate of shear is near zero and that it gives very little information on how concrete behaves at higher rates of shear, (Figure 2.7), when high frequency poker vibrators are used.

It is not therefore surprising that there is not a general agreement on the minimum acceptable slump required for site placing. While Schmidt[17] suggested that the minimum is 65 mm, ACI Committee 363 report[1] stated that slumps less than 75 mm have made special consolidation equipment and procedures a necessity. It advocated that a slump higher than 100 mm will, normally, provide the required workability; however, details of forms and reinforcement bar spacing should be considered prior to the development of mix designs. In order to be able to understand how differently fresh high strength concrete will behave from traditional concrete concerning workability, and to be able to optimise the mix design according to the requirements given by the methods of placement and compaction, there is a need for a fundamental knowledge both of how to measure the workability and how to control it[2]. According to the FIP/CEB report[2], such knowledge should, if possible, be expressed according to classical rheological models such as for example those of Bingham and Newton. Tattersall's two point apparatus appears, from the limited tests carried out by Helland[80], to be more suitable than the slump test for high strength concrete mixes.

Workability of high strength concrete is not only difficult to define but it often declines rapidly with time after mixing[32]. According to Mehta and Aitcin[46], the workability loss of normal Portland cement concretes is usually associated with the formation of ettringite, a calcium sulphoaluminate hydrate. When water is added to Portland cement, calcium

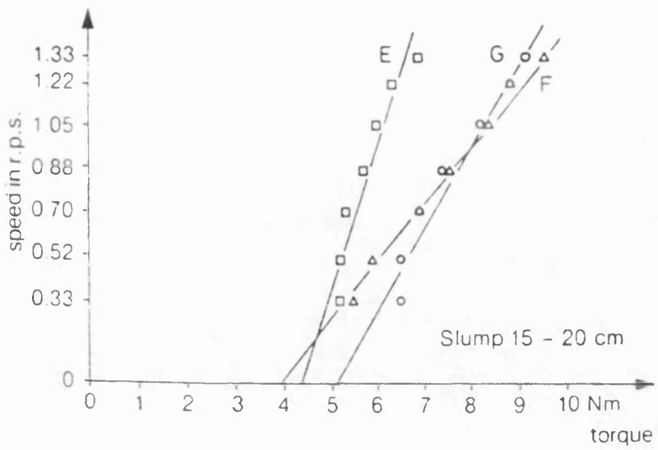


Figure 2.6: Yield (g) and plastic viscosity (h) values of the concretes with compositions as shown in Table 2.6[80].

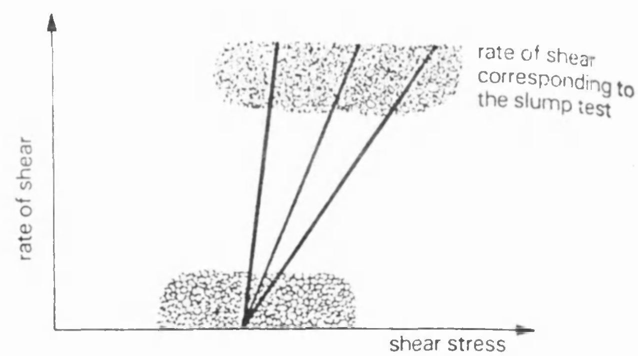


Figure 2.7: Plastic viscosity (h) values of fresh concretes with the same slump values[80].

sulphate and tricalcium aluminate are among the first anhydrous phases which go into solution. As a result, within a few minutes of cement hydration ettringite forms and, due to its low solubility, crystals of ettringite begin to precipitate out. The rate of the workability loss depends mostly on the rate of ettringite crystallisation and the morphology of the crystalline product. This is because flocs of precipitating crystals are able to entrap large volumes of free water; also, poorly crystalline ettringite, formed under certain conditions, is able to immobilise considerable free water by surface adsorption. Since high strength concrete mixtures will in general have higher cement contents, and therefore higher tricalcium aluminate content, and lower water contents than normal commercial concretes, the rate of workability loss may be higher. Therefore the placement of high strength concrete requires greater control of the scheduling of trucks as has been used for the above mentioned Royal Bank Project(1) and the First Republic Bank Plaza(59) where all trucks were equipped with radio-transmitters for close communication with the site.

Prior to using a retarder in the concrete mixes for the First Republic Bank Plaza(59) an investigation was made to study the effects on strength when the normal set and retarding admixtures were accidentally overdosed due to equipment malfunction or batchman error. According to Cook(59) these errors are not uncommon in the ready-mixed concrete industry. Their results having shown lower strengths, (Figure 2.8), with an overdose of the retarding water-reducing admixture, the decision was made to discontinue its use in conjunction with the superplasticiser. On arrival at the jobsite, the desired slump was successfully restored by redosing the concrete with the superplasticiser. The normal dosage of 42 oz./yd^3 (913 mL/m^3) was dispensed at the batching plant and 60 oz./yd^3 (1304 mL/m^3) were added on arrival at the jobsite.

2.3.4 Heat evolution during hydration.

The high cement contents that have generally been used for the production of high strength concrete can lead to high heat of hydration temperature rises even though the heat evolved per unit weight of cement is reduced at low water-cement ratios, (Figures 2.9[16,32]

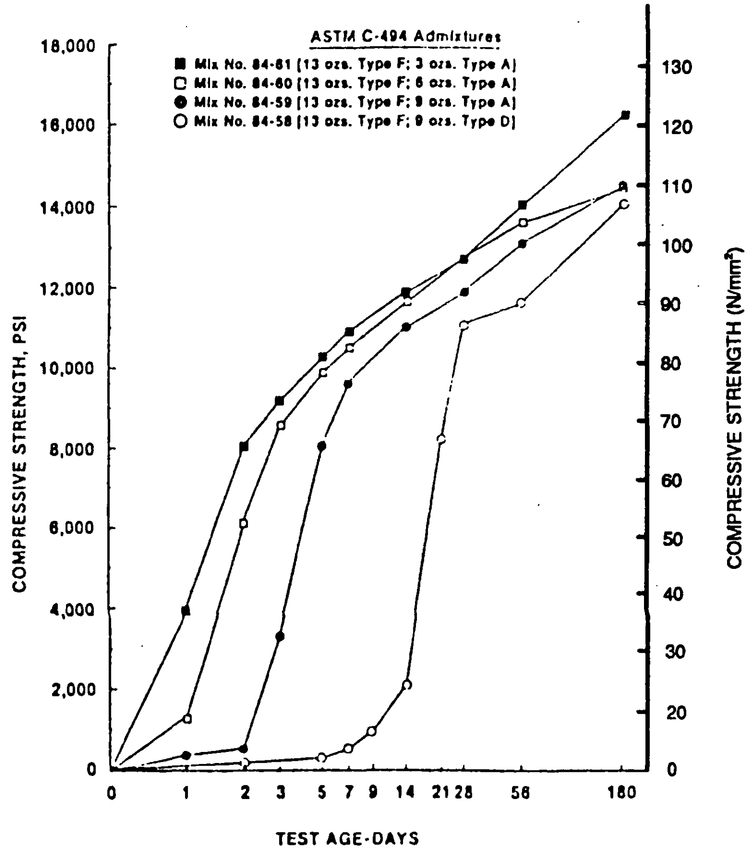


Figure 2.8: The effect of admixture type and dosage on the strength development of concrete[59].

TYPE A: Water reducer.
TYPE D: Retarder.
TYPE F: Superplasticiser.

and 2.10[82]). Aarsleff et al[82] attributed the lower heat to: "the increasing amount of unhydrated cement compounds with decreasing water-cement ratio, particularly when it is reduced below 0.35".

High early age temperatures can cause two problems:

(i) *Restraint stresses can lead to cracks.*

All concrete elements are strained to some degree because there is always some volumetric restraint provided either by the supporting elements or by different parts of the element itself. ACI Committee 207[83] has defined the term "restrain" as: "to hold back from action; check; suppress; curb; to limit; restrict". Restraint thus acts to limit the change in dimensions and produces strain, with corresponding stress in a concrete member. Numerically, the strain is equal to the product of the degree of restraint existing at the point in question and the change in unit length which would occur if the concrete were not restrained. Restrained volume change can induce tensile, compressive, or flexural stresses in the elements depending on the type of restraint and whether or not the change in volume is an increase or decrease. The restraint conditions which induce compressive stresses in concrete are not normally of concern because of the ability of concrete to withstand compression. The concern is primarily with restraint conditions which induce direct tensile or flexural stresses which can lead to cracking.

Theoretical prediction of the temperature changes and effects was not easy before the advent of the computer and reliance on the experience obtained from previous construction was the alternative[84]. This led to the recommendation that, as a rule of thumb, the differential should be kept below 20^{0}C to avoid cracking[85,86]. From the results of a thermocouple study in mass concrete in the Upper Tamar Dam, Dunstan and Mitchell[87] concluded that the analysis of the temperature differentials indicated that cracking occurs at a differential of 25-26^{0}C. They, however, advocated the use of 20^{0}C as the maximum differential for design purposes so as to allow for the increases due to severe changes in ambient temperature. This recommendation was questioned by Browne[88] who expressed his worry at the

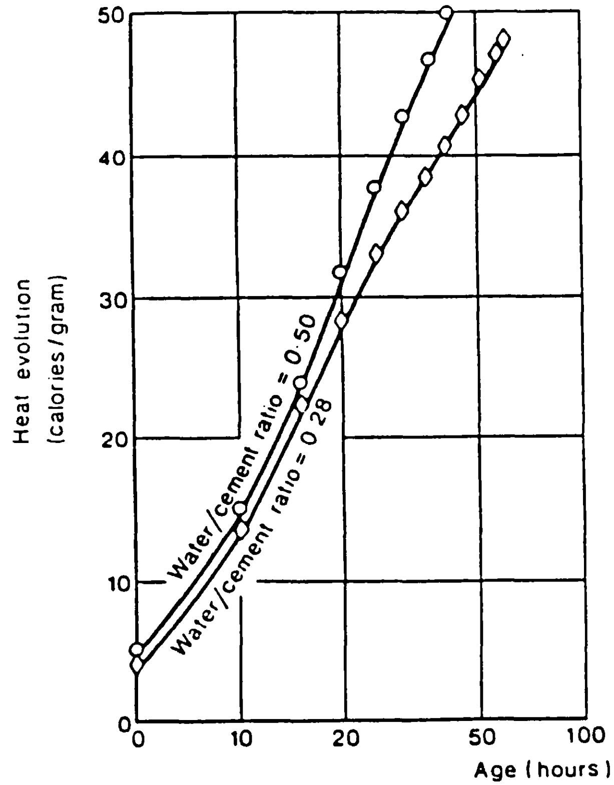


Figure 2.9: Effect of water-cement ratio on the heat evolution of cement[16,32].

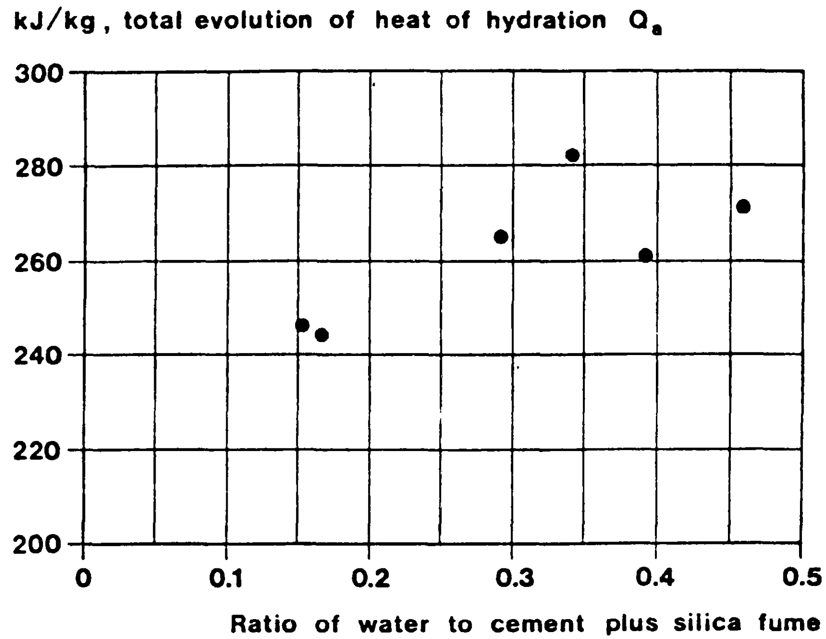


Figure 2.10: Total heat of hydration evolution versus water-binder ratio[82].

tendency to generalise in terms of temperature differentials, forgetting that thermal cracking is a stress concentration problem and temperature gradients are more important. Subsequently, Bamforth(89) showed that the modulus of elasticity and creep coefficient of concrete affect the temperature gradient which will cause cracking; the magnitude of thermal stresses in concrete will increase with an increase in the modulus of elasticity, and a reduction in creep will prevent adequate stress relief. Another property that influences the thermal stresses is the concrete's coefficient of expansion(83).

The recommendation of 20^{0}C minimum differential has also been questioned for its applicability to high strength concrete, which has a higher tensile strength and may therefore be able to tolerate higher differential strains(90). However, it is not only the higher tensile strength(91) but also the higher elastic modulus(91) and lower creep coefficient(8,92) that have to be considered. Nowadays, computers enable us to look at the many ramifications of the heat problem, and they should be used to determine the thermal gradients permissible in high strength concrete elements.

(ii) *The in-situ concrete strength may be lower than the laboratory concrete cured at the constant temperature of 20^{0}C.*

The effect of subjecting concrete to a temperature regime during its early life similar to that which would occur at the centre of a mass pour is to accelerate the early strength gain of the concrete but to impair the long term strength development. Bamforth's extensive study(93) on the beneficial effects of PFA and GGBFS in reducing the temperature rise of normal strength concretes, (described in more detail in Chapter 3), also showed the effect of the temperature cycle on the strength of concrete, (Figures 2.11, 2.12, and 2.13). He found that the relationship between the strength of heat cycled concrete and concrete cured in accordance with BS 1881: Part 3(79) was influenced by the use of both PFA and GGBFS. For Portland cement concretes the effect of the temperature cycle was to reduce the strength at 28-days by up to about 20 per cent, the effect increasing with casting temperature and cement content (and hence the early rate of temperature rise). The use of PFA to replace 30 per cent of the Portland cement

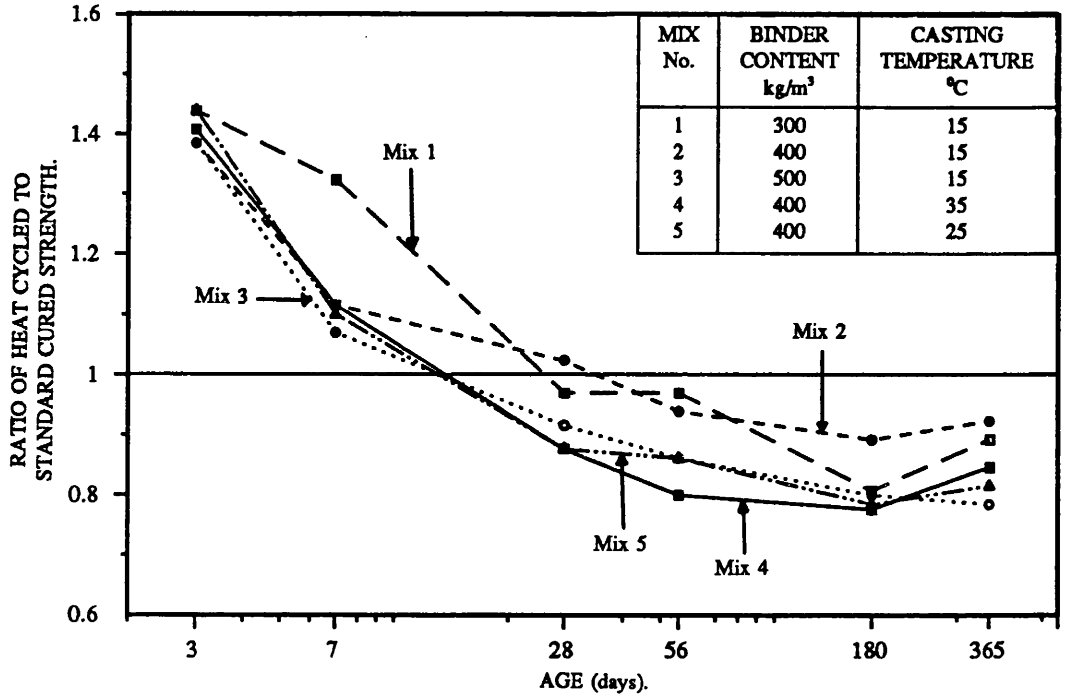


Figure 2.11: Relationship between standard cured and temperature matched cured strengths for OPC concretes[93].

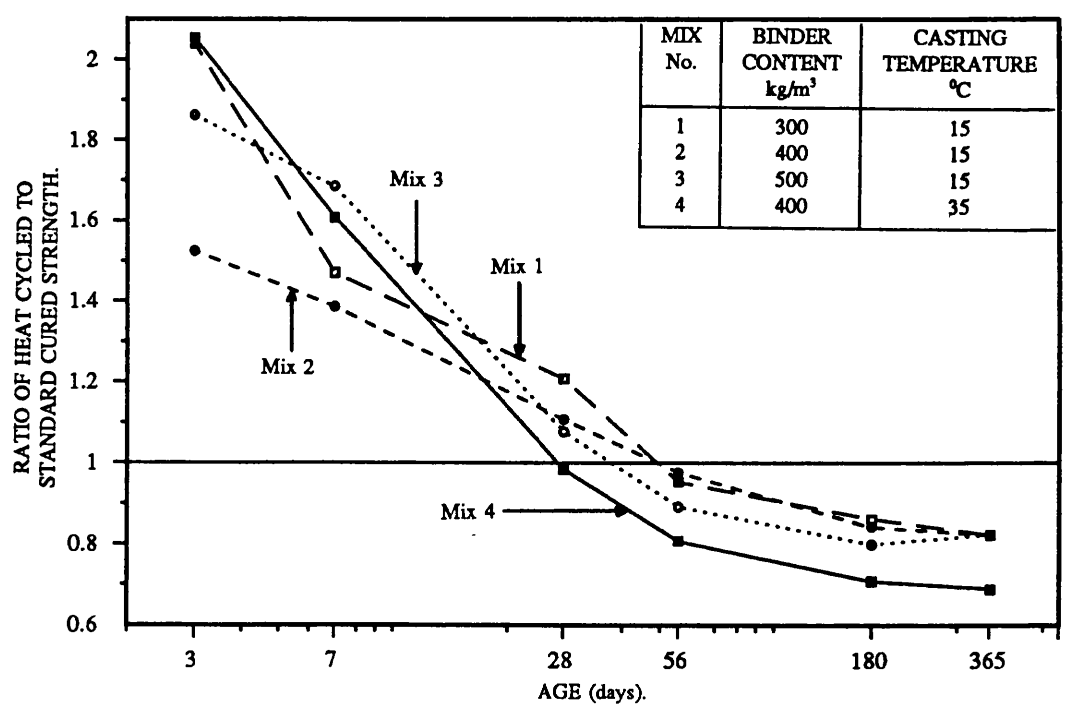


Figure 2.12: Relationships between standard cured and temperature matched cured strengths for concretes with PFA at the replacement level of 30 per cent[93].

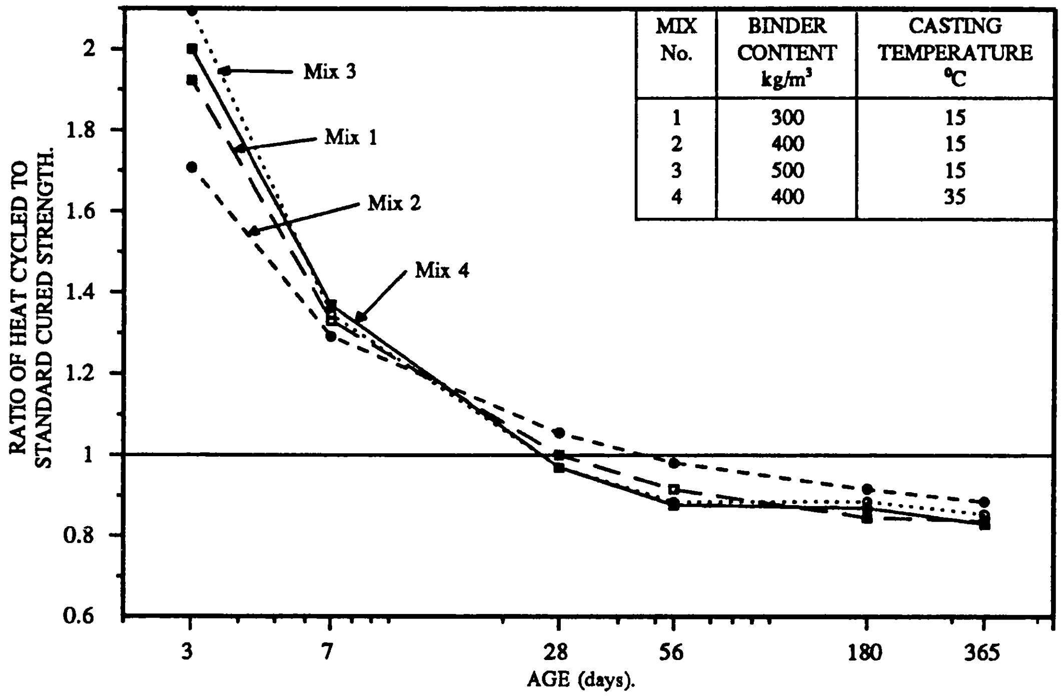


Figure 2.13: Relationship between standard cured and temperature matched cured strengths for concretes with GGBFS at the replacement level of 50 per cent[93].

Batched Time: 3:50 p.m.

	Batch Plant		Site (Position in Truckload)		
	Before Cooling	After Cooling	Start	Middle	End
Time	4:03	4:20	4:50	5:10	5:27
Concrete Temp (°C)	27	12	16	17	18
Air Temp (°C)	32 (Sun) 22 (Shade)	32 (Sun) 22 (Shade)	32	32	32
Slump (mm)	210	130	140	100	70

Table 2.7: Typical temperatures of materials prior to mixing and the effect of the nitrogen cooling during the construction of Scotia Plaza[61].

modified the relationship such that at 28-days the heat cycled strength was up to 20 per cent higher than the standard cured value. Despite the 28-day strength increase the effect of the temperature cycle was to reduce the strength at ages beyond 56-days. The use of GGBFS to replace 75 per cent Portland cement resulted in the heat cycled strength being marginally higher, of the order of 5 per cent, than the standard cured strength at 28-days, but again resulted in lower strengths at ages beyond 56-days. The impaired long term strength development, resulting from heating concrete at a very early age, is believed to be due to a fundamental change in the hydration products formed. Mechanical breakdown between the cement paste and aggregate as a result of thermal stressing has also been suggested.

Significant concern is, therefore, frequently expressed about the high heat evolution of high strength concrete, e.g., during the construction of the Columbia Center[57], Scotia Plaza[61], First Republic Bank Plaza[60], and the Texas Commerce Tower[58] and during trials by the British Cement Association[94].

There are two ways of moderating high early temperature rises:

(i) *Lowering the temperature of the fresh concrete as much as possible by the use of cooled or iced mix water, or liquid nitrogen cooling.*

Iced mix water was used during the construction of **Scotia Plaza**[61] as the contract specification restricted the maximum concrete temperature at 18^{0}C at the time of placing. However, because of the high temperature during the summer months, this was not possible to achieve by using ice in the mix. It was therefore agreed that nitrogen cooling of the concrete would be used. Typical temperatures of materials prior to mixing and the effect of the nitrogen cooling are shown in Table 2.7. There was no deleterious effect on strength.

The addition of ice to the batch in place of water may, however, add considerably to the cost. In order to avoid its use during the construction of the **First Republic Bank Plaza**[60] a study was carried out to examine the relationship between the maximum allowable temperature of the fresh concrete, and its compressive strength at 28 and 56-days. According to Anderson[60], concrete placed at 38^{0}C indicated only a small reduction in compressive strength at 56-days, as compared with similar concrete placed at 32^{0}C. Since the slightly diminished strength was acceptable to the engineer in view of the conservative margin of safety in the selected design, it was decided not to use ice. Also, during the construction of the **Veitshochheim Railway Bridge**[95] it was decided to use liquid nitrogen cooling instead of ice as the latter was uneconomical because of the small total concrete volume of 1450 m^3.

(ii) *The mix composition must be optimised, i.e. the cement content must be kept to the minimum required and if possible be partially replaced by a low heat binder.* When selecting the type of cement and mineral admixture not only a low heat of hydration but also a sufficient strength development has to be considered. Structural applications of high strength concrete where this was considered are now examined:

(a) During the construction of the 75-storey **Texas Commerce Tower**[58], a significant volume of high strength concrete was placed during Houston's hot summer months. Therefore, a revision of the high strength concrete mixes was recommended to offset the rate of cement hydration, (Table 2.8). Laboratory studies indicated that the 20 per cent PFA contents used in the original mix proportions was well below the optimum amount that could be used, (Figure 2.14). The effects of increasing the ratio of Class C (high calcium) PFA on the heat of hydration of the cement paste, (ASTM C 186-78 standard test method was used)[97], and the compressive strength of the concrete were therefore investigated. Test results indicated that an 11 per cent reduction of the heat of hydration at 7-days was obtained with 20 per cent cement replacement by PFA, 25 per cent reduction with 30 per cent and, surprisingly, only 18 per cent reduction with 40 per cent. On consideration of these results, other laboratory studies, and field evaluation of proposed mixes, a 30 per cent PFA content was approved for all concrete mix proportions. The original and revised mix

Mix identification No.	6000		6000-Revised		5000		5000-Revised	
	lb/yd^3	kg/m^3	lb/yd^3	kg/m^3	lb/yd^3	kg/m^3	lb/yd^3	kg/m^3
CEMENT	540	320.2	473	280.5	470	278.7	413	244.9
PFA	135	80.1	202	119.8	120	71.2	177	105.0
LIMESTONE	1900	1126.7	1900	1126.7	1900	1126.7	1900	1126.7
SAND	1176	697.4	1184	702.1	1296	768.5	1213	719.3
WATER	283	167.8	267	158.3	267	158.3	267	158.3
ADMIXTURE	21.5	467	21.5	467	18.8	408.4	18.8	408.4
	fl. oz.	mL/m^3	fl. oz.	mL/m^3	fl. oz.	mL/m^3	fl. oz.	mL/m^3
WATER-BINDER RATIO	0.419	0.419	0.395	0.395	0.453	0.453	0.453	0.453
Mix properties.								
SLUMP	4 in.	100mm	3.75-in.	95mm	3.25-in.	85mm	4.25-in.	110mm
WATER USED	254	150.6	240	142.3	249	147.7	244	144.7
	lb/yd^3	kg/m^3	lb/yd^3	kg/m^3	lb/yd^3	kg/m^3	lb/yd^3	kg/m^3
WATER-BINDER RATIO	0.36		0.35		0.42		0.41	
PFA, % BY WEIGHT	20.0		30.0		20.0		30.0	
Average Compressive Strength	psi	N/mm^2	psi	N/mm^2	psi	N/mm^2	psi	N/mm^2
7-DAY	6508	44.9	6384	44.0	6083	41.9	5872	40.5
28-DAY	8196	56.5	8082	55.7	7631	52.6	7586	52.3
56-DAY	8488	58.5	8966	61.8	8232	56.8	8515	58.7
180-DAY	9992	68.9	10788	74.4	10221	70.5	10381	71.6

Table 2.8: Original and revised concrete mix designs used for the construction of the 75-storey Texas Commerce Tower[58].

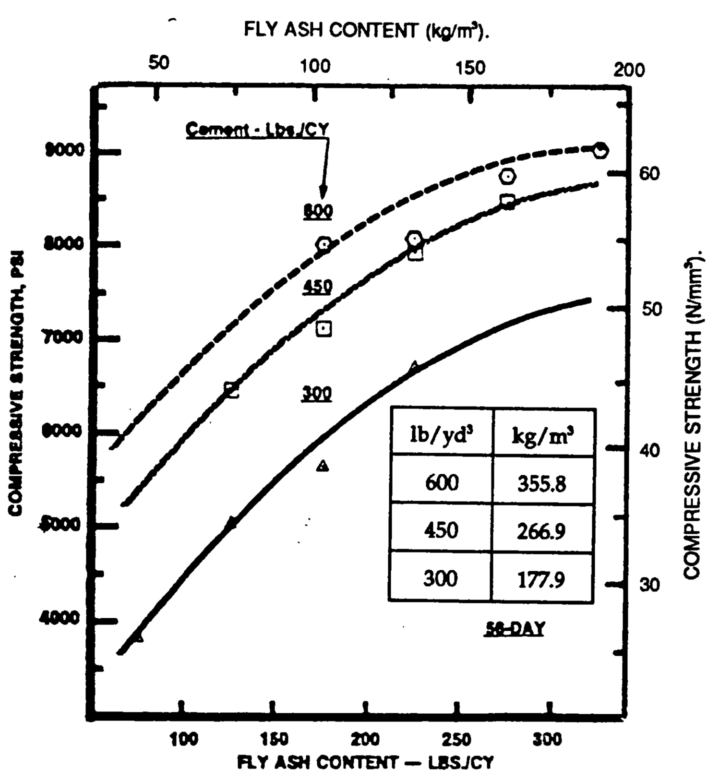


Figure 2.14: The effect of increasing ratio of Class C (high calcium) PFA on the concrete compressive strength[58].

proportions are shown in Table 2.8.

(b) During trials by the **British Cement Association**(94), carried out at the Acton plant of Pioneer Concrete (UK) Ltd., peak temperatures of 97^0C were recorded at the centres of 2.4 x 1.2 x 1 m blocks. The mix details and peak block temperatures are given in Table 2.9. The peak temperature of 97^0C and the cracking of one of the Lytag blocks led to the investigation of concretes with 40 per cent by weight of the OPC being replaced by ground granulated blast furnace slag. Four more 2.4 x 1.2 x 1 m blocks were cast with the revised mix proportions shown in Table 2.10. This time, however, the blocks were completely covered with 25 mm thick expanded polystyrene to minimise differential temperatures, which, according to Clarke and Adams(94), probably caused the cracking of the first Lytag block. Despite the use of 40 per cent GGBFS in the revised mixes, the peak temperatures recorded were again very high, 70 to 90^0C, (Table 2.10).

(c) The 10,000 psi (69.0 N/mm^2) concrete (binder content = 510 kg/m^3) used for the construction of the **First Republic Bank Plaza**(60) had a cement replacement level by Class C (high calcium) PFA of 29.5 per cent. Despite this, the maximum temperature in the full scale model of a typical 6 x 6 ft (1.83 x 1.83 m) column section reached more than 93^0C about 20 hours after placement during summer weather conditions. Thus, there existed a thermal gradient of approximately 60^0C per metre from the centre to the face of the concrete. To attenuate the thermal gradient, and thus minimise the internal stress during the early curing history, insulating blankets were applied to the columns immediately after placement, and left in place for seven days for both summer and winter concreting.

(d) The 9,500 psi (65.5 N/mm^2) concrete (binder content = 502 kg/m^3) for the **Columbia Center**(57), used a combination of Type I (ordinary Portland) and type II (moderate heat Portland) cement, and also had a cement replacement level by Class F (low calcium) PFA of 20 per cent. Despite

Mix	Lytag (kg/m³)	Limestone (kg/m³)
Binder: OPC	580	395
Microsilica (by wt. of OPC)	54 (9.3%)	32 (8%)
SPA (SG about 1.18)	8 (Rh 561)	6 (Rh 716)
Fine aggregate zone 3/2	290	680
Coarse aggregate 12 mm FG or 10 mm ss	785	355
Coarse aggregate 20 mm ss	-	825
Free water	169	158
TOTAL (approx. cube density)	1886	2451
Free w/c by weight	0.295	0.40
Free w/b by weight	0.27	0.37
Slump, 75 minutes	235	90
Temperatures at centre of 2.4 m x 1.2 m x 1.0 m high blocks (°C)		
Initial	29	27
Peak	97	67
Rise	68	40
Peak period (estimated)	20 - 24 h	16 - 20 h
100 mm cube crushing strength (MPa) Mean of 2 at BCA (Fig. 2 & 3)		
Water cured at 7 days	60.5	62.5
Sealed at 28 days	68.0	81.5
Water cured at 28 days	65.5	83.5
Water cured at 56 days	67.5	86.5

Table 2.9: Trial mixes cast at Acton on 20th, August 1987(94).

Mix	1	2	3	4
Aggregate coarse / fine	Lytag and River sand, zone 2/3 estimated	— Limestone —		
	kg/m³ SSD aggregates			
OPC + ggbs	595	505	490	590
Microsilica	56	40	40	-
SPA (Rh) Elsac	7.8(716)	17E	7.9(1000)	9.6(561)
Fine aggregate	325	590	580	440
Coarse 10 or 12 mm FG	710	360	355	380
20 mm	-	845	825	875
Free water	211	152	145	160
TOTAL (approx.)	1900	2500	2440	2455
w/c + ggbs	0.355	0.300	0.295	0.285
w/c + ggbs + ms	0.325	0.280	0.275	-
slump (mm)	60	Collapse	10	105
after ( ) mins	(60)	"FISM" (60)	(65)	(60)
NB FISM - flowing in slow motion - like treacle				
Temperatures at centres of 2.4 m x 1.2 m x 1.0 m blocks (°C)				
Initial	26	24	27.5	26.5
Peak	90	70	72	82
Rise	64	46	44.5	55.5
Ambient approx. mean	10	10	10	10
Peak period (h)	22-29	27-29	21-25	35-41
100 mm cube crushing strength (MPa) 20°C water cure at BCA (see Fig. 3)				
Age 7 days	53	96	73.5	68.5*
Age 28 days	67.5	121	93.5	85.5+
Age 56 days	69.5	124.5	103.5	89

* Mean of 77 Mpa at Pioneer Laboratory and 94 MPa at BCA Laboratory

\+ Estimated from 82 MPa at Pioneer Laboratory

Table 2.10: Revised mixes with 40 per cent GGBFS(94).

this, the maximum temperature in the 6 x 6 x 6 ft (approx. 1.8 x 1.8 x 1.8 m) test blocks reached 180^{0}F (approx. 82^{0}C). Surprisingly, the differential recorded between the inside and the outside was only 35-40 F (approx. 19-22^{0}C). The blocks were closely monitored for several months and none showed any visible surface cracking. Another concern was that of possible strength variation due to the temperature differential. Cores taken from the blocks at 28-days indicated the average strength of the concrete to be within 110 psi (0.8 N/mm^2) from the outside to the inside, and only 500 psi (3.5 N/mm^2) below that of the companion laboratory cured cylinders. Bamforth's PFA Mix No. 4, (Figure 2.12), also showed a very small decrease in the 28-day strength, but the strength decrease due to the temperature cycling of the concrete was higher than 30 per cent at 180-days. It is therefore surprising that the strength variation was, for the Columbia Center, investigated only at 28-days while the design strength of the 9,500 psi (65.5 N/mm^2) concrete was for 180-days.

It may therefore be concluded that attempts to moderate high early temperature rises by lowering the temperature of the fresh concrete, have mainly been with the use of liquid nitrogen, since the use of iced mix water was uneconomical. Despite the use of low heat binders, such as type II (moderate heat Portland) cement, PFA and GGBFS, the peak temperatures recorded are as high as 90^{0}C. This therefore raises some doubts as to the effectiveness of PFA and GGBFS in reducing the temperature rise of high strength concrete mixes.

2.3.5 Structural effects.

This section deals with the design and performance of structural elements made with high strength concrete. Although this is beyond the scope of this work it is included for completing the list of the disadvantages.

According to Zia(96), in current codes, many design parameters affecting the strength and behaviour of structural members are related empirically to the compressive strength of concrete. These empirical parameters are based on experiments and experience with concrete having a compressive strength considerably lower than 9,000 psi (62 N/mm^2). Therefore, one must question if the present code provisions would still provide suitable assurance of safety and serviceability for high strength concrete and if there are any limitations that would prevent its effective use. Some of the provisions affected by the properties of high strength concrete have been identified and possible modifications have been suggested as described below:

(i) Lateral as well as spiral reinforcement has been found to be less effective in creating confinement stresses during failure[1,98,99]. Improved versions of the code equations for the prediction of the strength of the columns have been suggested by Martinez et al[99].

(ii) High strength concrete tends to be brittle when overloaded to failure, lacking the plastic deformation typical of lower strength concrete[99]. Lack of ductility leads to the possibility of brittle failure and therefore raises, justifiably, a serious concern for structural and human safety.

(a) In order to enhance the ductility of high strength concrete columns, the new Norwegian Code NS 3473[6] includes detailing rules where the cross section and density of stirrups must be increased for concrete characteristic strength above 60 N/mm^2. Also the minimum longitudinal reinforcement has to be increased in proportion to the design compression strength. In addition, the increased strength due to spiral reinforcement is not allowed for concrete characteristic strength above 55 N/mm^2. Similar suggestions for enhancing the ductility of high strength lightweight concrete have been proposed by Uzumeri and Basset[100].

(b) The question of beam ductility is an obvious one, because of the more brittle nature of high strength concrete. Remarkably, even though the compressive strain limit in flexure was less for high strength than for normal strength beams, both section ductility and member deflection ductility were actually greater, according to tests run at Cornell[101]. A more recent study by Uzumeri and Basset[100] verified that it is possible to get significant flexural ductility from beams constructed with both normal weight and lightweight high strength concrete.

2.3.6 Fire resistance.

High strength concrete at high temperatures shows a greater loss of strength than normal strength concrete, especially in the temperature region from 100 to 350^{0}C[2]. This, in combination with its lower permeability, increases the risk of damage by fire.

The most important factors which have an influence on the spalling of concrete are the moisture condition and permeability of the concrete. The moisture heats up to a vapour which causes a pore pressure near the surface, consequently forcing the free water inward. In a concrete with a low permeability, like high strength concrete, this flow is prevented, and the pore pressure may exceed the tensile capacity of the concrete with spalling occurring as a result. The higher risk of spalling has to be taken into account in the design of high strength concrete elements[2].

2.4 CONCLUSIONS.

High strength concrete has been used in high-rise buildings, bridges and offshore platforms. Because of its technical and economical advantages, summarised in Table 2.11, its use continues to grow.

Many proponents of high strength concrete also indicate that it has certain disadvantages, summarised in Table 2.12, which although not insurmountable can dilute or, in some cases, negate its advantages. Some of these have been chosen to be studied in this programme, and they are:

1.	High strength concrete enables the amount of costly reinforcing steel to be reduced without sacrificing strength.
2.	High strength concrete has a comparatively greater compressive strength per unit weight and unit volume. This advantage has been exploited in: *a) Buildings:* The number of storeys can be increased while maintaining an acceptable column size at the lower floors of high rise buildings. The reduction of the dimensions of columns and beams also results in a decrease of the dead weight of the structure, which can substantially lessen the design requirements for the building's foundations. *b) Bridges:* Span capability can be increased, or the girder depth reduced. The latter will allow lighter and more slender bridge piers, consequently reducing the load that has to be carried by the foundations. *c) Offshore Platforms:* The resulting reduction in the required concrete volume reduces the construction time and therefore gives a considerable cost saving.
3.	The additional stiffness provided by high strength concrete with its high modulus of elasticity reduces the amount of sway in the upper storeys of buildings.
4.	The relatively high early concrete strengths allow early stripping of the formwork, thus accelerating the construction.
5.	Architectural features may favour a reinforced concrete or composite, concrete/steel, design, concrete being mouldable to architectural advantage over steel.
6.	High strength concrete's increased abrasion resistance has been exploited in, for example, Norwegian highway pavements.
7.	The low creep of high strength concrete means that prestress losses should be less than for lower strength concrete members.

Table 2.11: Advantages of high strength concrete.

1.	Increased cost per unit volume. Production of high strength concrete places more stringent requirements on material selection and the mix proportioning becomes a more critical process than for lower strength concretes.
2.	High strength concrete requires increased testing, quality control and inspection. The quality and consistency of the concrete is crucial and additional steps must be taken to insure that quality and consistency.
3.	High heat evolution may necessitate use of low-heat binders and cooling measures to avoid early age thermal cracking.
4.	Workability of high strength concrete is difficult to define and often declines rapidly after mixing. Timing of the concrete delivery and the addition of the superplasticiser become critical.
5.	Structural effects: a) Lateral as well as spiral reinforcement is less effective in creating confinement stresses during failure. b) Lack of ductility of high strength concrete columns leads to the possibility of brittle failure. c) Use of high strength concrete is not specifically sanctioned by codes.
6.	High strength concrete at high temperatures shows a greater loss of strength than normal strength concrete, and a higher risk of spalling.

Table 2.12: Disadvantages of high strength concrete.

(i) the rheological properties of high strength concrete,

and, (ii) the effectiveness of mineral admixtures, i.e. PFA, GGBFS and CSF, in reducing the adiabatic temperature rise.

However, the investigation of these parameters required firstly the development of high strength concrete mixes. Existing guidelines for the selection and proportioning of materials to produce these are therefore reviewed in the next chapter.

CHAPTER 3.
LITERATURE REVIEW:
MATERIAL SELECTION AND MIX DESIGN OF HIGH STRENGTH CONCRETE.

3.1 INTRODUCTION.

As we have seen in the last chapter, it is both technically and economically feasible to produce, using readily available materials and conventional techniques, concrete having a characteristic compressive strength of 14,000 psi (96.5 N/mm^2) and above[30]. However, this has not been achieved by chance; it is a concrete in which all the factors that contribute toward an increase in strength have been maximised and those that can lessen strength have been minimised. Developing a high strength concrete may or may not require the use of special materials, but it definitely requires materials of highest quality and their optimum proportioning[45]. It has therefore required intensive research work, the establishment of an efficient system of quality control and a good knowledge of what helps and what hinders achieving a concrete of good quality[102]. It is surprising that, quoting Laplante and Aitcin[103]:

> "As to why and how very high strength concrete is made, the readily available answers to the first question contrast with the predominately empirical approach that has characterized research into producing very high strength concrete up to now. In fact, there are no miracle mixes that will universally guarantee the availability of 100 N/mm^2 ready-to-use concretes. Nonetheless some guidelines have been established that should be followed in order to avoid pitfalls."

These guidelines are now reviewed.

3.2 SELECTION OF CONSTITUENT MATERIALS.

The production of high strength concrete that meets requirements for workability and strength development places more stringent requirements on material selection than for lower strength concretes. Quality materials are needed and some exacting specifications are required[1]. According to Aitcin[102]:

"It really takes the best of everything."

3.2.1 Cement.

The properties of the cement that influence early and final strength development are[2]:

(i) *Composition.* The cement compounds C_3S, C_2S and C_3A have the greatest influence on the strength development in cement paste. C_3S contributes both to a rapid early age strength development and a high final strength. C_2S hydrates somewhat slower, but can contribute significantly to the final strength. C_3A has particular influence on the early strength. Other components in the cement may also have an influence, for example, a high content of alkalies will result in an increased early strength and reduced final strength potential.

(ii) *Fineness.* The rate of hydration of the clinker minerals is influenced by the fineness of the cement. A high specific surface leads to a rapid reaction but may, however, reduce the strength development after 28-days of curing.

The effects of the above parameters were considered by the Norwegian company Norcem Cement in developing new cements in response to higher concrete strength requirements and developments in the design process of the North Sea platforms[69]. The cement designated SP30-4A, was developed in 1978 to replace the standard cement, an ordinary Portland cement designated SP30, that was then being used for the offshore platforms. The SP30-4A had lower C_3A, C_3S and alkali contents but a higher C_2S content, (Table 3.1).

PROPERTY	SP 30	SP 30-4A	SP 30-4A MOD
Fineness (Blaine) cm^2/g	3000	3100	4000
Setting time (min.)			
• initial	120	140	120
• final	180	200	170
Mineral composition			
% C_2S	18	28	28
% C_3S	55	50	50
% C_3A	8	5.5	5.5
% C_4AF	9	9	9
Chemical composition			
% MgO	3	1.5 - 2	1.5 - 2
% SO_3	3.3	2 - 3	2 - 3
% Na_2O-ekv.	1 - 1.2	0.6	0.6
Heat of hydration kcal/kg	71	56	70

Table 3.1: Compound compositions and fineness of the cements used in concrete for North Sea platforms[69].

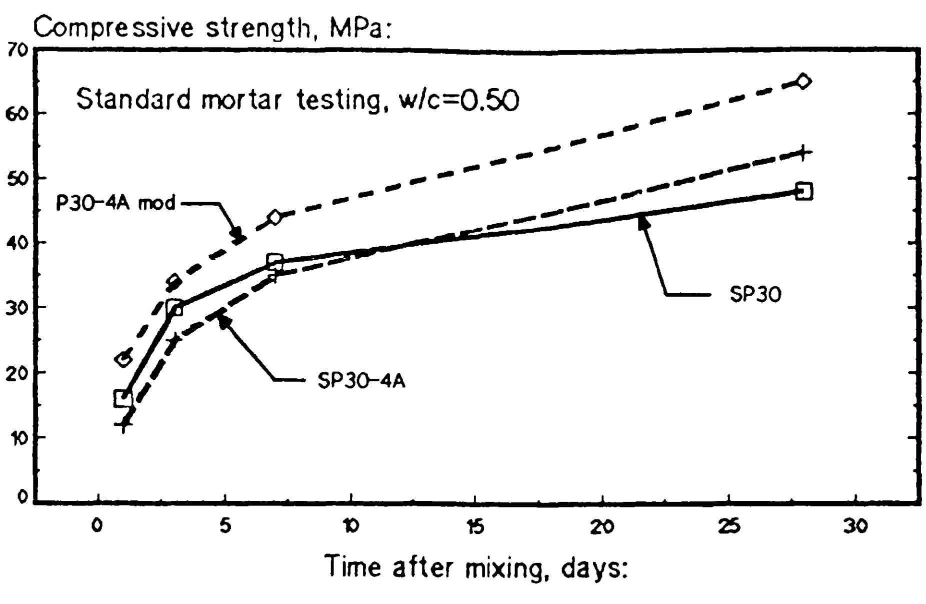


Figure 3.1: Compressive strength development of the cements used in concrete for North Sea platforms[69].

Typical properties for this cement were high long term strength, (Figure 3.1), and a moderate heat of hydration. An increased setting time and a decrease in early strength were, however, serious disadvantages during slipforming of the shafts where a rate of at least 10 ft (3 m) per day was desirable. To achieve a reduced setting time and increased early strength, Norcem modified the SP30-4A by increasing the fineness, to produce SP30-4A MOD in 1981.

Norway appears to be the only country that has developed a new cement specifically for the production of high strength concrete. Other regions, e.g., Chicago, have produced high strength concrete using readily available cements, but by carefully selecting its type and brand. Indeed, Blick et al(104), Lacroix and Jaugey(31), and the Chicago Committee on high rise buildings(8) stated that the selection of both type and brand of cement *is* extremely important. They recommended that careful studies be made of variations within one brand and between brands for any area of the country which has plans to produce high strength concrete. Strength differences attributable to different brands of cement have been shown to be less than 15 N/mm^2 at 28-days(104), (Figure 3.2), but Bedard and Aitcin(105) indicated that differences could be larger, (Figure 3.3). These differences arise because of the variations in compound composition and fineness that are permitted by ASTM C 150(106) and BS 12(3).

The combined effect of different compound compositions and various finenesses on the strength potential of the cements is so complicated that it can only be accurately assessed by concrete trial mixtures. The silo test certificates include mortar cube tests (according to ASTM C109(107) and BS 12(3)) which will give an indication of the strength characteristics of the cement. However, it has been recommended in many reports(1,8,11,102,104,108) that because the mortar compressive strength does not always vary in the same way as the concrete compressive strength, the final selection of a cement should not be based solely on mortar cubes but more so on its performance in concrete. The trial concrete mixtures should, moreover, contain the materials to be used in the job and, also be prepared at the proposed slump. This recommendation appears to be based on the study carried out by Blick(108) on the effect of five cements on the strength development of mortar and concrete. The chemical and physical analyses of these cements are shown in

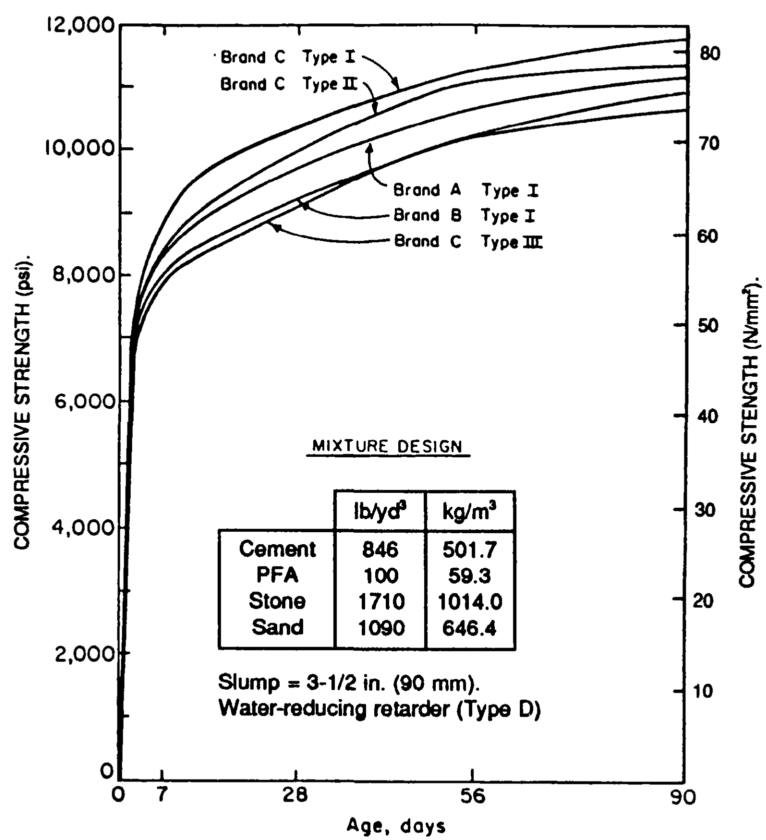


Figure 3.2: Strength differences attributable to different brands and types of cements available in Chicago[104]. Type I is an ordinary Portland cement. Type II is a moderate heat Portland cement. Type III is a rapid hardening Portland cement.

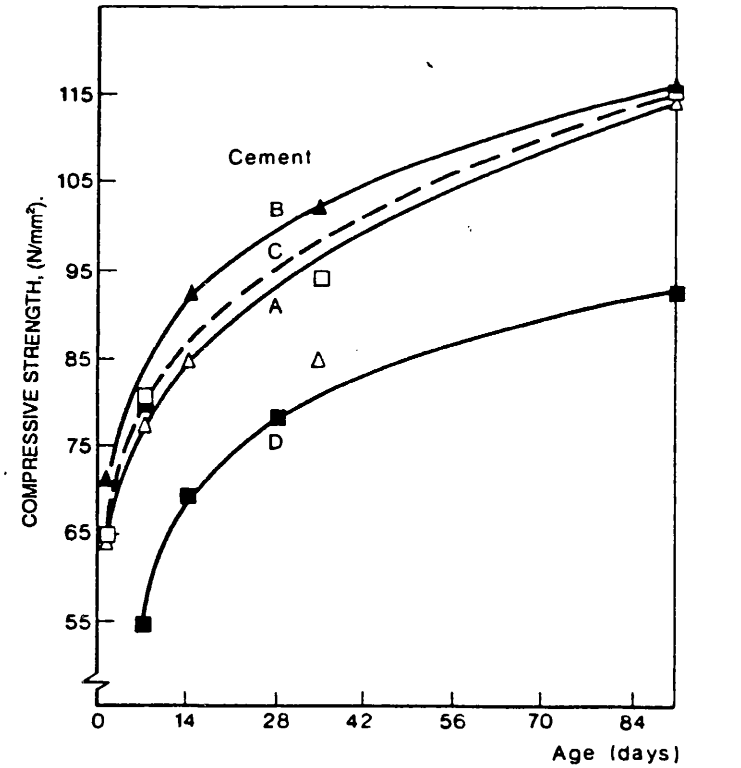


Figure 3.3: Strength differences attributable to different Canadian brands of Type 30 (high early strength) cement[105].

Table 3.2, and their mortar cube strengths, determined according to ASTM C 109-70 which requires the use of a water-cement ratio of 0.485, are shown in Figure 3.4. The concrete cylinder strengths, (Figure 3.2), were obtained from trial mixtures that were prepared at a slump of approximately 3 in. (75 mm). Although Blick has not reported the water-cement ratios used for these, he reported that:

> "With the exception of Brand B, Type I (ordinary Portland) cement, the water demand for concrete trial mixtures resulted in compressive strengths that conformed to Abrams relationship between the water-cement ratio and the strength. The mixtures that had the highest water demand resulted in the lowest compressive strength."

Despite the use of a chemical admixture, it appears that the required slump was obtained by adjusting the water content, while the dosage of the chemical admixture was kept constant. The ratio of (water-binder ratio of concrete)/(water-binder ratio of mortar) was therefore different for each brand and type of cement, and consequently their test showed that no relationship existed between the mortar and concrete strength, (Figure 3.5). Further research is therefore required to determine if silo test certificates that include only mortar cube tests will give a reliable indication of the strength potential of the cements in concrete.

Peterman and Carrasquillo[27] also based their selection of the Portland cement on strength tests of concretes with equal workability and therefore different water-cement ratios. Type II (moderate heat Portland) cement had, except for the 10 sacks/yd^3 (557.4 kg/m^3) mix with no superplasticiser, lower mixing water requirements than mixes made with Types I and III (ordinary and rapid hardening Portland) cements, and therefore resulted in higher compressive strengths, (Figures 3.6 and 3.7). Type I (ordinary Portland) cement, however, had lower compressive strengths than Type III (rapid hardening Portland) cement for superplasticised concretes, (Figure 3.7), despite the lower water-cement ratios.

The laboratory study that preceded the construction of the Willows Bridge[76] showed that the Type III (rapid hardening Portland) cement had superior strength producing properties

PROPERTY	Brand A Type I	Brand B Type I	Brand C Type I	Brand C Type II	Brand C Type III
SiO_2, %	21.8	20.6	20.5	21.9	20.4
Al_2O_3, %	5.3	6.1	5.7	4.1	5.5
Fe_2O_3, %	2.0	3.2	2.1	2.9	2.0
CaO, %	65.2	63.2	63.8	65.1	63.6
MgO, %	2.5	2.8	3.6	2.7	3.5
SO_3, %	2.1	2.6	2.5	2.0	3.0
Ignition Loss, %	1.1	1.1	2.0	1.0	2.1
Na_2O, %	0.19	0.31	0.11	0.14	0.12
K_2O, %	0.33	0.59	0.09	0.15	0.12
Na_2O Equiv., %	0.41	0.70	0.17	0.24	0.20
C_3S, %	55	48	55	61 *	56
C_2S, %	21	23	17	17	17
C_3A, %	10.6	10.7	11.5	5.9 *	11.2
C_4AF, %	6.1	9.7	6.4	8.9	6.1
Blaine (Air Permiability)	3670	3780	3550	4220	5400
+325	4.1	8.4	5.6	2.5	0.9
N.C.	25.8	23.6	23.0	24.8	29.6
Setting Time Vicat	2:35	2:15	1:35	1:55	1:45
False Set:					
H_2O, %	30.0	30.0	30.0	30.0	33.0
3 min.	50	50	50	50	50
5 min.	35	50	50	50	50
8 min.	18	50	50	46	45
11 min.	12	50	50	40	39
Remix, 15 min.	11	None	None	50	50
Air Content, %	9.1	9.5	7.6	6.6	6.2
H_2O, %	69.0	68.0	68.0	68.0	71.0
Comp. Strength					
H_2O, %	50.4	49.0	48.0	47.0	51.4
Flow, %	114	106	113	106	108
1-Day, psi	1460	1400	1200	1860	2090
3-Day, psi	2930	2870	2700	3280	4420
7-Day, psi	4460	3980	4370	4560	6290
28-Day, psi	6200	5730	6410	6770	7870
63-Day, psi	6870	6320	7310	7850	8320

* This cement does not meet Type II specifications for moderate heat of hydration. Maximum percent of $C_3S + C_3A = 58$, and maximum percent of $C_3A = 8\%$ (according to ASTM C150).[2]

Table 3.2: Chemical and physical analyses of the cements used by Blick[104].

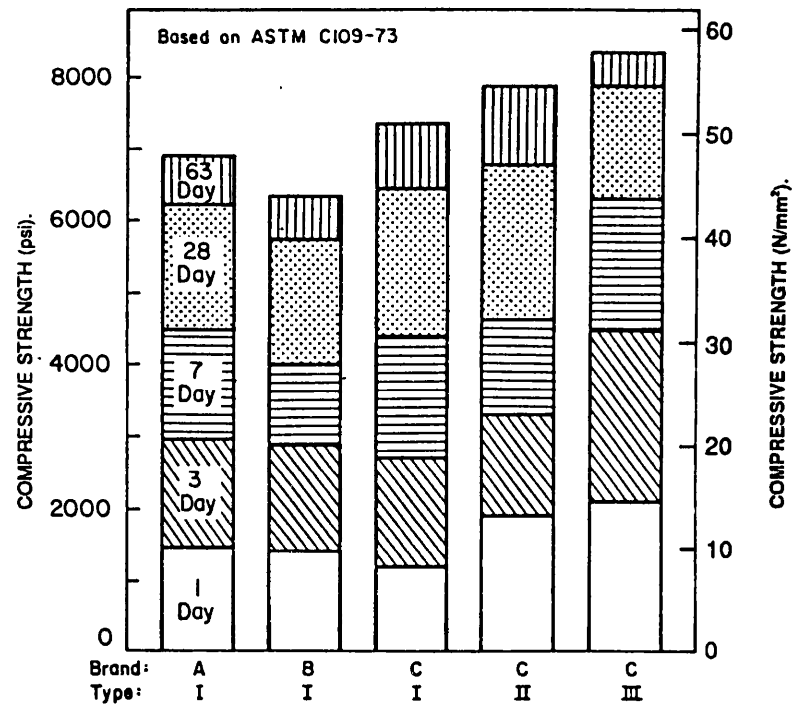


Figure 3.4: Mortar cube compressive strengths of the cements described in Table 3.2[104].

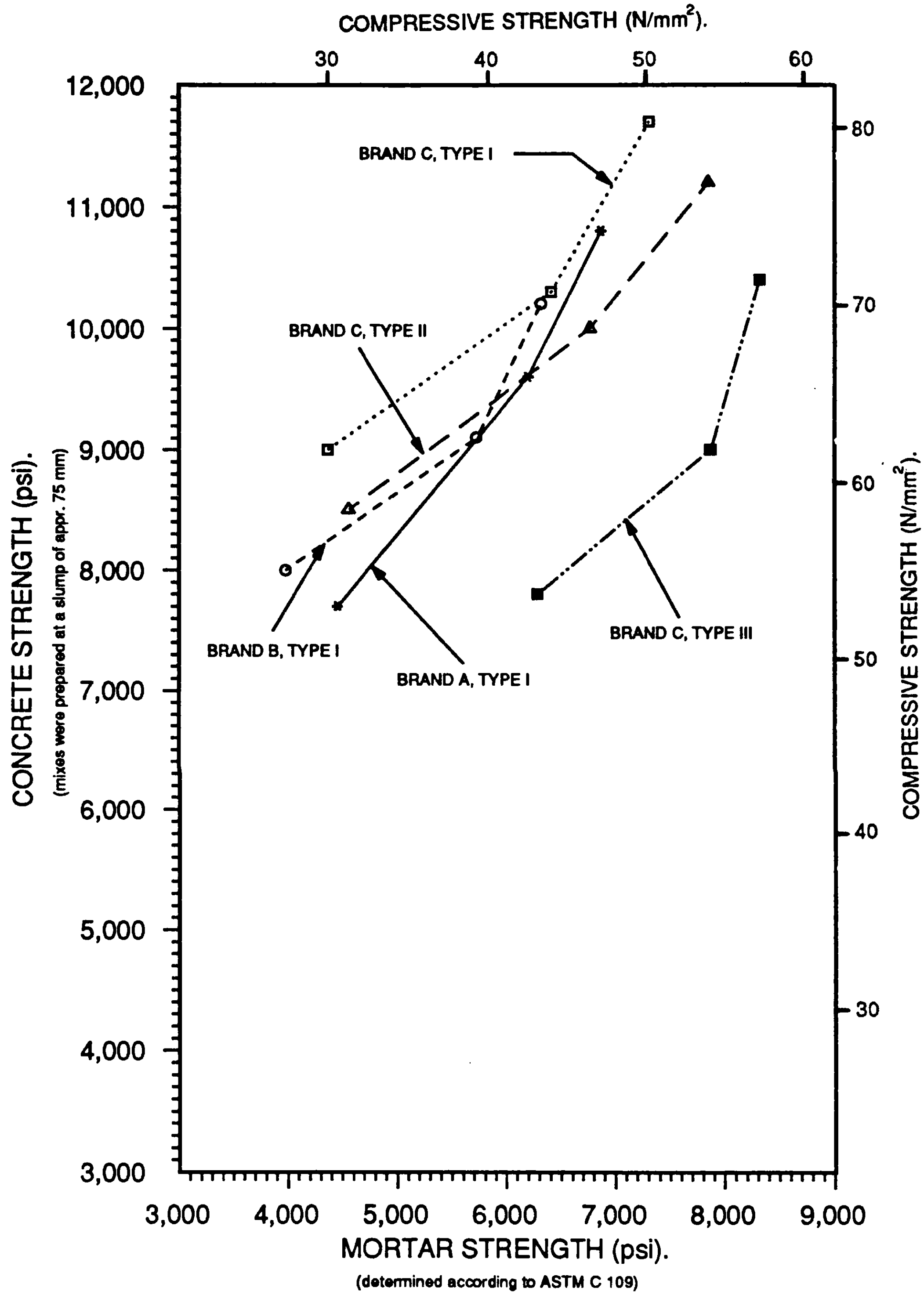


Figure 3.5: Concrete strength versus mortar strength for various brands and types of cement[104].

Note: The ratio of (water-binder ratio of concrete)/(water-binder ratio of mortar) was different for each brand and type of cement.

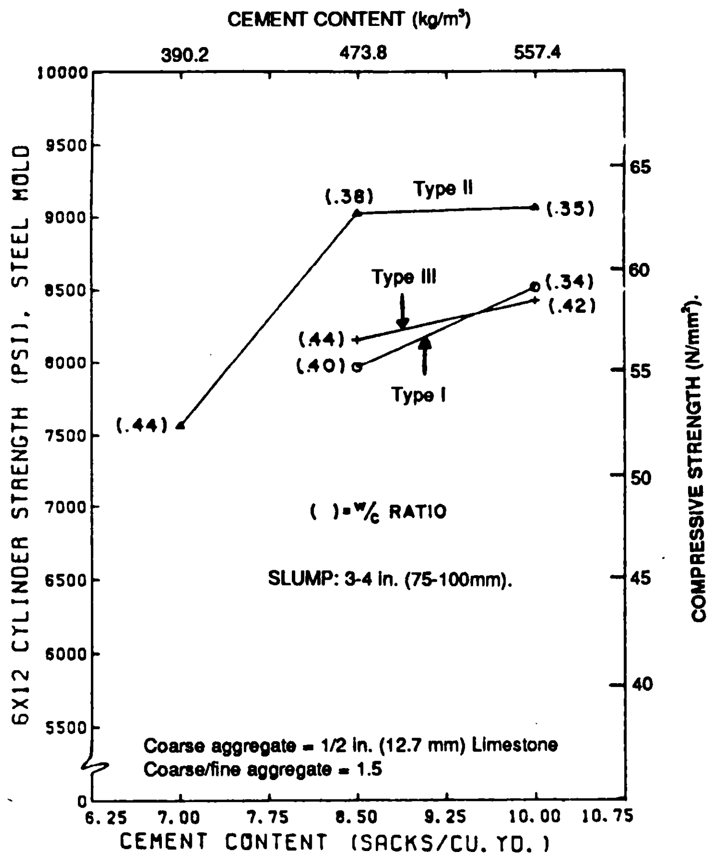


Figure 3.6: Effect of cement type and cement content on the 56-day compressive strength of non-superplasticised concrete[27].

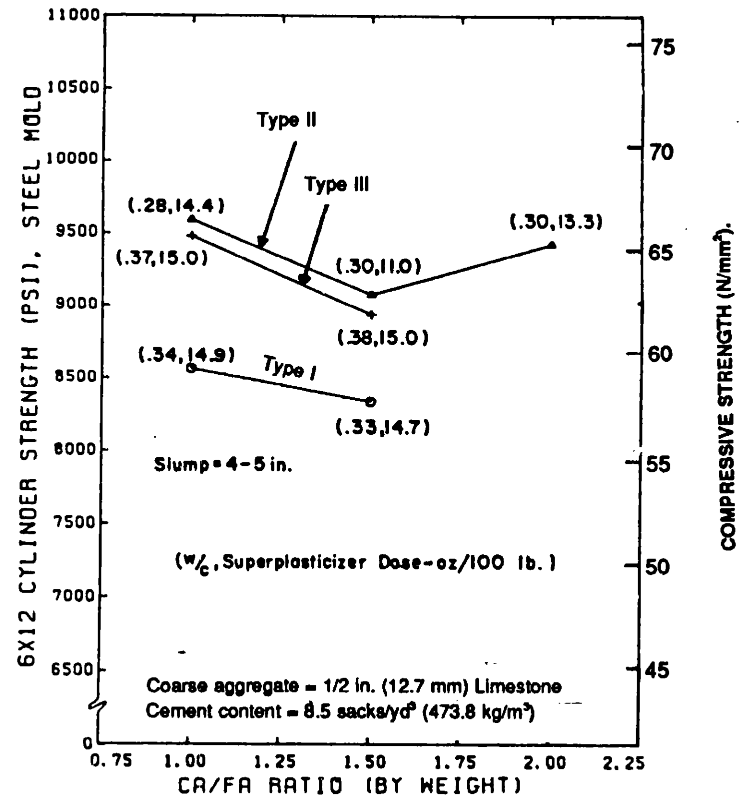


Figure 3.7: Effect of cement type and coarse/fine aggregate ratio (CA/FA) on the 28-day compressive strength for superplasticised concrete[27].

in comparison with Type I (ordinary Portland) cement, (Figure 3.8). Although the strengths have been compared on the basis of a range of slumps, between 40 and 60 mm, Burgess et al[76] did not state if this was retained by adjusting the water content or the dosage of the chemical admixture. With the materials tested a change in cement type was generally as beneficial to 7 and 28-day strengths as a change from gravel to limestone coarse aggregate. At 90-days with the three highest cement contents there was little strength advantage to the limestone concrete with a change to Type III (rapid hardening Portland) cement.

It may therefore be concluded that, in general, high strength concrete can be produced with any type of cement when a compatible superplasticiser is added to the mix. Unless high initial strength is the objective, such as in prestressed concrete, there is no need to use a Type III (rapid hardening Portland) cement[1]. When the temperature rise is expected to be a problem, a Type II (moderate heat Portland) cement can be used, but there are other ways of reducing the temperature rise as will be discussed in Section 3.2.6.

After the selection of the cement is made, limits on the physical and chemical properties should be established and submitted to the cement producer for compliance[11]. Hester[109] stated that if the tricalcium silicate content varies by more than 4 per cent, the ignition loss by more than 0.5 per cent, or the fineness by more than 37.5 m^2/kg (Blaine), then problems in maintaining a uniform high strength may result. A periodic sampling and testing programme of the cement being used should also be initiated to insure uniformity of the product and conformance to the modified limits or specifications[11].

3.2.2 Water.

The requirements for water quality for high strength concrete are no more stringent than those for conventional concrete[1]. The usual specification is for the water to be of potable quality. Aitcin[102] advocated the use of the coldest water possible in order to limit the workability loss and temperature of concrete during setting.

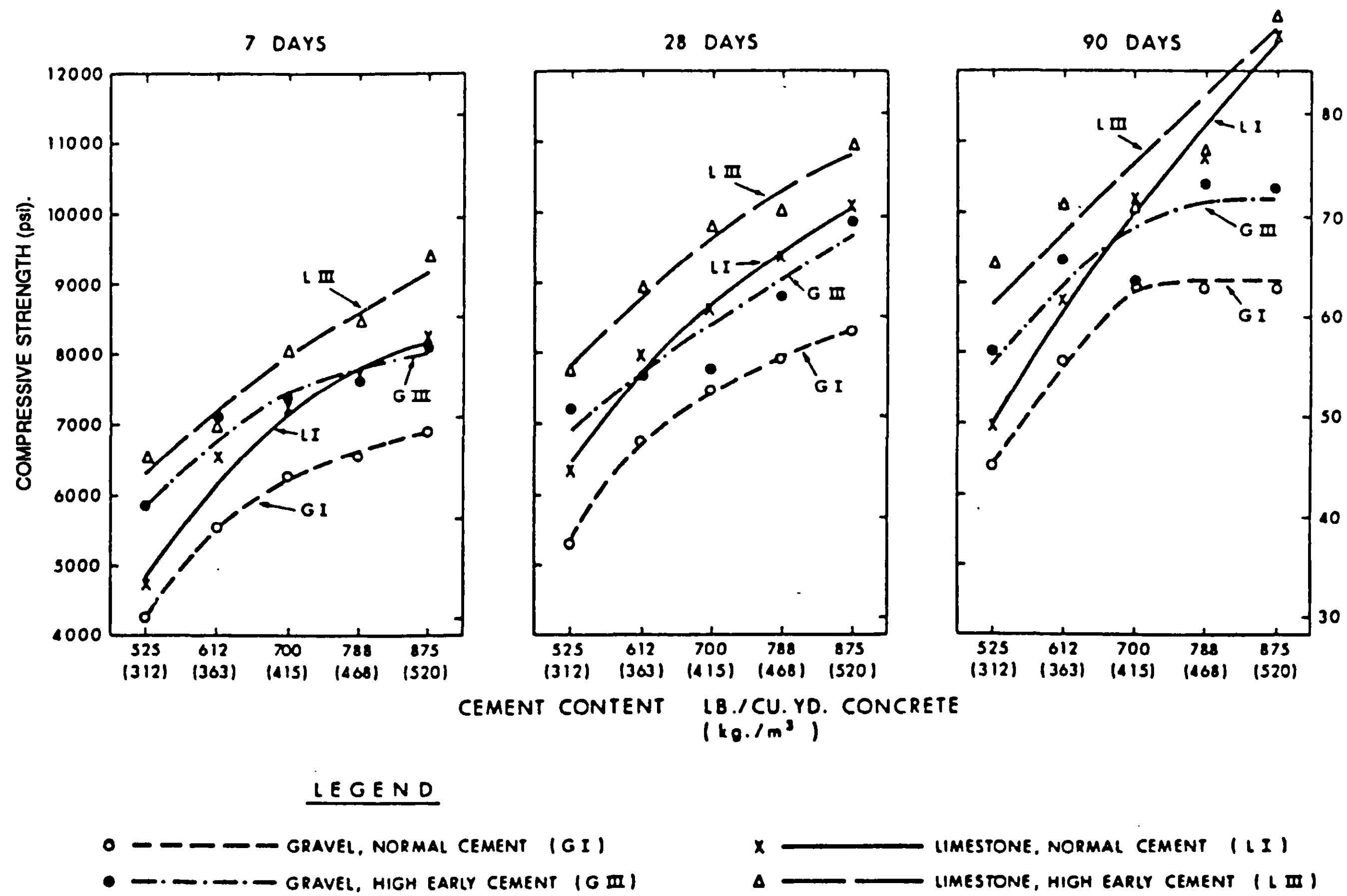


Figure 3.8: Effect of cement type and content on the compressive strength of concrete at various ages[76].

3.2.3 Chemical admixtures.

Admixtures are widely used in the production of high strength concrete. Selection of type, brand, and dosage rate of admixtures should be based on their performance in combination with the other materials being considered or selected for use on the project[1].

(I) Air-entraining admixtures.

The use of air entrainment has been recommended to enhance durability when high strength concrete is subjected to freezing and thawing while wet[1]. According to Philleo[110]:

> "Wherever high strength concrete is used, the question arises as to the need for entrained air."

Hester[109] reported that air voids reduce the strength of the concrete by 3 to 5 per cent for each 1 per cent increase in air over that normally entrapped, although data presented by Okada et al[111], (Figure 3.9), suggest that the strength reduction may not always be so high. Hester[109] also reported that while in mixes with low cement content, air entrainment permits a significant water reduction and provides improved workability, the workability is changed little by air entrainment in mixes with high cement content, (greater than 360 kg/m^3), and only a slight reduction in water-cement ratio may be achieved. Although air entrainment often necessitates an increased amount of cement for a given strength, the increased cement content alone may not overcome the full loss in attainable strength because the gain per kg of cement added, i.e. the cement efficiency, will progressively decline. Okada et al[111] presented the very positive observation, based on their results shown in Figure 3.9, that a volume of air decreases strength only half as much as an equal volume of water. Thus, entrained air may be compensated for by decreasing the water content through the use of a superplasticiser.

Air-entraining admixtures have not generally been used in North America because of the

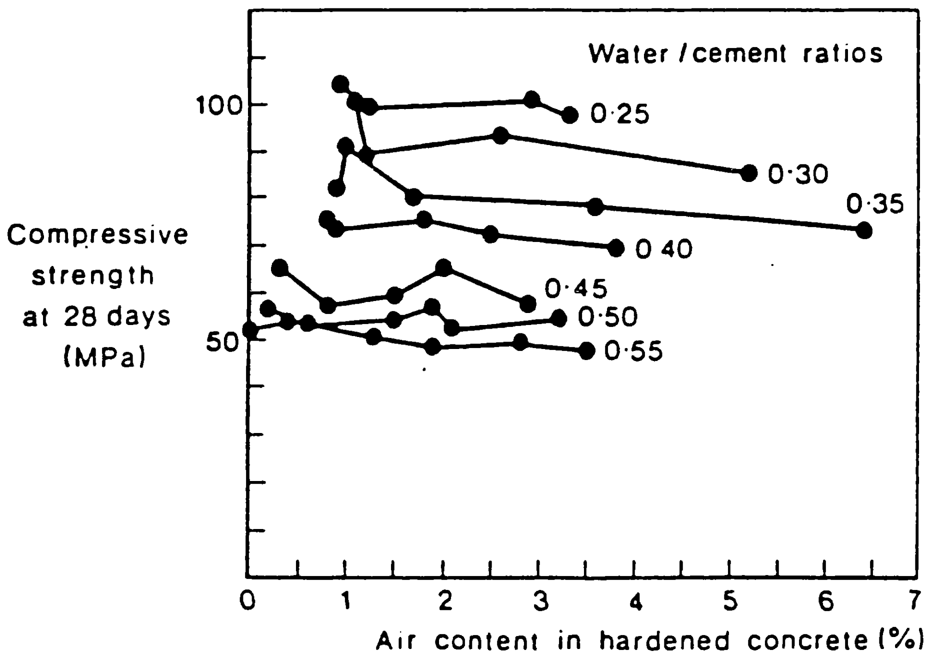


Figure 3.9: Effect of entrained air on the compressive strength[111].

accompanying strength loss and because many types of application, such as caissons, interior columns, and shear walls, will not normally require air-entrained concrete. However, air entrainment must be considered for concretes used in highway structures and offshore oil platforms because of their exposure. The question of whether high strength concrete needs a proper air entrained pore system to be frost resistant continues to be a relevant one with conflicting reports in the literature. The conflict concerns laboratory test results and not recorded field performance, of which there is little(2). The laboratory tests that have most commonly been employed to rate frost resistance are:

(1) ASTM C 666(112) which records volume degradation (internal microcracking) due to fast freeze-thaw cycles in fresh water,

and, (2) ASTM C 672(113) which records surface scaling after slower freeze-thaw cycles with the surface covered by a salt solution.

The British Standard BS 5075: Part 2: 1982(114) also describes freezing in water. However, most of the research on high strength concrete has been carried out using the ASTM procedures.

Philleo's(110) conclusions from the literature review of tests carried out in accordance with ASTM C 666 are:

> "When specimens are tested in strict compliance with ASTM C 666, including curing conditions, the preponderance of evidence is that the concrete needs to be air-entrained with a reasonable spacing factor. Non-air-entrained concretes which have proven durable in laboratory testing have benefitted either from a longer period of hydration or a period of drying before being tested. Available data confirm the need for a spacing factor of about 0.2 mm for low and medium strength concretes in order to satisfy the requirements of ASTM C 666, but they suggest that below a water-cement of 0.50 the required spacing factor is a function of water-cement ratio. Thus, the required spacing factor becomes larger as the water-cement ratio decreases. Concrete containing CSF appears capable of developing a pore structure, an absence of freezable water, and an immunity from resaturation from the environment which renders it immune

to freezing and thawing, but it cannot do so in 14-days. It is also apparent that at early ages high levels of cement replacement by CSF are incompatible with durability. This may simply be a time problem in that the greater time required for the reaction of the larger amounts of CSF may merely delay the age at which durability is achieved. ASTM C 666 is a good test for judging the inherent frost resistance under severe conditions of a specimen of concrete. The ambient conditions in the test are more severe than in most field exposures. It is an extremely severe test because of the young age at test, the lack of a drying period, and the very rapid cooling cycle."

However, the overriding problem in practice appears to be salt scaling(2). Whiting(115) used both ASTM C 666(112) and ASTM C 672(113) on PFA concretes that were subjected to both moist and air cures. His findings can be summarised as follows:

"All moist cured, non-air-entrained concretes performed poorly, exhibiting rapid deterioration irrespective of strength level. Entrained air contents, measured in the fresh concrete, of 3 to 4 per cent were found to be necessary in order to assure adequate durability when concretes were subjected to freezing and thawing in water. However, moist-cured, air-entrained, high strength concretes, prepared at 8,000 and 10,000 psi (55.2 and 69.0 N/mm^2), while performing satisfactorily with respect to freezing and thawing in water, were less resistant to applications of deicing agents than were air-entrained concretes prepared at the lower strength level. This was true even with air contents between 7 and 8 per cent in the fresh concrete."

Similar conclusions were reached by Pigeon et al(116), who investigated OPC as well as CSF concretes:

"Very low spacing factors offer no special advantage; normal spacing factors, i.e. below 0.2 mm provide the same protection against scaling as very low spacing factors. Contrary to internal cracking due to freeze-thaw cycles, scaling can never be completely prevented, i.e. the critical spacing factor concept cannot apply to scaling. Critical spacing factors derived from freeze-thaw cycle tests, their values generally range between 0.2 mm and 0.6 mm, are higher than those normally required for good scaling resistance."

The effect of CSF was reported as follows:

"Concretes containing small amounts of CSF can have a fair resistance to scaling caused by deicer salts, as long as they have a correct air void spacing factor for the water-binder ratio used, and are adequately cured. These concretes are somewhat more susceptible to scaling than normal concretes, but this is not very significant if the CSF content is not too high, i.e. an acceptable limit would be of the order of 8 to 10 per cent of the cement by mass. It must also be remembered that ASTM C 672 tests are quite severe, probably more so than most natural conditions of exposure."

The FIP/CEB report[2] also concluded that frost resistance of high strength concrete is a question of the relevance of the test procedures, and advocated that the greatest present need is to relate field performance to laboratory test methods.

(II) Retarders.

The rate of workability loss is increased and the setting time is decreased with the use of a high quantity of cement. According to Neville[117], setting, i.e. the change from a fluid to a rigid state, is caused by a selective hydration of cement compounds; the two first to react are C_3A and C_3S. The flash setting properties of the former are prevented by the addition of gypsum ($CaSO_4.2H_2O$) to the cement clinker. Gypsum and C_3A react to from first an insoluble calcium sulphoaluminate ($3CaO.Al_2O_3.\ 3CaSO_4.31H_2O$), which, as mentioned on page 46, is responsible for the workability loss of concrete. Eventually a tricalcium aluminate hydrate is formed, although this is preceded by a metastable

$3CaO.Al_2O_3.CaSO_4.12H_2O$, produced at the expense of the original high-sulphate calcium sulphoaluminate. But the addition of gypsum delays the formation of calcium aluminate hydrate, and it is thus C_3S that sets first. Since high strength concretes will in general have higher cement contents, and therefore higher C_3S contents, than normal commercial concretes, a retarder is frequently beneficial in controlling early hydration.

Aitcin et al[118] suggested that if the delivery time plus the placing time exceed half an hour, a retarder should be used. Further, structural design frequently requires heavy reinforcing steel and complicated forming with attendant difficult placement of the concrete. A retarder can control the rate of hardening in the forms to eliminate cold joints and provide more flexibility in placement schedules[1].

(III) Water-reducers.

A water-reducer can be defined as an admixture that reduces the amount of mixing water of concrete for a given workability. Usually, according to standards the reduction of mixing water by the use of these admixtures must be at least 5 per cent. However, commercial water-reducers can reduce mixing water up to 10-15 per cent. The use of a water-reducer in high strength concrete becomes necessary to efficiently use all cementitious materials and to maintain the lowest practical water-cement ratio[11]. The amount of water reduction is dependent upon the original water content, and the dosage and type of the water-reducer.

There are a number of formulations described in patents concerning water-reducers. Usually the main components are water soluble organic compounds, which can be divided into four groups. The first one contains Ca, Na or NH_4 salts of lignosulphonic acids. The second group contains the hydroxycarboxylic acids generally as Na, NH_4, or triethanolamine salts. Carbohydrates form the third group, and the last group comprises other compounds. All the water reducing admixtures are generally offered by the suppliers as aqueous solutions with specific gravities in the range of 1.10 to 1.30.

When Portland cement is mixed with water, solid particles tend to flocculate, because of a lack of mutual electrostatic repulsion of particles. In the presence of a water-reducer the flocs break up almost to individual particles. Most of the mechanisms that have been suggested to explain the fluidifying effect of water-reducers are based on this dispersion. Collepardi[(119)] collated these mechanisms as follows:

a. *reduction of interfacial tension.* According to Prior and Adams[(120)], a dispersed system is thermodynamically unstable compared to that in a flocculated state, where the liquid-solid interface and therefore the interfacial tension is reduced. Dispersion is facilitated by the adsorbed water-reducer molecules making the transition from the solid phase to the aqueous phase less abrupt.

b. *multilayer adsorption of organic molecules.* The presence of an adsorbed layer of thickness corresponding to several tens of molecular layers[(121)] would change the interparticle interaction energy. Banfill[(122)] believes that such a multilayer adsorption would give steric stabilization a role at least as important for the dispersing action, as for instance, the change in zeta potential.

c. *increase in electrokinetic potential.* Electrophoretic measurements of water-cement suspensions indicate that cement particles do not migrate in an electrical field, whereas cement particles in a lignosulphonate solution move towards the anode, demonstrating that a negative charge is present on solid particles[(123)]. Ernsberg and France[(123)] ascribed this negative charge to adsorbed lignosulphonate anions, and dispersion of cement to a mutual electrostatic repulsion among particles. Similar results have been obtained by Mielenz[(124)] with salts of hydroxy acids.

d. *protective adherent sheath of water molecules.* The negative charge on the cement surface orients the water dipoles forming a hydrated sheath which prevents cement particles from coalescing[(120)].

e. *release of water trapped among cement particle clumps.* According to Scripture[(125)], part of the mixing water, that otherwise would be trapped within the particle clusters, is made free to contribute to the fluidity of fresh concrete as a result of the dispersion.

Two other mechanisms, that are not based on the dispersion of cement particles, have also been suggested:

f. *retarding effect on cement hydration.* According to Massazza and Costa[126] the slower rate of ettringite formation caused by the presence of lignosulphonate is responsible *per se* for the reduction in water demand.

g. *change in morphology of hydrated cement.* A decrease in the interlocking effect of the ettringite bridges connecting solid particles has been observed in the presence of lignosulphonate. This may be caused by a reduction in the crystal size of ettringite with the resulting improvement in the rheological behaviour[126].

As mentioned earlier, normal water-reducers are capable of reducing water requirements by about 10-15 per cent. Further reductions can be obtained at higher dosages but this may result in undesirable effects on setting, air content, bleeding, segregation and hardening characteristics of concrete[127]. In 1970's, the advent of high range water reducing admixtures, popularly known as superplasticisers, provided a solution to this problem[46].

Superplasticisers belong to a new class of water-reducers chemically different from the normal water-reducers and capable of reducing water contents by about 30 per cent. They are broadly classified into four groups, i.e. sulphonated melamine formaldehyde condensate, sulphonated naphthalene formaldehyde condensate, modified lignosulphonates, and others including sulphonic-acid esters, carbohydrate esters, etc. Variations exist in each of these classes and some formulations may contain a second ingredient. They all however have a high molecular weight, and they are anionic surfactants with a large number of polar groups in the hydrocarbon chain. Normal water-reducers, such as lignosulphonates, exhibit a high degree of cross-linkage and formation of spherical microgel flocs when used in a much larger than the recommended dosage, (Figure 3.10).

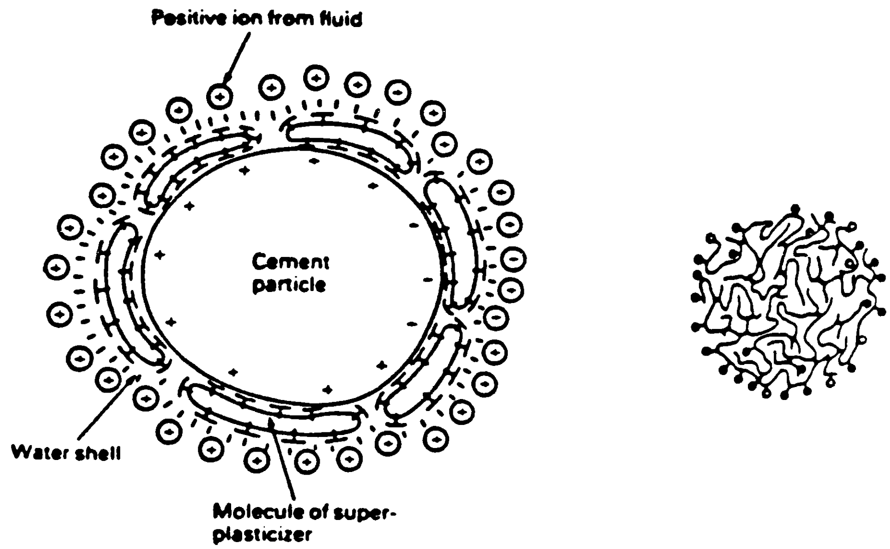


Figure 3.10: Effectiveness of linear polymers in coating cement particles (left), compared to cross-linked polymers present in lignosulphonates (right)[46].

On the other hand, the linear molecules of a superplasticiser do not form microgel flocs and are therefore much better dispersants of Portland cement particles. Using a large dosage of a superplasticiser, e.g., up to 2 per cent solids by weight of Portland cement, high-slump concrete mixtures (200 to 250 mm) with very low water contents (water-cement ratios less than 0.30), can usually be made without experiencing excessive retardation. This is why, according to Mehta and Aitcin[46], the use of superplasticisers has become almost mandatory with high strength concrete mixtures. Under certain job conditions, when retardation of the setting time is deliberately desired, a blend of a superplasticiser and a water-reducer may be advantageously used, (c.f. Table 2.2).

From the aforementioned mechanisms of the fluidifying effect of water-reducers, the two that have extensively been investigated in relation to superplasticisers are:

(a) adsorption of organic molecules on cement,

and, (b) electrokinetic (zeta) potential of cement particles in solutions containing superplasticisers.

Figure 3.11 shows the adsorption characteristics of sulphonated melamine formaldehyde on cement, C_3A and C_3S in an aqueous medium[127]. Adsorption of superplasticiser on C_3A occurs in substantial amounts even within a few seconds, while C_3S adsorbs only a small amount in the first hour. Further adsorption after 5 hours is both due to increased dispersion and hydration. The amount adsorbed on cement varies with the length of exposure to solution. After immediate adsorption, it is almost nil up to about 4 to 5 hours, but after that it is continuous. Adsorption beyond 5 hours is caused by the hydrating C_3S component in cement[127].

Daimon and Roy[128] compared the results from adsorption and zeta potential measurements, (Figures 3.12 and 3.13), to the results of mortar flow tests, (Figure 3.14), carried out as described in ASTM C 109[107]. They concluded that the changes in mortar flow with increasing concentration of two commercial superplasticisers, both of the sulphonated melamine formaldehyde condensate type, coincided with the results of zeta potential measurements and adsorption experiments. Their adsorption isotherms also show that the quantity of superplasticiser adsorbed on the cement particle surface reaches a

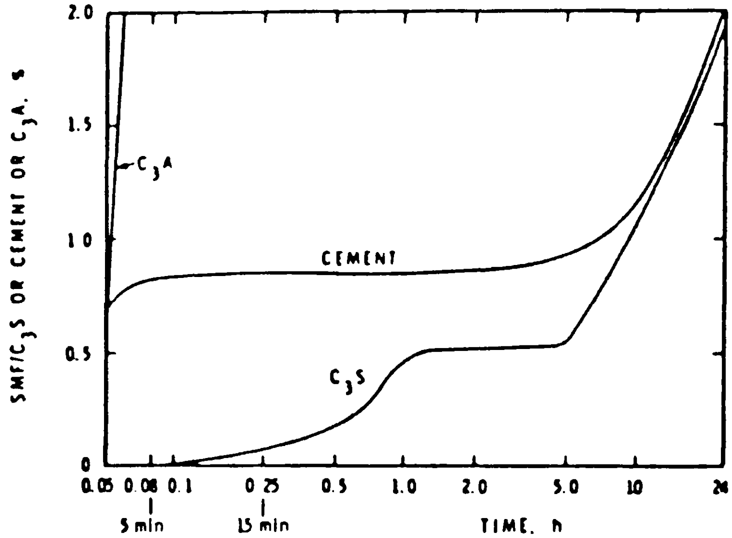


Figure 3.11: Adsorption of sulphonated melamine formaldehyde on cement compounds during hydration[132].

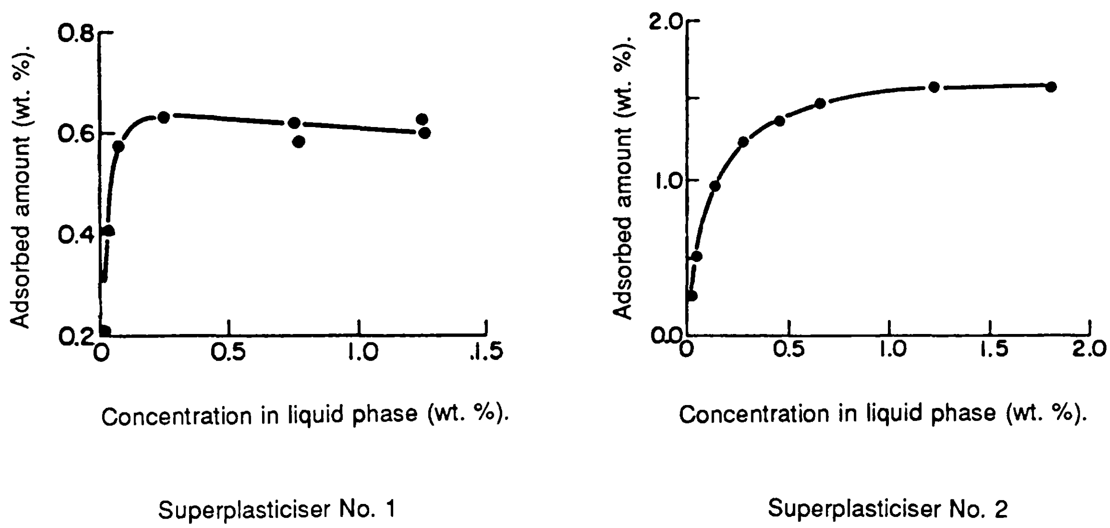
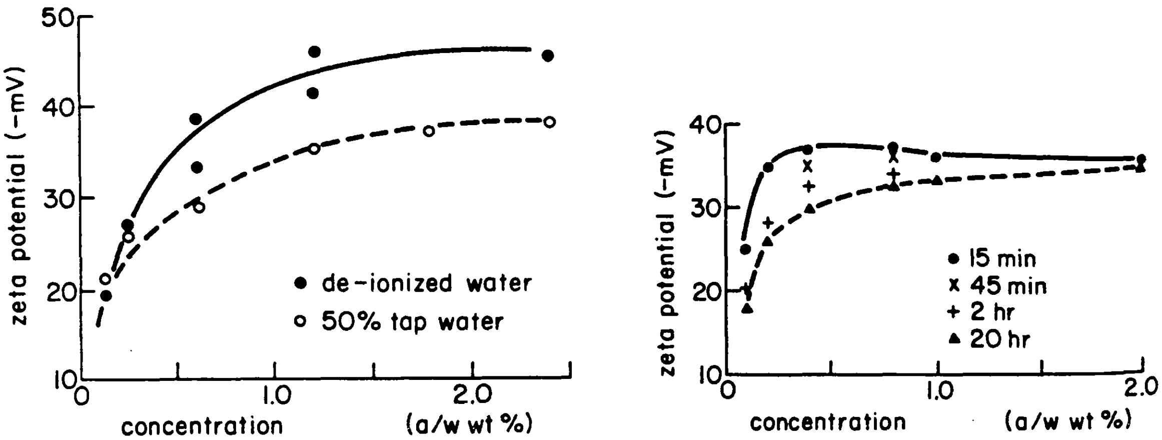


Figure 3.12: Adsorption isotherms of two superplasticisers on ordinary Portland cement[128].



Superplasticiser No. 1 Superplasticiser No. 2

Figure 3.13: Change in zeta potential of cement with increasing concentration of two superplasticisers(128).

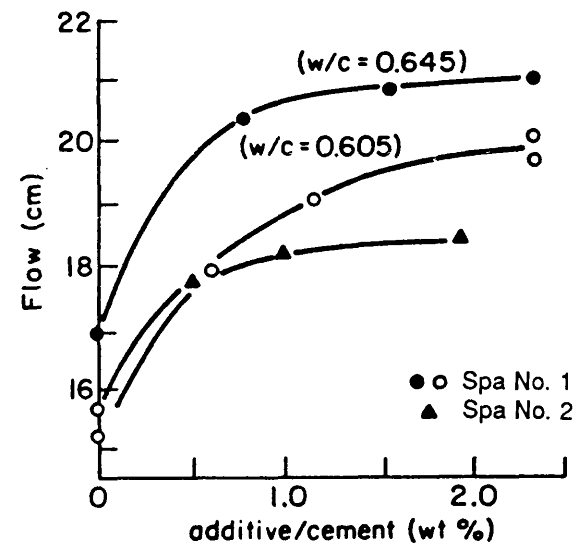
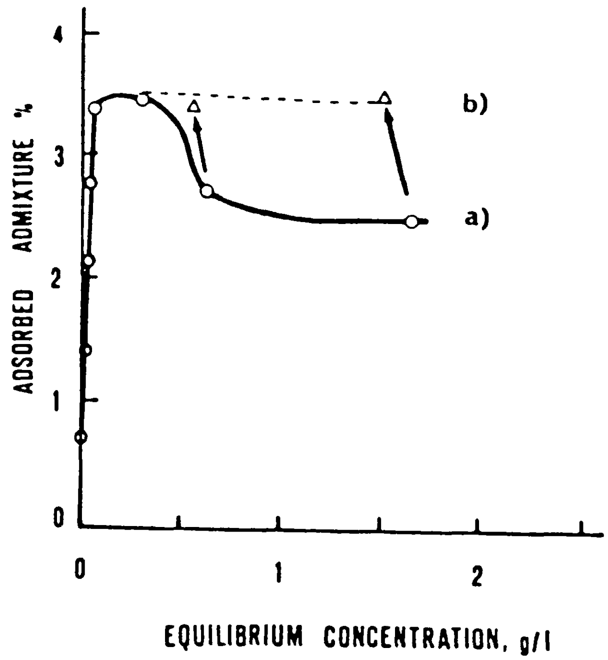


Figure 3.14: Change in mortar flow with increasing dosage of two superplasticisers(128).

ceiling value, i.e. saturation of the surfaces occurs. Massazza et al's[126] results of the adsorption of sulphonated melamine formaldehyde on calcium aluminate monosulphate hydrate also showed a saturation value, (Figure 3.15). The presence of a maximum in the adsorption isotherm beyond which the adsorption decreases has been attributed to the formation of associations (micelles) among the isolated molecules (monomers). The equilibrium occurring between these associations and the monomers is conditioned by the attainment of given critical concentrations. In solutions containing different molecular species (in the present work where polymeric molecules of different molecular weight are present) these equilibria become very complex and modify the ratios among the different monomers. It is considered that each of these species has a different surface adsorbability, and hence beyond certain concentrations, the adsorption decreases and may even rise again. According to this interpretation the presence of the maximum depends on the properties of the adsorption solution and not of the substrate. This was confirmed by modifying the modes by which the solid substrate and the adsorbed admixture were made to react; when the admixture was added in small successive doses, properly spaced in order to allow the attainment of the equilibrium, a simple curve, also shown in Figure 3.15, was obtained where the maximum value remained constant.

From the aforementioned, the question arises as to whether the saturation values determined represent mono- or multi-layer adsorption. Although Daimon and Roy did not comment on this, Banfill[122] used their results to show that the saturation point did not represent a mono- but a multi-layer, with on average 40 layers. However, Buil et al[130] showed, using the adsorption isotherm shown in Figure 3.16, that the weight of superplasticiser adsorbed on CSF particles is not far from a rough estimate of the weight of a mono-molecular film covering their surface. There is therefore no general agreement if mono- or multi-layer adsorption is occurring on cementitious particles.

Dahl and Meland[131] also found possible saturation points of the surfaces of PFA and CSF particles, (Figures 3.17 and 3.18), but did not comment whether this represents mono- or multi-layer adsorption. Also, although possible points of saturation have been found by many workers, none has commented on how the efficiency of superplasticisers will be affected at higher dosages than those required for saturation of the surfaces to occur.



(a) Admixture added to the solution: all at once
(b) Admixture added to the suspension: at small successive doses

Figure 3.15: The adsorption of melamine formaldehyde condensate on monosulphate in aqueous suspensions[126].

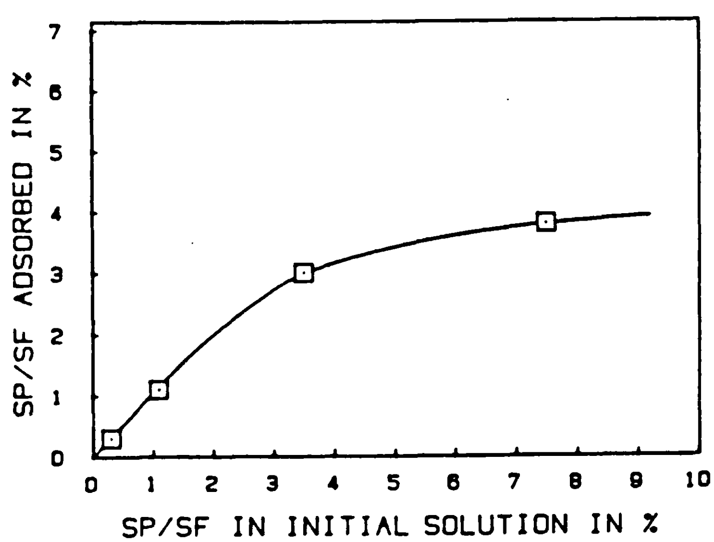


Figure 3.16: Adsorption isotherm of superplasticiser (SP) on condensed silica fume (SF)[130].

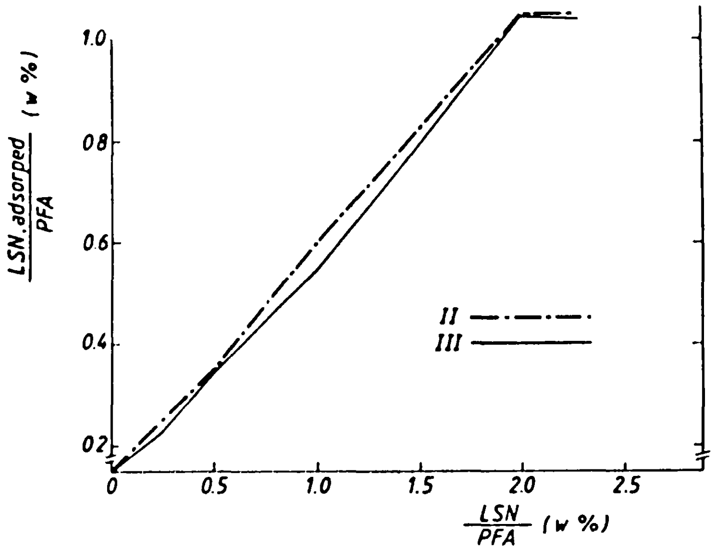


Figure 3.17: Adsorption isotherms of two brands of lignosulphonate (LSN) on pulverised fuel ash (PFA)[131].

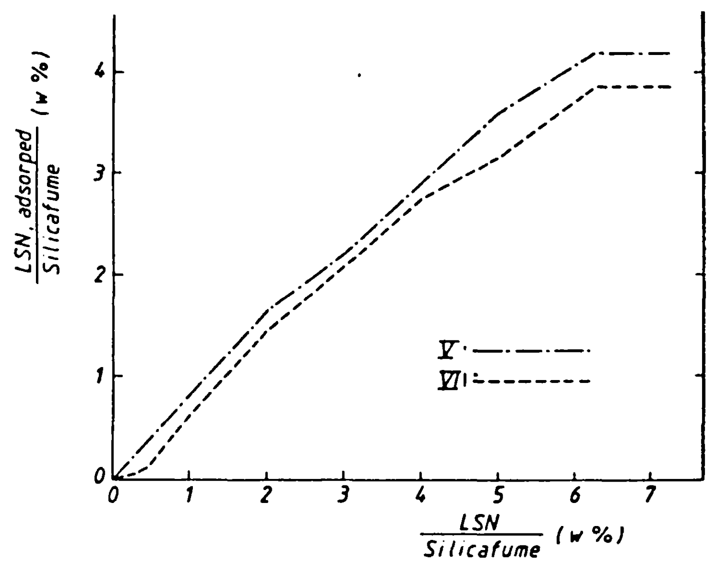


Figure 3.18: Adsorption isotherms of two brands of lignosulphonates on condensed silica fume (CSF)[131].

Two investigators[20,132] suggest that sulphonated naphthalene formaldehyde condensates are more efficient than those with a melamine base. Figure 3.19[20] indicates the respective unit water contents needed to obtain the required slumps. The ratios of the solids of superplasticiser to cement content are indicated on the abscissa. To obtain equal consistencies, it appears that more of the solids of the sulphonated melamine formaldehyde are needed compared with the naphthalene sulphonate formaldehyde condensates. When the dosage of superplasticiser is constant, more water will be required in proportion to the increase in cement content. However, it is usually advantageous to select the most effective combination of superplasticiser and cementitious binder on the basis of testing[32]. Penttala[132] observed that binders containing GGBFS seemed less sensitive to the choice of superplasticiser than those containing CSF or PFA. No particular superplasticiser was found to be consistently better than the others and the variability of the admixture/binder interaction was considered to be large enough to warrant selection on the basis of testing. Lacroix and Jaugey[31] have also suggested that the efficiency of superplasticisers can be variable, and Hanna et al[133] have cautioned that physical/chemical interactions between cements with high C_3A content and superplasticising admixtures containing large amounts of free sulphate cause rapid stiffening or slump loss, (c.f. page 46). Use of a cement with a low C_3A content has therefore been recommended to overcome the problem.

The time when a superplasticiser is added to a concrete mix can greatly influence its effectiveness[20,31,118]. Rixom and Dodson[134] stated that if the admixture is added just at the end of the mixing time of aggregates, cement and total gauging water, greater adsorption of the admixture on to the initial hydrates is obtained resulting in a higher workability. Alternatively a greater reduction in water-cement ratio is obtained for the same workability. The water-cement ratios they had investigated were higher than 0.51.

Collepardi[119] has also stated that an enhanced effect of the superplasticiser is obtained when it is added a few minutes after the mixing water has been added to concrete. This he explained as follows:

"Added along with the mixing water the superplasticiser is rigidly attached

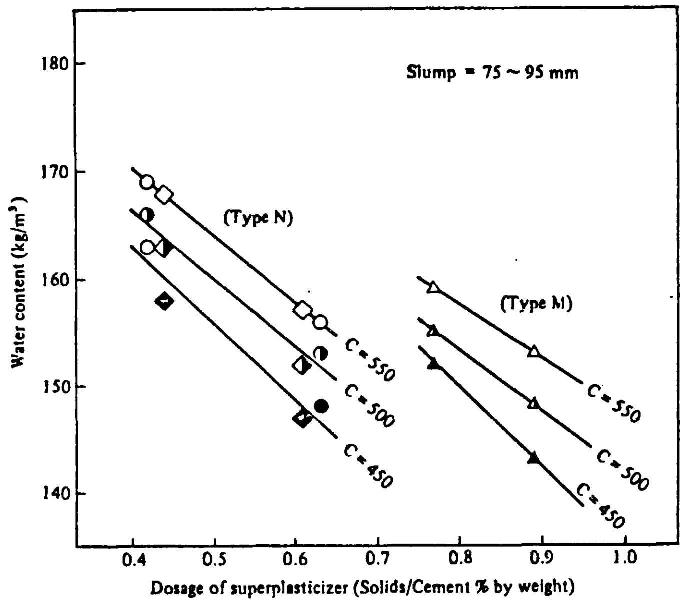


Figure 3.19: Relation between water content and required dosage of superplasticiser[20] for a slump of 75-95 mm.
Type N: Naphthalene based superplasticiser.
Type M: Melamine based superplasticiser.

> in substantial amounts by C_3A-gypsum mixtures leaving only small amounts for dispersion of the silicate phases, (c.f. Figure 3.11). By late addition the admixture is adsorbed to a lesser extent and there will be enough left in the solution to promote dispersion of the silicate phases and to lower the viscosity of the system."

Aitcin et al[118] found that adding the superplasticiser and retarder to the aggregates when filling the premixer was not efficient. At its arrival on the jobsite 45 minutes later, a second addition of superplasticiser was required in order to obtain a concrete slump of 80-100 mm lasting for 15 minutes, the required placing time. The mixing sequence was altered so that the superplasticiser and the retarding agent were directly added in the mixer. With this procedure it was not necessary to add any superplasticiser at the jobsite. They concluded that when the additives are added directly in the mixer they act directly on the cement grains (and CSF particles), whereas when they are added on the incoming aggregates a part of the additives is diverted to coat the fine and coarse aggregate particles.

Peterman and Carrasquillo[27] found that addition of the superplasticiser to dry cement resulted in segregation and a slimy appearance of the fresh concrete. Addition of more water to the mix restored workability after sufficient mixing. However, control over the water-cement ratio was lost, since more than the usual amount of mixing water was required for the desired slump. They concluded that the quality of fresh concrete was adversely affected because hydration was hindered greatly when the dry cement particles were coated by superplasticiser before they were combined with the water. They advocated that at least half of the superplasticiser dose should be added to the concrete with the last portion of mixing water and the remainder be added directly to the fresh concrete after mixing starts. With high strength concrete mixes, thorough mixing of all materials in all parts of the concrete mixer is particularly important.

Due to the high loss of workability with time after mixing of superplasticised high strength concrete (the admixture fluidifying action is limited to 30-45 minutes), Lacroix et Jaugey[31] advocated that their mode of introduction into the concrete mix is also of great importance on the retention of workability. The admixtures should not simply be mixed with concrete. The operation should be done in two phases: 1/3 to 2/3 of the total admixture content should be added just before the end of mixing diluted in some gallons of the total water, while the last 1/3 must be added in the mix after a 20 to 30 minutes time delay; concrete workability was then maintained for up to one hour.

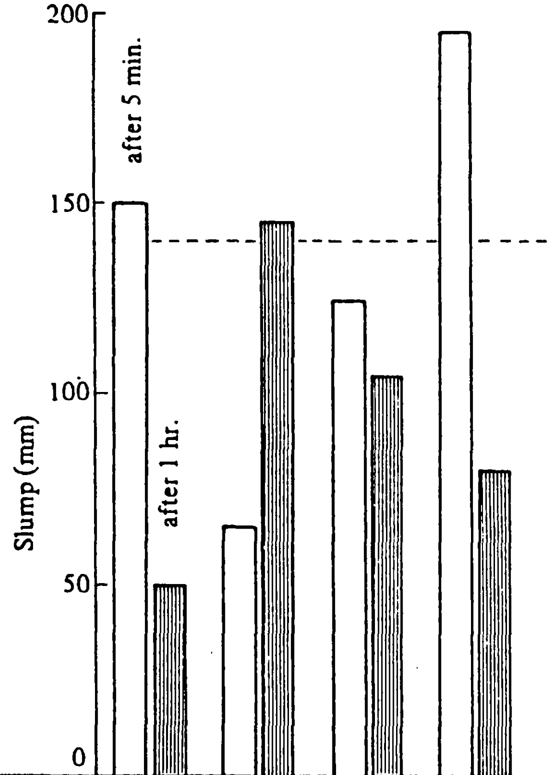
The effect of split addition of superplasticisers on the slump of concrete after an hour from the initial mixing was also investigated by Murata et al[20], (Figure 3.20). They advocated that only sufficient superplasticiser for a 50 mm slump (i.e. the slump at which uniform mixing can be done with easy discharge) should be added at the mixing plant. As much of the superplasticiser as possible is therefore added immediately before placement, reducing the problem of slump loss. This can also be seen from the results of Kishitani et al[135], shown in Figure 3.21. It must be noted, however, that the rate of slump loss of superplasticised concrete increased with longer delays in the addition of the superplasticiser.

3.2.4 Coarse aggregate.

The following factors have usually been considered when selecting a coarse aggregate for high strength concrete:

(i) Strength,
(ii) Particle shape and surface texture,
(iii) Maximum size,
(iv) Modulus of elasticity,
(v) Mineralogy,
and, (vi) Cleanness.

These factors are now discussed in turn:



addition to mixing water (%)	100	50	50	50
delayed addition after 3 min. (%)	0	10	30	50
readdition after 1 hr. (%)	0	40	20	0

Admixture: Type N
Total dosage: Cement wt. x 1.55%
W/C: 0.31
Cement: 520 kg/m^3

Figure 3.20: Effect of split addition of the superplasticiser[135].

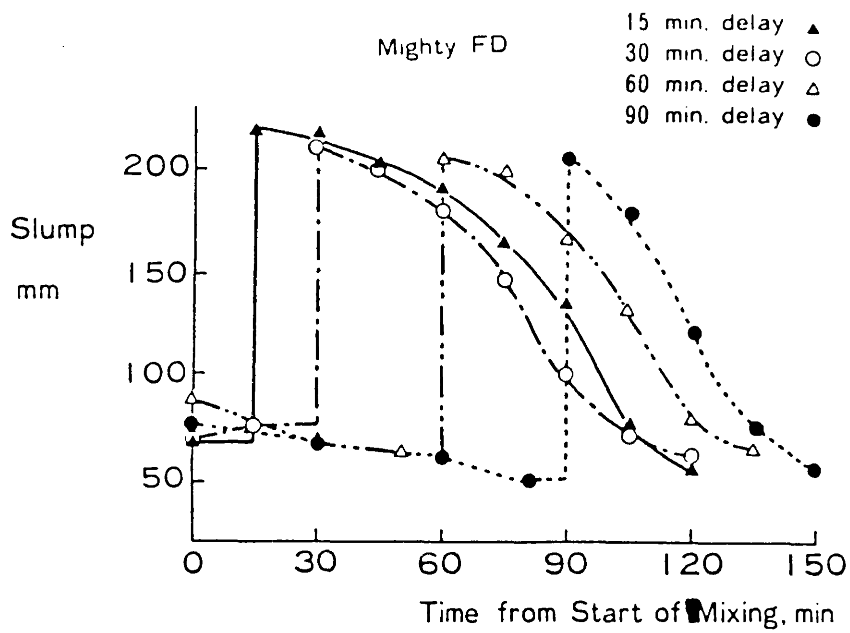


Figure 3.21: Variation of slump due to difference in timing of the addition of the superplasticiser (Mighty FD)[20].

(I) Strength.

Wittmann[136] stated that the strengths of aggregates are decisive for determining the ultimate load-bearing capacity of high strength concrete. For concretes with strengths of less than 5,000 psi (34.5 N/mm^2), the aggregate strength is generally greater than the mortar strength, and failures are therefore characterised by fractures through the mortar and in the transition zone between mortar and aggregate. The transition zone is the hydrated cement paste in the immediate vicinity of the aggregate that has significant microstructural differences from that of the cement paste at some distance away from it. In ordinary concrete, the transition zone is typically 0.05 to 0.1 mm wide and contains relatively large pores and large crystals of hydration products. Reduced water-cement ratio improves both the mortar strength and the transition zone, and subsequently the difference in strength between the aggregate and the mortar becomes an important parameter[44].

Nagataki[18] also advocated the necessity for careful selection of the aggregates for high strength concrete, and showed the dependence of the compressive strength of the concrete on the crushing value of the coarse aggregate, (Figure 3.22), determined according to BS 812: Part 3[137]. Tachibana et al[138] also showed a similar correlation, (Figure 3.23). However, the decrease in compressive strength due to the increase in the crushing value was comparatively small within the limits of their experiment. It is unfortunate that while Tachibana et al have specified the types of aggregates they used, Nagataki has not done so.

(II) Particle shape and surface texture.

The compressive strength of concrete does not only depend on the strength of its constituent materials, i.e. mortar and coarse aggregate, but also on the strength of the bond, or transition zone, between the two. Ways of improving this is by increasing the roughness of the coarse aggregate, reducing the water-cement ratio, or by other means, e.g., the use of CSF which will be discussed later.

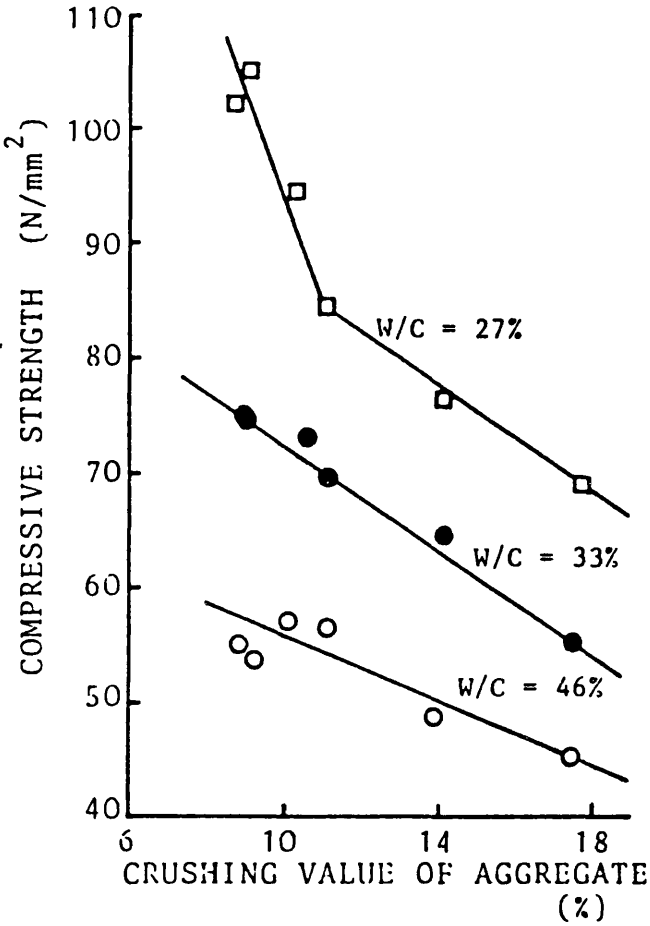


Figure 3.22: Nagataki's[18] relation between crushing value of coarse aggregate and the compressive strength of concrete. The types of aggregates used have not been specified.

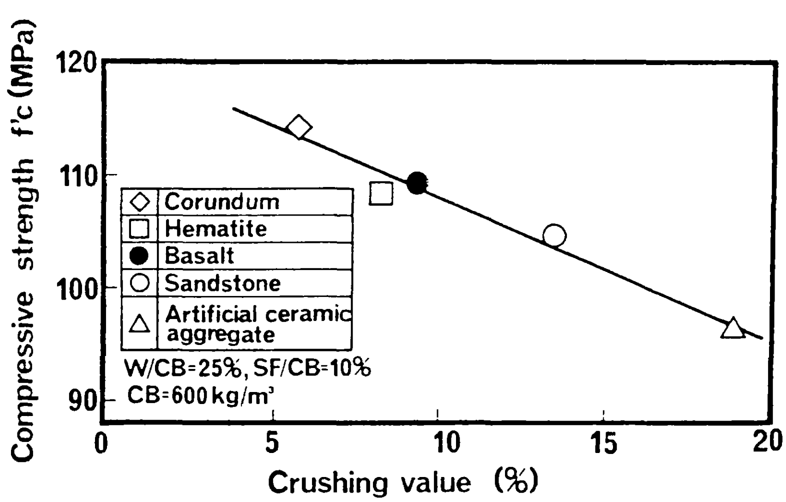


Figure 3.23: Tachibana et al's[138] relation between crushing value of coarse aggregate and the compressive strength of concrete.

It is not therefore surprising that the difference in surface texture and particle shape between gravel, or rounded aggregate, and crushed stone, has received considerable attention. Two studies[76,104] have shown that crushed stone produces higher strengths than rounded gravel. It has also been found that for the gravel aggregate a point is reached beyond which further increases in cement content produce no increase in the compressive strength of the concrete, (Figure 3.8)[76]. This apparently is not due to having fully developed the compressive strength of the concrete but to having reached the limit of the bonding potential of that cement-aggregate combination, this being different for different types of cement; ACI Committee 363[1] called this the "intrinsic aggregate strength".

The higher strengths obtained with crushed stone have been attributed to the greater mechanical bond which can develop with angular particles. According to Aitcin[102], the increased surface roughness creates fixing points for the hydrated cement that help prevent any movement of either material relative to the other. On the other hand, smooth, rounded coarse aggregates require much less mixing water to obtain a workable concrete. This raises the question of which is more important for concrete strength: the lower water-cement ratio possible when using gravel, or the stronger aggregate-mortar bond resulting from the use of crushed stone. Although, it has been concluded that strength gains from using crushed aggregates are more important[104], concrete with a mean 56-day cylinder strength of as high as 19,000 psi (131.0 N/mm^2) has been successfully produced using round glacial aggregates[30].

(III) Maximum aggregate size.

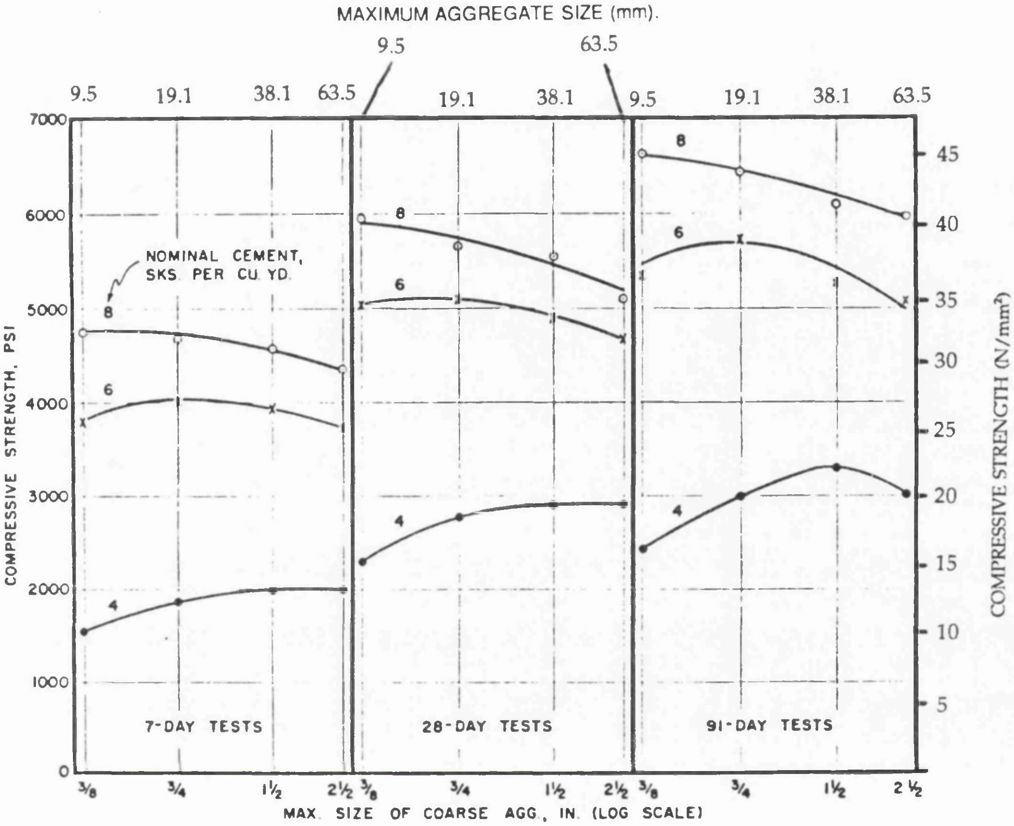
State-of-the-art reports[1,44,108,139] generally recommend that for optimum compressive strength with high cement contents and low water-cement ratios the maximum size of coarse aggregate should be kept to a minimum, at about 10 mm. They, however, seem to base their recommendation on research that was carried out between 1960 and 1975, despite this having been supplemented with more recent data. Since this is often thought to be an important factor, the data will now be presented and discussed in some detail.

Around 1960, it was realised that changes in maximum size of coarse aggregates had effects on strengths other than those produced simply by changes in water requirements, i.e. that concrete made with larger sizes of aggregates requires less mixing water for the same workability. Subsequent research indicated that different levels of the water-cement ratio versus strength relationship prevail not only for different cements and different types of aggregate, but also for different maximum sizes of the same aggregate. For example:

(i) Walker and Bloem[(140)] investigated the effect of maximum sizes of coarse aggregate ranging from 3/8 to 2-1/2 in. (9.5 to 63.5 mm), using three cement contents, from 4 to 8 sacks per yd^3 (223.0 to 445.9 kg/m^3) both with and without air entrainment. For the same cement content and slump a reduction in mixing water always accompanied an increase in aggregate size. This advantage was, however, found to be opposed by a reduction in strength, (Figure 3.24), to the extent that the increases in maximum size even caused strength reductions, predominantly in the concretes of higher cement contents.

(ii) The extensive investigation carried out by Cordon and Gillespie[(141)] (sixty-nine concrete mixes made with wide variations in water-cement ratio and maximum size of aggregates) allowed them to plot the strength efficiency, i.e. the unit strength in N/mm^2 obtained for each kg of cement used in a cubic metre, for different sizes of aggregates. These data, shown in Figure 3.25, indicate that for each strength level, maximum efficiency is obtained with a different maximum size of aggregate, and a different cement content. The smaller size aggregates provided the most efficient use of cement in the higher strength concretes.

(iii) Bloem and Gaynor[(142)] verified the above observations by showing that increasing the aggregate size from 3/4 in. (19.1 mm) to 1-1/2 in. (38.1 mm) produced an increase of about 2.5 N/mm^2 in compressive strength and about 0.35 N/mm^2 in flexure, (Figure 3.26). The average water reduction that was required to offset the detrimental effect of the larger aggregate size on strength was approximately 15 litres per m^3, (Figure 3.27).



Sacks/yd³	kg/m³
8	445.9
6	334.5
4	223.0

Figure 3.24: Effect of coarse aggregate size on the compressive strength of concrete[140].

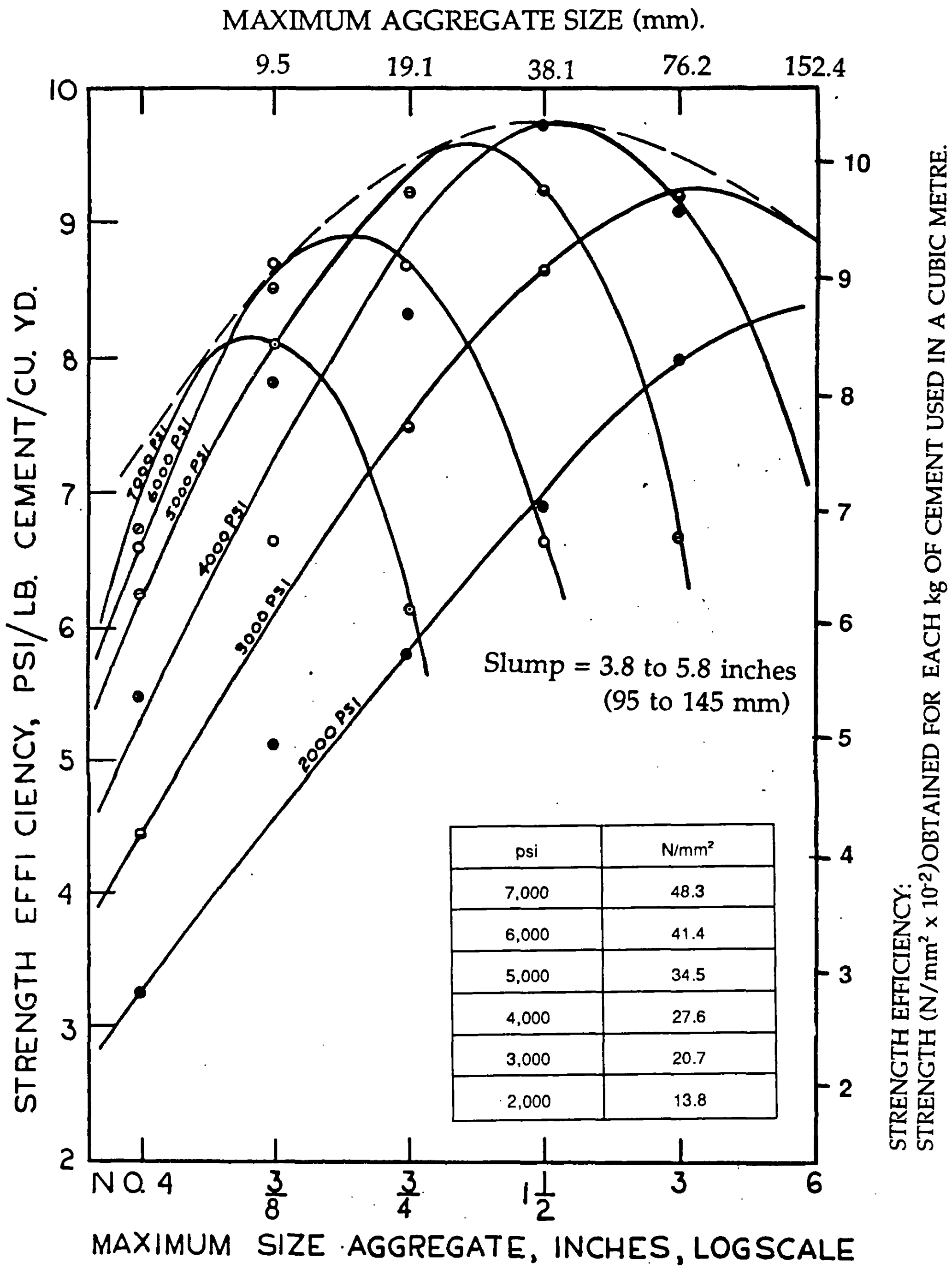


Figure 3.25: Maximum aggregate size for strength efficiency(141).

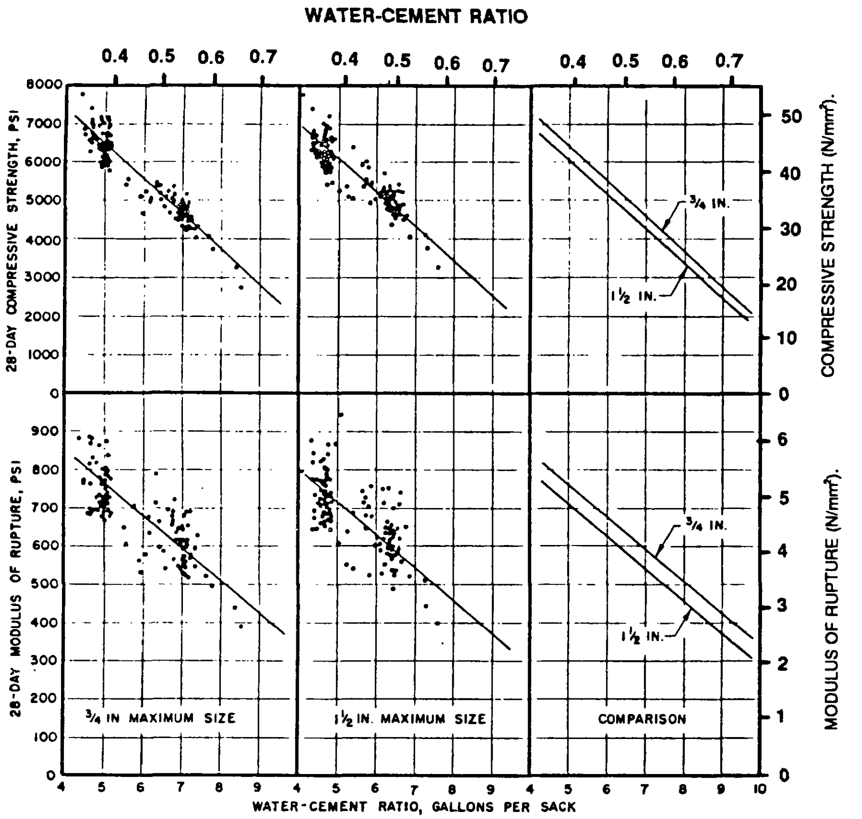


Figure 3.26: Compressive strength and modulus of rupture versus water-cement ratio(142).

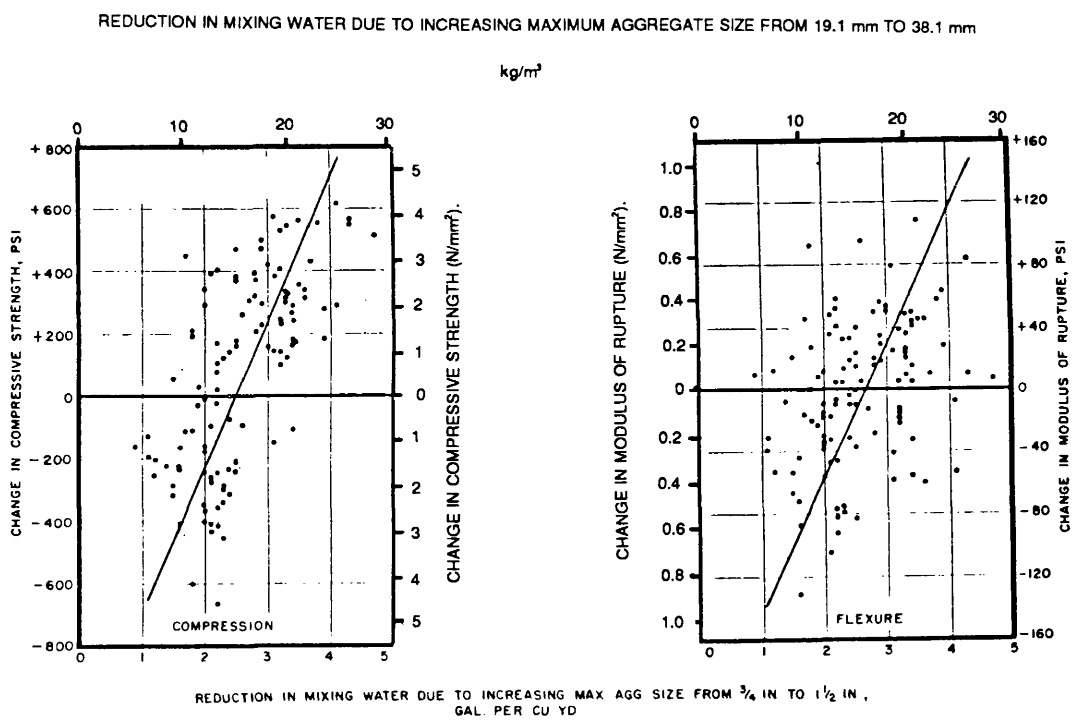


Figure 3.27: Reduction in mixing water required to counteract the effect of increasing the maximum aggregate size from 3/4 to 1-1/2 in. (19.1 to 38.1 mm)(142).

All of these results so far discussed were published between 1960 and 1963. More recently, indications of the effect of maximum aggregate size have come mainly from investigations of local materials carried out in order to select the materials and establish the mix proportions that were to be used for the construction of either experimental columns or particular projects. For this reason, these investigations were by no means as extensive as the ones carried out between 1960 and 1963. For example:

(i) From the investigation to determine the mix design of the 55 N/mm^2 concrete that was used for the Collins Place Project, Melbourne, Australia, Day[143] concluded that the interaction and compatibility of matrix and coarse aggregates (which might, loosely, be described as "bond") is a critical factor in obtaining the 55 N/mm^2 concrete strength. This factor can cause a reversal of normal aggregate preferences for both particle size and parent rock. A 1/2 in. (12.7 mm) maximum sized crushed vessicular basalt gave better results than either larger sizes of the same aggregate or alternative harder stronger rocks. Trial mixes suggested that 10 mm aggregate would have been even better. The (unproven) assumption was that the smaller size provided a better mechanical interlock and reduced the stress concentrations. The aggregate size finding was, however, regarded with some suspicion and by no means was it fully implemented.

(ii) Peterman and Carrasquillo[27] stated that a smaller maximum size of coarse aggregate is required for the production of high strength concrete than for the production of normal strength concrete when no chemical admixtures are used, (Figures 3.28 and 3.29). For a cement content of 7 sacks per yd^3 (390.2 kg/m^3) the compressive strength of concrete was controlled by the water-cement ratio in mixes containing no chemical or mineral admixtures. As a result, mixes made with 1 in. (25.4 mm) maximum aggregate size, which required the least mixing water for a given slump, produced the highest compressive strengths. For higher cement contents, however, using 1/2 in. (12.7 mm) maximum size coarse aggregate resulted in the highest 56-day compressive strengths, despite that the water-cement ratio of these mixes was not the lowest. Their results on superplasticised concrete, however, were limited and inconclusive, (Figures 3.30 and 3.31). Although the

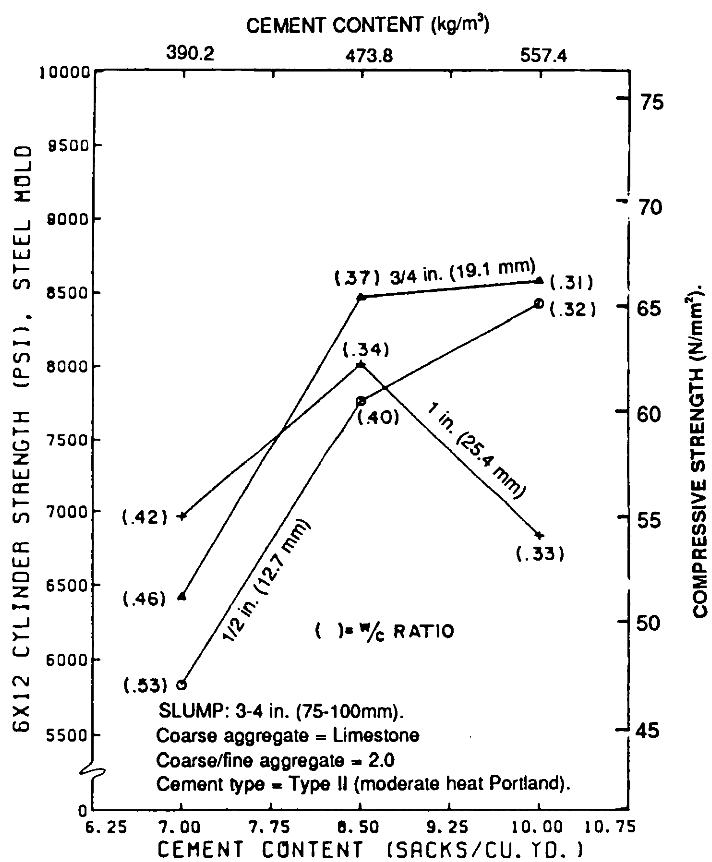


Figure 3.28: Effect of maximum aggregate size on the 28-day compressive strength of non-superplasticised concrete[27].

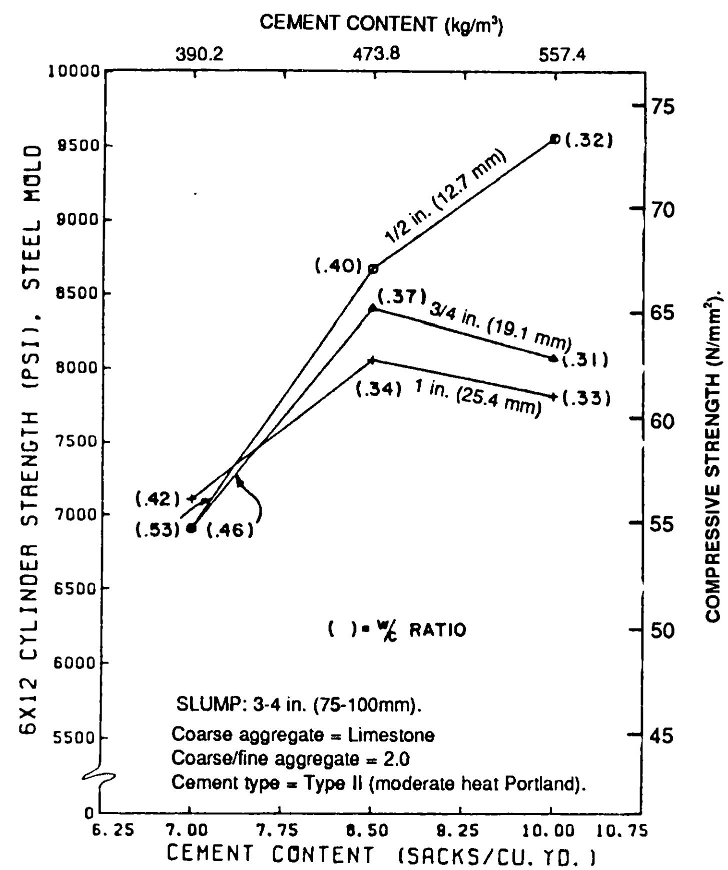


Figure 3.29: Effect of maximum aggregate size on the 56-day compressive strength of non-superplasticised concrete[27].

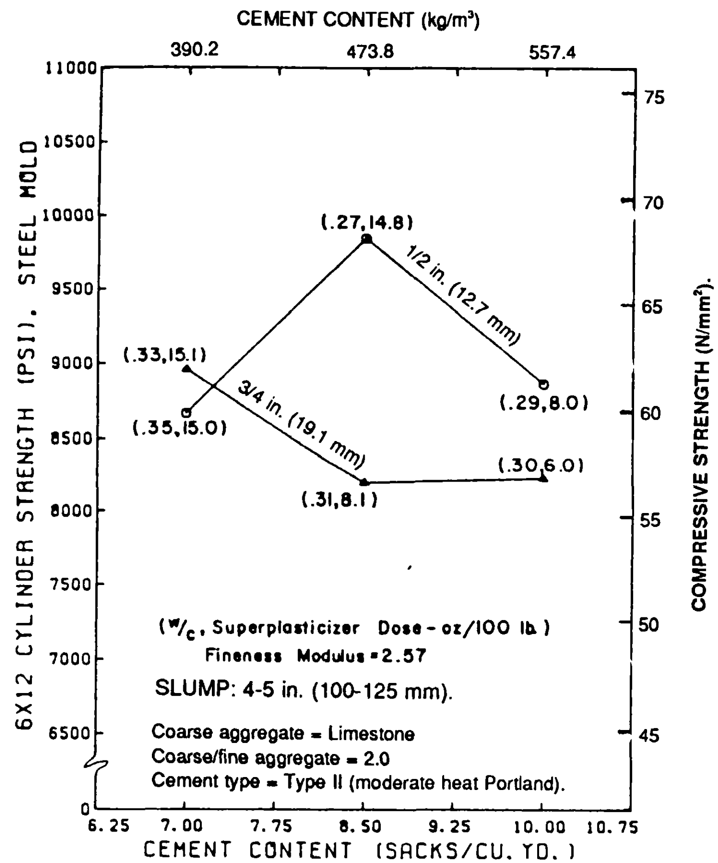


Figure 3.30: Effect of maximum aggregate size on the 28-day compressive strength of superplasticised concrete[27].

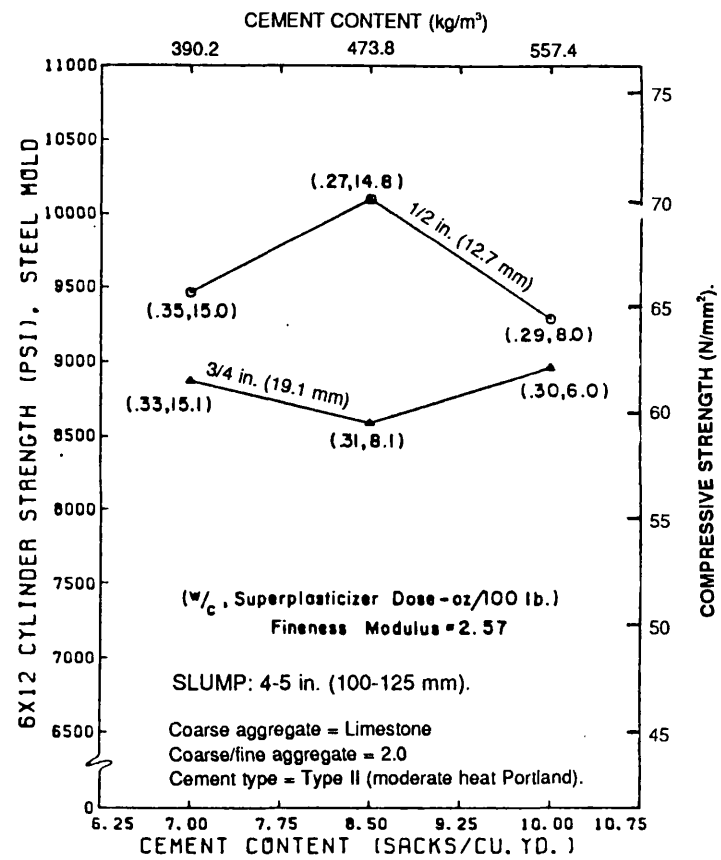


Figure 3.31: Effect of maximum aggregate size on the 56-day compressive strength of superplasticised concrete[27].

highest compressive strengths were achieved using 1/2 in. (12.7 mm) maximum aggregate size two out of the three mixes with 1/2 in. (12.7 mm) aggregate had lower water-cement ratios than the corresponding mixes with 3/4 in. (19.1 mm) aggregate.

(iii) Cook[59] also concluded that the smaller size of coarse aggregate produced the higher strength for non-superplasticised concrete with a given water-binder ratio, (Figure 3.32). The use of a superplasticiser, however, resulted in strength enhancements for both aggregates when the water-(cement + PFA) ratio dropped below 0.50. The concrete strength of the 1 in. (25.4 mm) coarse aggregate concrete with water-binder ratio of about 0.30 increased by 3,000 psi (20.7 N/mm^2) by the use of a superplasticiser resulting in the same strength as a 3/8 in. (9.5 mm) coarse aggregate of the same water-binder ratio, (Figure 3.33). He attributed this strength enhancement to the additional dispersion of cement particles resulting from the use of a superplasticiser. The principal concern of the engineer in the design of this structure was the stiffness of the concrete; a high modulus of elasticity was needed. Since, the larger the amount of coarse aggregate with a high elastic modulus in a concrete mixture will result in a higher modulus of elasticity of the concrete, the 1 in. (25.4 mm) coarse aggregate was chosen for this project.

(iv) Bedard and Aitcin[105] studied the influence of the maximum aggregate size on the compressive strength of very low water-binder ratio, (0.22 to 0.25), superplasticised concrete with 11 per cent cement replacement by condensed silica fume. Their results are summarised in Table 3.3. The small variation of compressive strength led them to conclude that the size of the coarse aggregate has only a small influence on the compressive strength.

The conclusions from the investigations carried out between 1960 and 1963 have been derived mainly for non-superplasticised Portland cement concrete, with "high strength concrete" referred to in these studies as having strength of hardly more than 50 N/mm^2. Also, Cordon and Gillespie's strength efficiency envelope[141], (Figure 3.25), has been derived from mixes with "well rounded river aggregate". It is therefore surprising that, for

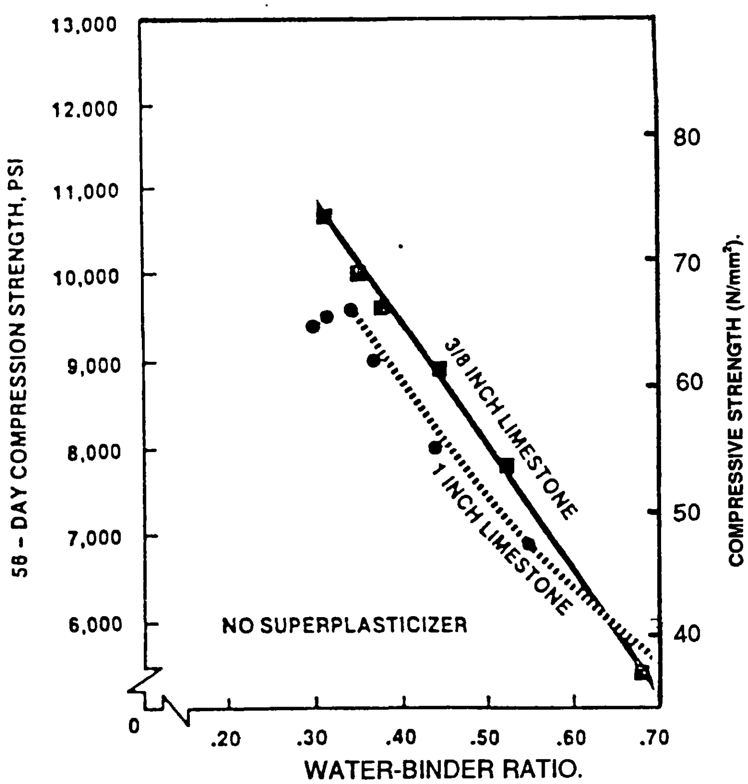


Figure 3.32: Compressive strength versus water-binder ratio for non-superplasticised concrete[59].

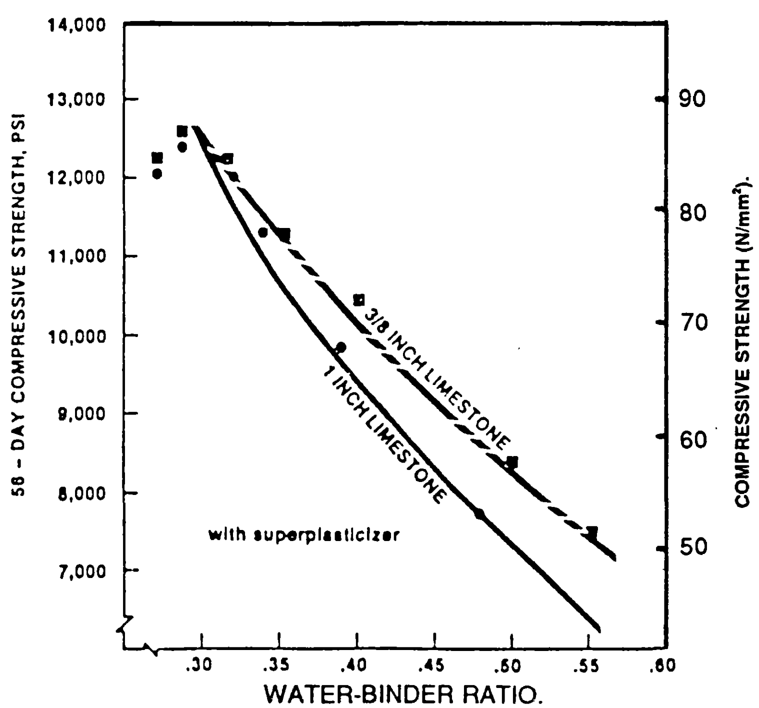


Figure 3.33: Compressive strength versus water-binder ratio for superplasticised concrete[59].

Maximum aggregate size	Compressive strength of concrete, (N/mm^2).			
	7-days	28-days	91-days	180-days
10 mm	76.7	98.4	109.3	112.4
14 mm	77.0	95.1	107.1	113.4
20 mm	77.0	95.1	109.6	113.2
28 mm	75.6	96.8	106.3	113.8

Table 3.3: Variation of the compressive strength of concrete as a function of the maximum size of coarse aggregate[(105)].

example, the ACI Committee 363[1] while it recommends the use of crushed aggregate, uses Cordon and Gillespie's strength efficiency envelope in order to recommend that "higher cement efficiencies are achieved at high strength levels with lower maximum aggregate sizes".

It may therefore be concluded that the compressive strength of concrete increases gradually as the maximum size of coarse aggregate decreases. The later data have shown that this effect may be reduced by the use of a superplasticiser, in combination with a good quality crushed aggregate and the use of a mineral admixture. More conclusive data are therefore needed in this area.

(IV) Modulus of elasticity.

The difference in the elastic moduli of the paste and the aggregate has received some attention, but experimental data on its effect on the compressive strength of concrete are lacking. Aitcin[102] used the following example in order to show the importance of having similar elastic moduli:

> "When the interface of the hydrated cement and aggregate is subjected to stress, the two materials must deform the same amount in order to avoid local stress concentrations. If, for example, the aggregate has a Young's modulus that is 1.5 times that of the hydrated cement the latter will, for a given increase in stress, be deformed roughly 1.5 times as much as the aggregate. This will inevitably lead to the movement of one surface over the other."

It has been suggested that the smaller aggregate sizes produce higher concrete strengths because of less severe concentrations of stress, caused by the different elastic moduli of the paste and the aggregate[1]. Since it has been shown that the effect of maximum size of aggregate on strength may be eliminated by improving the mortar-aggregate bond the effect of the difference in the elastic moduli on strength appears to be negligible.

(V) Mineralogy.

In certain circumstances bonding can also be mineralogical when some kind of chemical adhesion forms between the hydrated cement and the minerals included in the aggregates. According to Aitcin[102], crushed granite aggregates are not very desirable because the hydrated cement will not adhere to any extent to the micaceous and quartz grains present on the surfaces of such aggregate. There are, however, no experimental data to support this, but ACI Committee 363[1] advocates that it is a promising area for further research.

(VI) Cleanliness.

Coarse aggregates used in high strength concrete should be clean, i.e. free from detrimental dust coatings. Dust is generally important in so far as it may affect the total quantity of fines, and therefore the water demand, of the resulting concrete mixture. Detrimental, tenacious coatings of clay or shale, if not removed by washing or in the process of drum mixing, may also affect the aggregate-paste bond. Washing of crushed stone coarse aggregates may not always be necessary, but is generally recommended[139].

3.2.5 Fine aggregate.

Some studies[1,11,44,104,108], have stated that the fine aggregate gradation is not highly critical for the production of high strength concrete. However, it has also been reported that the properties of the fine aggregate, especially the particle shape and texture, have as great an effect on the mixing water requirement of concrete as the properties of coarse aggregate, and hence will at least indirectly influence strength[44,104,108].

High strength concretes typically contain such high contents of fine cementitious materials that it is sometimes helpful to increase the fineness modulus (FM) of the fine aggregate[1,44,104]. (The fineness modulus is an index number roughly proportional to the average particle size of a given aggregate; that is, the coarser the aggregate, the greater the fineness modulus. It is computed by adding the cumulative percentages passing standard sieves, starting at 0.15 mm and increasing in size by a factor of 2, and dividing the sum by 100). A sand with an FM below 2.5 results in a "stickier", less workable fresh concrete with a greater water demand(11,104,108). Use of a coarse sand with an FM of about 3.0 has been recommended(1,44,102,104,108). A National Crushed Stone Association report(139) recommended, in the interest of reducing the water requirement, to keep low the amounts passing the No. 50 and 100 (0.30 and 0.15 mm) sieves, thus increasing the FM.

Peterman and Carrasquillo's investigation(27) on non-superplasticised concrete, however, contradicts this. The coarsest sands required more mixing water for producing concrete with a slump of 3 to 4 in. (75 to 100 mm), (Figures 3.34 and 3.35). In several cases the mixes with the finest sand had the lowest water-cement ratios and they therefore produced the strongest concrete. Their investigation on superplasticised concrete showed that the mixes with finer sands required the greatest dosage of superplasticiser, (Figures 3.36 and 3.37). It must be noted that the fineness moduli of the sands used for non-superplasticised concrete were different from those used for superplasticised concrete. Their results, as was the case for their investigation of the effect of maximum coarse aggregate size on the compressive strength, are limited and inconclusive.

The use of a coarse sand, with an FM of about 3.0, in superplasticised high strength concrete may therefore be advantageous in reducing the "stickiness" and the water demand of the concrete. The increased water demand resulting from the use of finer sands can, however, be counteracted in high strength concrete by increasing the superplasticiser dosage. However the data are limited and inconclusive.

3.2.6 Mineral admixtures.

PFA and CSF have been widely used in North America and Norway respectively for the production of high strength concrete. Although GGBFS has been used in Europe for a number of years, data on its use in high strength concrete are limited(1).

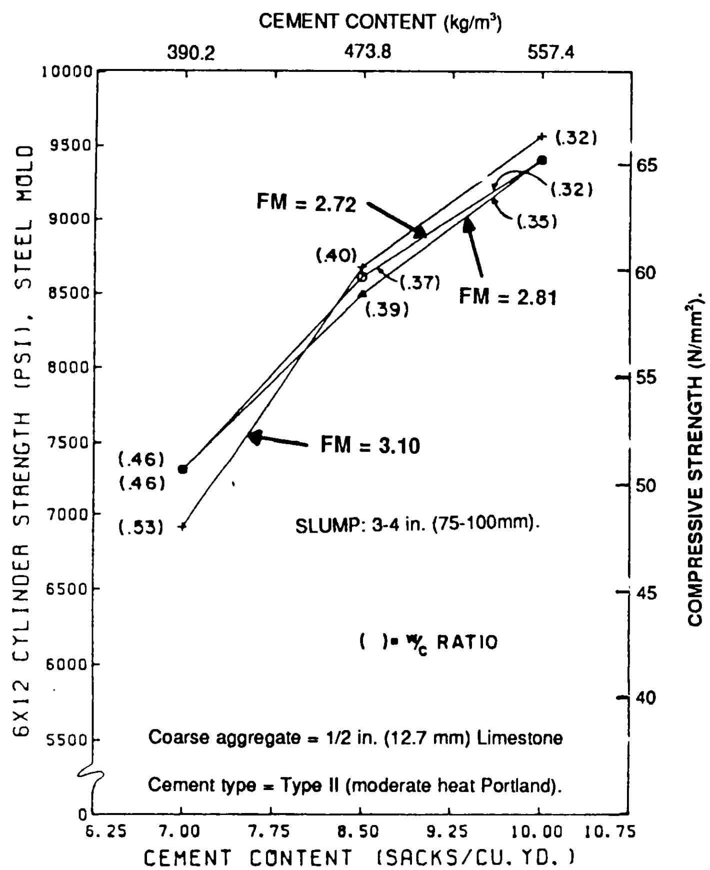


Figure 3.34: Effect of sand fineness and cement content on the 56-day compressive strength of non-superplasticised concrete with coarse/fine aggregate ratio of 1.5[27].

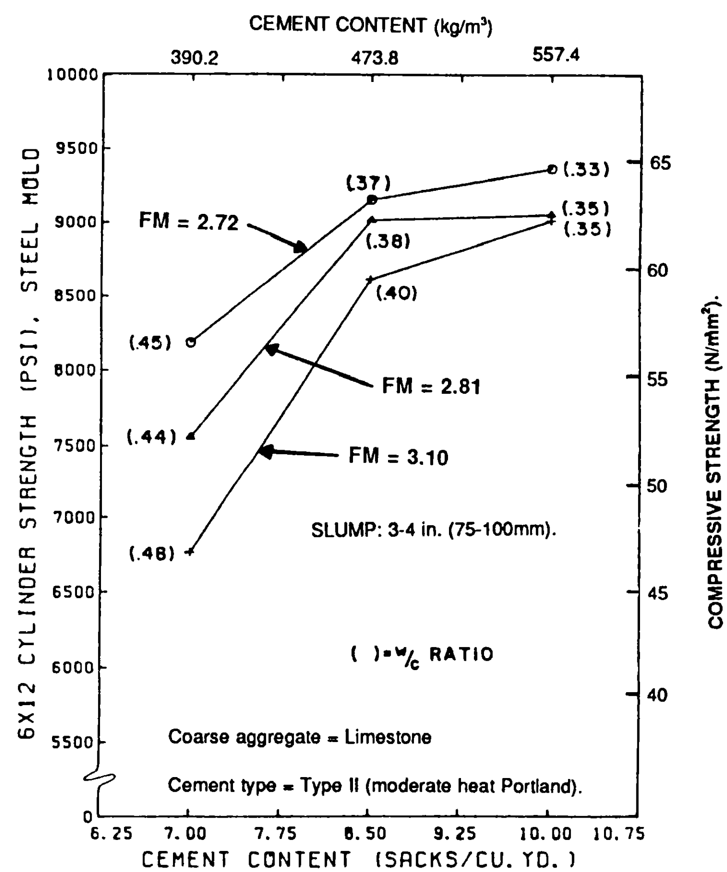


Figure 3.35: Effect of sand fineness and cement content on the 56-day compressive strength of non-superplasticised concrete with coarse/fine aggregate ratio of 2.0[27].

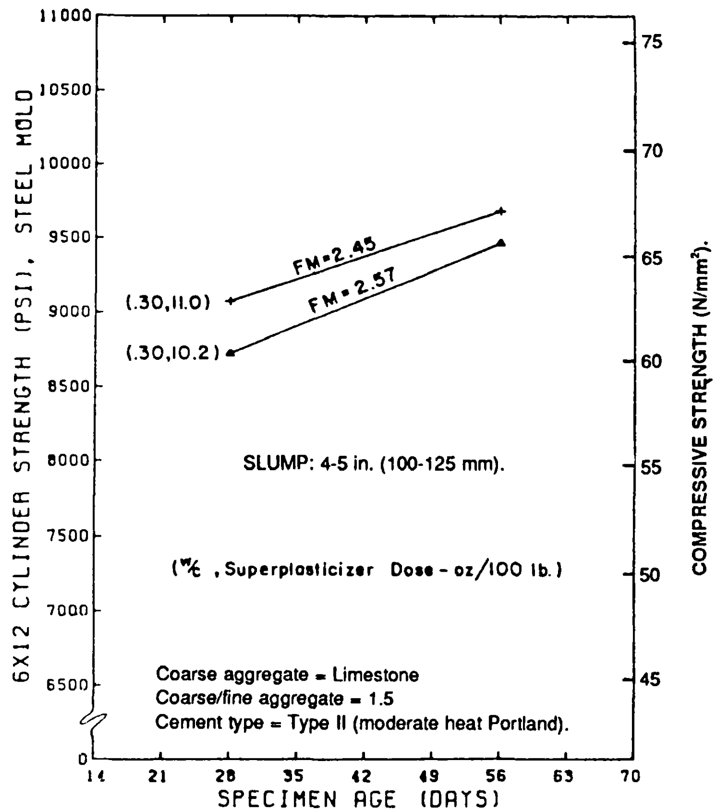


Figure 3.36: Effect of fineness modulus (FM) of sand on the compressive strength of concrete with a cement content of 8.5 sacks/yd³ (473.8 kg/m³)(27).

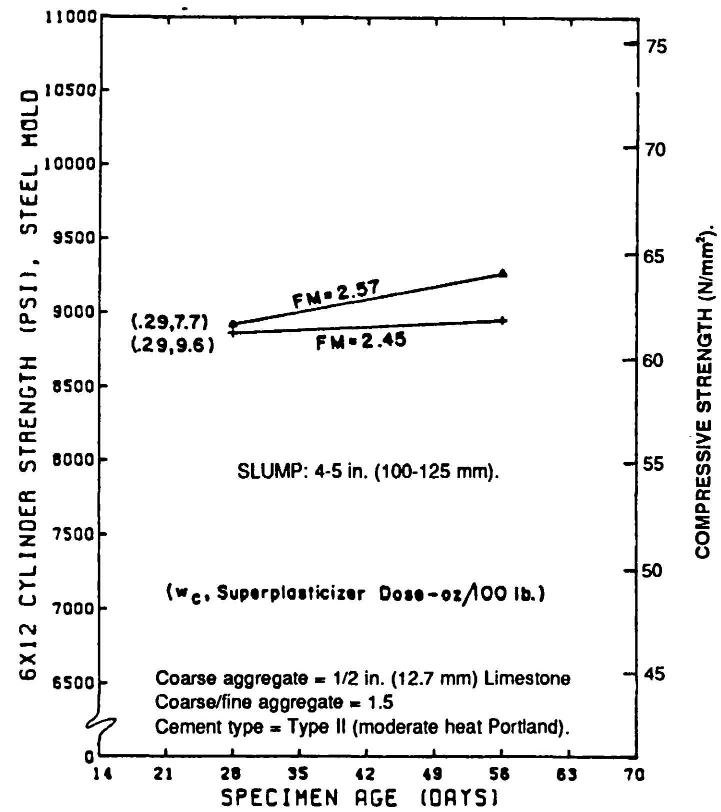


Figure 3.37: Effect of fineness modulus (FM) of sand on the compressive strength of concrete with a cement content of 10 sacks/yd³ (557.4 kg/m³)(27).

(I) Pulverised fuel ash (PFA).

PFA is a by-product of the combustion of pulverised coal in thermal power plants. It is removed by the dust collection system as a fine particulate residue from the combustion gases before they are discharged into the atmosphere.

PFA particles are typically spherical, ranging in diameter from less than 1 to more than 150 μm, the majority being less than 45 μm. The range of particle sizes in any given PFA is largely determined by the type of dust collection equipment used. The PFA from boilers at some older plants, where mechanical collectors alone are employed, is coarser than from plants using electrostatic precipitators.

The chemical composition of PFA is determined by the types and relative amounts of incombustible matter in the coal used. More than 85 per cent of most PFAs comprise chemical compounds and glasses formed from the elements silicon, aluminum, iron, calcium, and magnesium. Generally, PFA from the combustion of sub-bituminous coals contains more calcium and less iron than PFA from bituminous coal. Unburned coal collects with the PFA as carbon particles, the amount being determined by such factors as the rate of combustion, air-fuel ratio, and degree of pulverisation of the coal. Plants that operate only intermittently (peak load stations), burning bituminous coals, produce PFA with the largest percentages of unburned carbon.

PFAs exhibit pozzolanic activity. A pozzolan is defined[(144)] as "a siliceous or siliceous and aluminous material which in itself possesses little or no cementitious value but which will, in a finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperature to form compounds possessing cementitious properties." PFA contains meta-stable alumino-silicates that will react with calcium ions, in the presence of moisture, to form calcium silicate hydrates.

PFAs can differ significantly due to their composition and, to some extent, their origin. Canadian[(145)] and U.S.[(146)] specifications recognise two general classes of PFA:

- Class C, normally produced from lignite or sub-bituminous coals;
- Class F, normally produced from bituminous coals.

The Class C PFAs differ from Class F materials principally in often having a capacity for self-hardening in the absence of cement. The most notable chemical difference between these two classes is that the Class C PFAs contain high levels of calcium, (Table 3.4), and this has led to the use of an alternative, and in some ways preferable, terminology: high-calcium and low-calcium PFA for Classes C and F, respectively. Thermal power plants in the United Kingdom use bituminous coals, and therefore the PFAs contain a low level of calcium, (Table 3.5)[(147)], corresponding to the ASTM Class F PFA.

In a review, Parrott[(32)] concluded that using PFA in high strength concrete does not produce exceptionally high 28-day strengths. Indeed, Saucier[(12)] did not regard it as an essential ingredient for the production of concretes with compressive strengths in the range of 5,000 to 10,000 psi (34.4 to 68.9 N/mm^2). However, many other reports state that a good quality PFA is mandatory for producing high strength concrete[(8,104,108,139,148)]. For example, the Chicago Committee on High-Rise Buildings[(8)] has stated: "The use of a good quality PFA, meeting the specifications of ASTM C618 (Class F), ... is a must in the production of high strength concrete". Class F PFA appears to be the readily available pozzolan in Chicago, (c.f. page 30). These reports also advocate that: "PFA functions by providing increased strength at later ages of curing (56 to 91-days) that cannot be achieved through the use of additional Portland cement". None of these reports provides experimental data to support this statement, but all three refer you to the work done by Saucier et al[(149)]. His results, (Figure 3.38), only indicate that the main contribution of PFA to the strength of concrete is between 28 and 90-days, and that concretes with 10 per cent cement replacement by PFA may have higher 90-day strengths than the OPC concretes.

The small size and the essentially spherical form of low-calcium PFA particles have been credited with causing a reduction in the amount of water required for a given degree of workability from that required for an equivalent paste without PFA[(150)]. Advantage can be taken of the improved workability to reduce the amount of water used in a concrete whilst maintaining the same workability as OPC concrete.

Oxides	Mass percentage OPC	Low-lime pfa	High-lime pfa
SiO_2	20	50	40
Al_2O_3	5	28	18
Fe_2O_3	3	9	8
CaO	64	3	20
MgO	2	1	4
SO_3	2	1	2
Others	4	8	8

Table 3.4: Typical bulk oxide compositions of Class C (high lime) and Class F (low lime) PFAs[147].

Oxides	Mean (wt %)	Range (wt %)
SiO_2	48.7	41.7–52.7
Al_2O_3	27.8	20.3–35.7
Fe_2O_3	9.2	5.4–12.5
CaO	3.0	1.1– 7.8
MgO	1.9	1.5– 2.4
SO_3	0.9	0.1– 1.2
Na_2O	1.3	0.1– 6.3
K_2O	2.4	1.1– 3.1
TiO_2	1.1	0.1– 1.5
P_2O_5	0.3	0.1– 0.7
LOI	3.9	1.1– 8.0

Table 3.5: Typical major oxide analyses of PFAs in the United Kingdom[147].

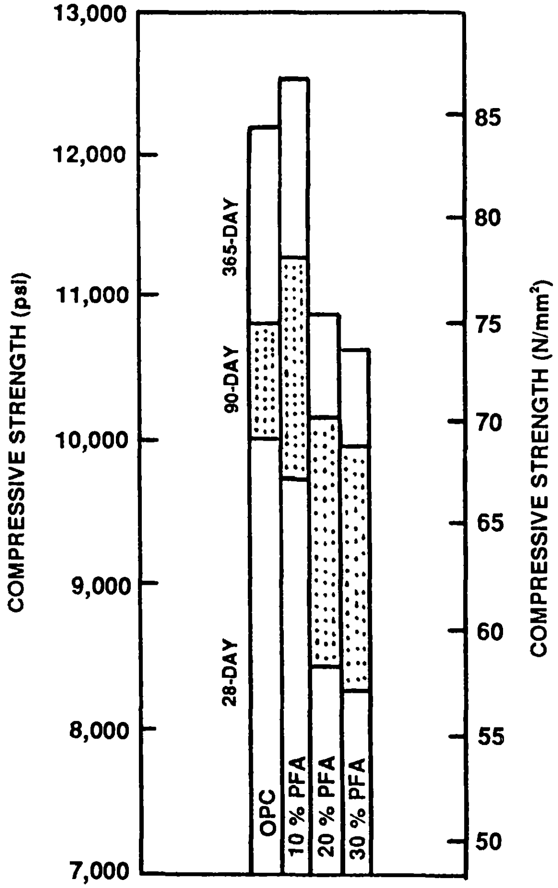


Figure 3.38: Effect of partial cement replacement by PFA on the compressive strength of concrete[149].

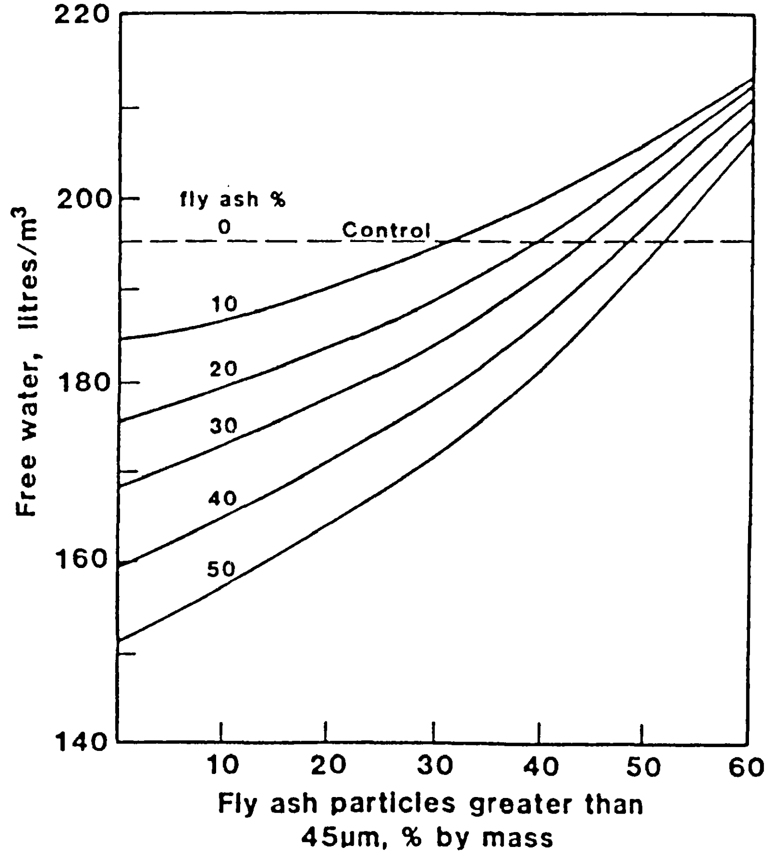


Figure 3.39: Influence of coarse particulate content of PFA on the water content required for 50-75 mm slump concrete[151].

According to Owens[151], the major factor influencing the effects of PFA on the workability of concrete is its proportion of coarse material (>45 μm). He has shown that, for example, substitution of 50 per cent by mass of the cement with fine particulate PFA can reduce the water demand by 25 per cent; a similar substitution using PFA with 50 per cent of the material greater than 45 μm has no effect on water demand. The general effect of coarse PFA particles on the water demand of 50-75 mm slump concrete is illustrated in Figure 3.39.

The effects of PFA on the workability of concrete have also been investigated using Tattersall's two point apparatus[152,154]. For example, Ellis[152] showed that increasing cement replacement by PFA reduced both the yield (g) and plastic viscosity (h) values of concrete, (Figures 3.40 and 3.41). Two experimental attempts[153,154] have also been made to relate the effects of PFA on the rheology of pastes to its effects on fresh concrete. It must be noted that while a standard viscometer was used by both for tests on pastes, two different apparatuses were used for concrete. Ivanhov and Zacharieva[153] used a ball immersed into vibrated concrete, the experimental apparatus was not described, while Banfill[154] used Tattersall's two point test. Nonetheless the results are contradictory:

(i) The results of Ivanhov and Zacharieva[153] showed that cement replacement by PFA increases the plastic viscosity value, that they called (η), of pastes. Their results on concrete showed a decrease of both the yield stress value, that they called (τ_0), and (η) for cement replacement by PFA of 20 per cent, while an increase for higher replacement levels, (Figure 3.42). From this they also concluded that the higher the specific surface of the PFA the higher is its contribution to the improvement of the rheological properties of fresh concrete with low cement content. These effects were however shown to be affected by the water-cement ratio, (Figure 3.43). This shows that the (τ_0) value of the 50 per cent PFA mix is lower than that of the neat OPC mix for water-cement ratios of 0.64 and 0.67, but is higher for higher water-cement ratios. Further interpretation of their results is, as also reported by Banfill[154], made difficult by ambiguity in the reporting of their data.

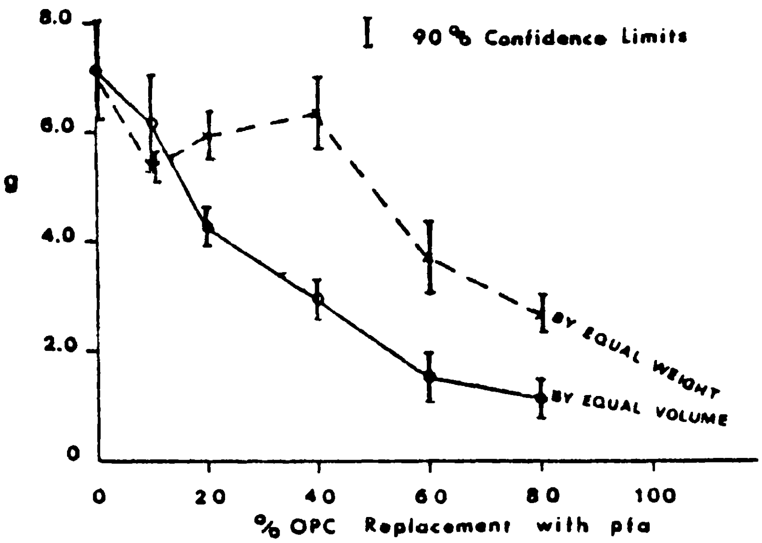


Figure 3.40: Relationship between yield (g) value and percentage cement replacement by PFA[(152)].

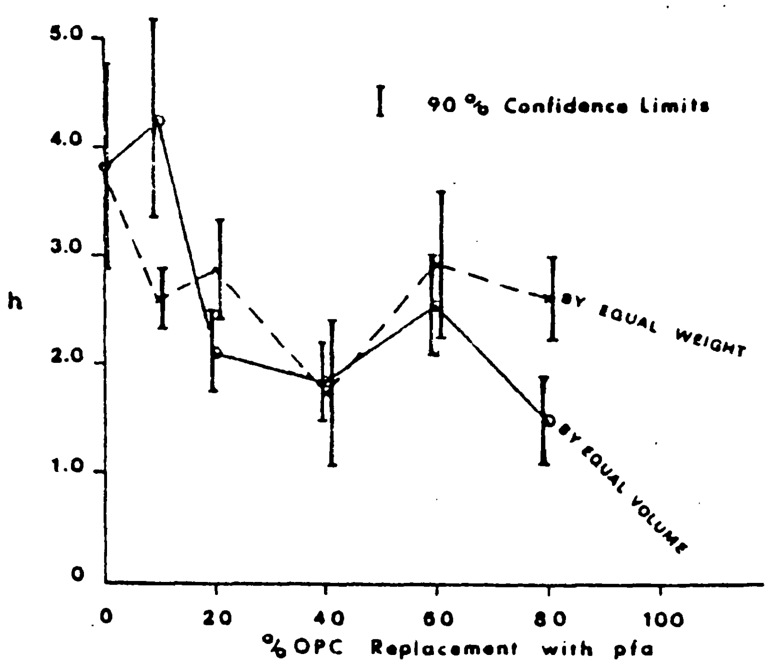


Figure 3.41: Relationship between yield (g) value and percentage cement replacement by PFA[(152)].

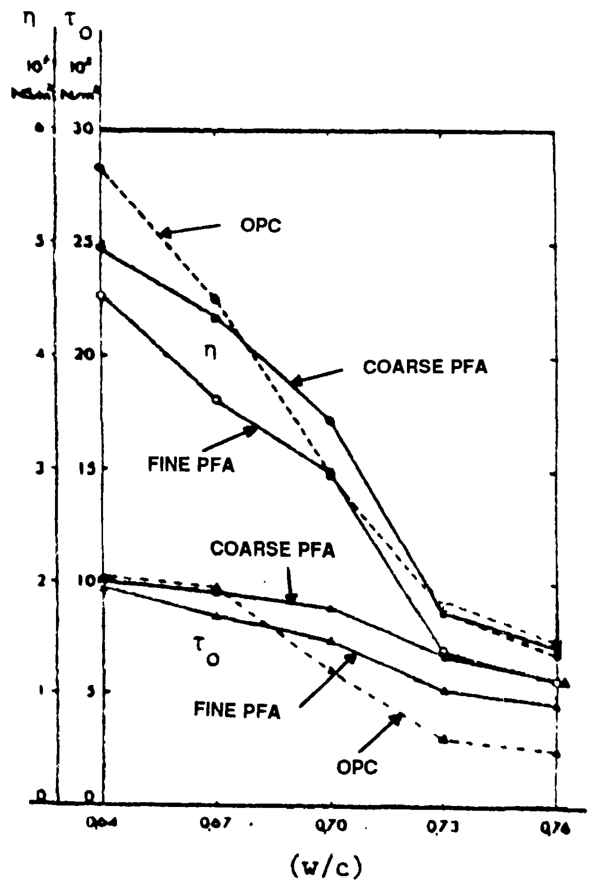


Figure 3.42: Effect of water-cement ratio on the yield (τ_0) and plastic viscosity (η) values for concretes with 150 kg/m^3 cement content[153].

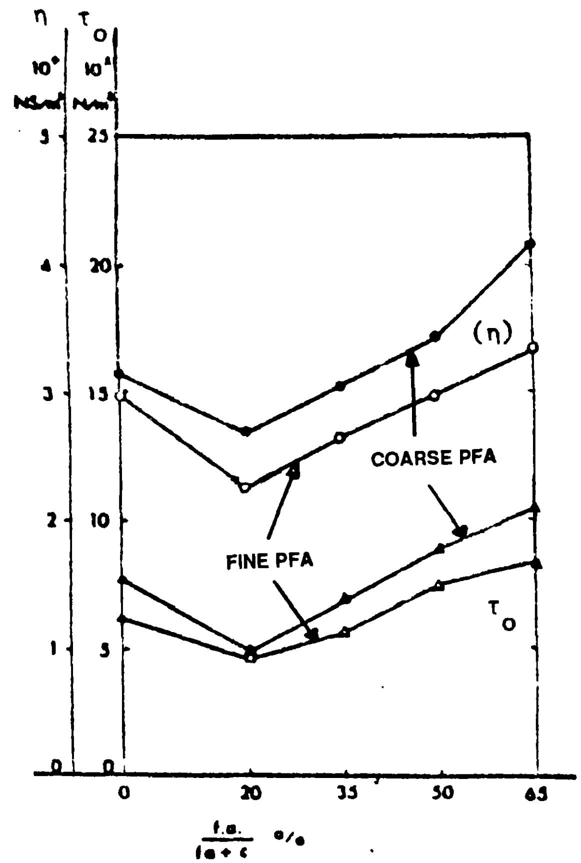


Figure 3.43: Effect of the level of cement replacement by PFA on the yield (τ_0) and plastic viscosity (η) values for concretes with a water-cement ratio of 0.7 and cement content of 150 kg/m^3.[153]

(ii) Banfill[154] on the other hand showed that with increasing level of cement replacement by PFA, the yield value of pastes was reduced while the plastic viscosity value was unchanged within experimental error, (Table 3.6). The effects on the rheological properties of fresh concrete were similar, (Table 3.7). Mix B showed not only a reduction of the yield (g) value but also of the plastic viscosity (h) value. No explanation was given for this.

From an extensive laboratory study of the combined use of PFA with superplasticisers, Lane and Best[155] have drawn the following conclusions:

> "... superplasticisers are compatible with PFA in concrete and produce no detrimental effects. The benefits claimed for these admixtures in plain concrete were not, however, as apparent in PFA mixtures, particularly with respect to compressive strength gains and duration of increased plasticity. The highly plastic phase diminishes after 15 minutes and ceases after about 30 minutes with PFA concrete. Water reductions for equal slump did not exceed 15 per cent, improving this characteristic only slightly over a standard water-reducing agent. The low water reductions can be attributed to the lower water requirement for PFA concrete as compared to plain concrete for equal consistencies. Since there is less excess water initially available, the addition of water-reducers is less effective."

Almeida and Goncalves[156] have also shown that the "highly plastic phase" diminishes very quickly when a superplasticiser is used, (Figure 3.44). In both superplasticised and non-superplasticised groups the OPC mixes were the first to reach zero slump. From this they concluded that partial cement replacement by PFA, CSF or natural pozzolana reduces the rate of workability loss.

Djellouli et al[157] advocated that:

> "Replacing in some cases up to 20 per cent of cement by a less reactive cementitious material like PFA or up to 50 per cent by GGBFS can solve

Cement %	pfa %	Yield Value N/m^2	Plastic Viscosity Ns/m^2
100	0	71.9	0.29
90	10	67.9	0.30
80	20	61.0	0.31
60	40	31.5	0.34
40	60	15.7	0.30

Table 3.6: Effect of cement replacement by PFA on the rheology of cement pastes with a water-binder ratio of 0.35[154].

Mix	Cement kg/m^3	Water kg/m^3	Aggregate kg/m^3	Sand %	pfa %	Slump mm	g Nm	h Nms
A	250	165	2025	35	0	22	10.4	5.3
					10	33	7.4	6.7
					20	50	4.5	6.3
					40	*	2.9	7.0
					60	*	1.7	5.3
B	250	180	1985	45	0	28	6.7	7.2
					10	25	7.7	6.4
					20	90	2.8	3.0
					40	180	1.5	3.2
					60	180	1.6	2.3
C	450	165	1825	35	0	27	7.3	4.7
					10	30	4.4	5.8
					20	46	6.7	4.1
					40	55	5.5	4.9
					60	95	4.3	3.8
D	450	180	1785	45	0	24	5.8	2.3
					10	45	7.2	2.1
					20	72	3.4	2.8
					40	90	3.3	2.3
					60	120	2.4	2.7

* Denotes collapse slump, mix also segregated.

Table 3.7: Effect of cement replacement by PFA on the rheology of fresh concrete[154]

Compositions and some characteristics of concretes

COMPONENTS (kg/m³)	B1	B2	B3	B4	B5	B6	B7	B8	B9	B10
C. aggregate 15/25	660	696	700	691	617	616	691	685	650	626
C. aggregate 5/15	428	451	454	448	401	400	448	446	421	406
F. aggregate	645	681	685	676	602	600	676	670	635	611
Cement	500	500	450	450	450	450	450	450	450	450
Water	182	122	31*	120	159*	212	122	126	182	201
Admixture (l/m³)	-	15	15	15	-	-	15	15	-	-
Silica fume A (l/m³)	-	-	105*	-	76*	-	-	-	-	-
Silica fume B	-	-	-	50	-	50	-	-	-	-
Fly ash	-	-	-	-	-	-	50	-	50	-
Pozzolana	-	-	-	-	-	-	-	50	-	50
CHARACTERISTICS										
Water content variation (%)	0	-34	-31	-34	+15	+17	-33	-31	0	+10
W/(C+a) ratio	0.36	0.24	0.25	0.24	0.42	0.42	0.24	0.25	0.36	0.40
Compressive strength (MPa) 28d; 15cm cube	67	84	106	105	75	72	87	91	66	61

* Depending on the water content of the product

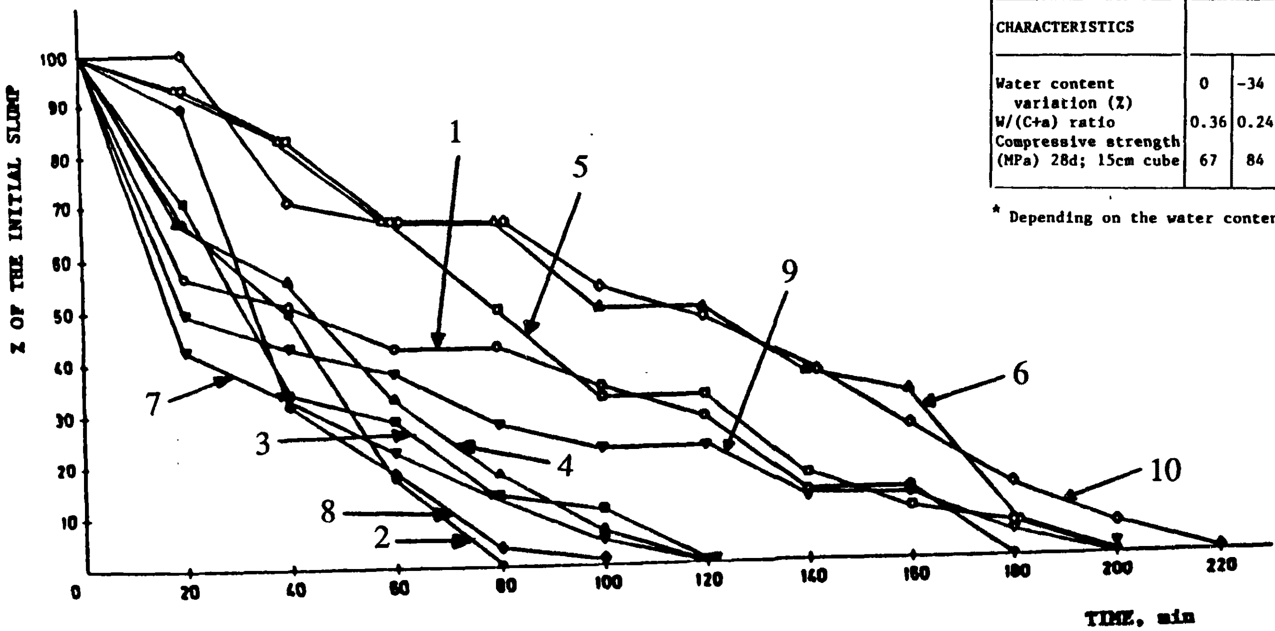


Figure 3.44: Rate of workability loss of concretes with and without a superplasticiser and mineral admixtures(168).

the slump loss problem observed with some very reactive cements when used at water-binder ratios ranging from 0.25 to 0.30."

However, they gave no experimental data to support this. Instead some justification for this recommendation was given in the following:

"Replacing a certain fraction of cement by PFA or GGBFS, which are less reactive and reduce the amount of ettringite developed during the early hydration, results in a better control of the slump loss problem and reduces the superplasticiser dosage required to achieve a given workability."

Several reports[2,12,32,102] recommend the use of PFA in order to moderate heat evolution and thereby limit early age temperature rises of the cement-rich mixes used for the production of high strength concrete. However, they seem to base their recommendation on investigations of the effect of partially replacing cement by PFA on the temperature rise of normal strength concrete, e.g., Bamforth's study[93] on the effect of partial cement replacement using PFA at levels up to 50 per cent, for concretes with a total cement content of 400 kg/m^3. This has shown that at the 15 per cent replacement level, the temperature of the PFA concrete marginally exceeded that of the OPC concrete, despite a small reduction in the early adiabatic temperature rise, (Figure 3.45). It was therefore concluded that PFA, under certain conditions, can have a potentially higher level of hydration than OPC. Higher replacement levels were required to reduce the adiabatic temperature rise; the reduction with 30 per cent PFA was 10 to 15 per cent, increasing to 30 per cent with 50 per cent PFA.

The above study, which also included partial cement replacement by GGBFS (which will be described in the next section), was carried out in order to solve the controversy, that existed in 1973-76, over the benefits which might be gained, in reduced temperature rise during hydration, from the use of either PFA or GGBFS to partially replace Portland cement. In-situ data for concrete pours up to 4.5 m deep with binder contents ranging from 300-465 kg/m^3 indicated that a temperature reduction could be achieved[87,89,158,159]. However, the Cement and Concrete Association advocated that all binders, including PFA

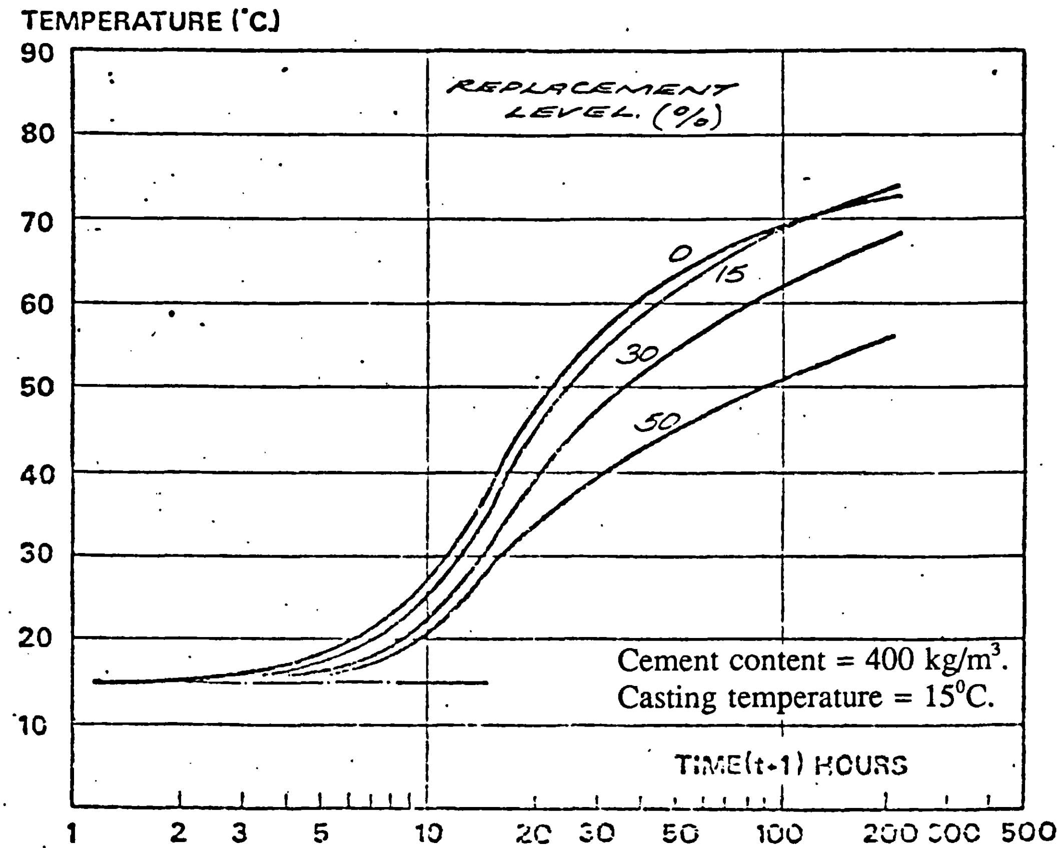


Figure 3.45: The effect of partially replacing cement with PFA on the adiabatic temperature rise of concrete[93].

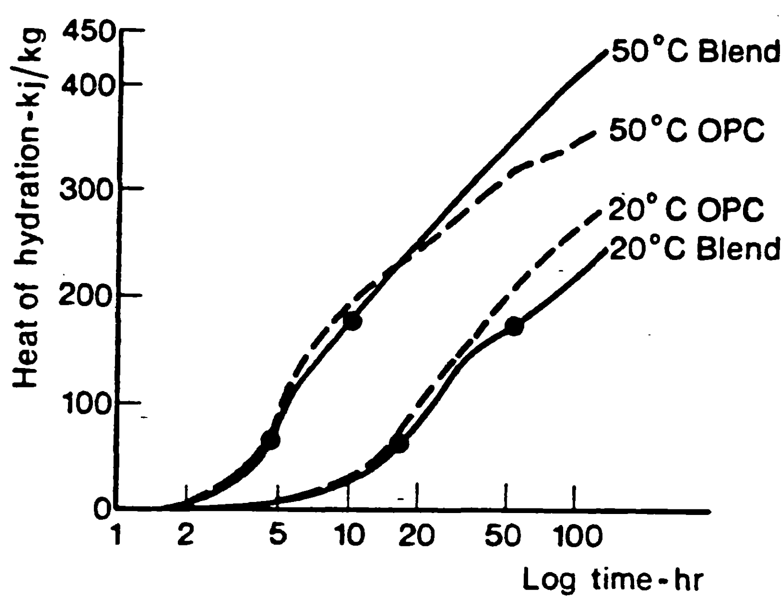


Figure 3.46: Rates of heats of hydration of pure OPC and 3 to 1 OPC and GGBFS blend[86]. Isothermal calorimetry was used and the temperatures for the tests were 20 and 50°C.

and GGBFS, should be treated alike when considering problems associated with heat of hydration[85,86]. In particular, there was some doubt as to whether in very large cement-rich pours cast in hot weather, the effect of Portland cement replacement was as significant as that monitored under less rigorous conditions, e.g., the casting temperature, which traditionally for mass concrete was purposely kept as low as possible. FitzGibbon[86] claimed that casting temperatures on site were not always as low as 10^0C or below, i.e. the low temperature required to bring out the full low heat advantages of blended cements, but more realistic casting temperatures were 20^0C or even 30^0C for summer casting. He showed, using isothermal calorimetry, that the heat generated by a 3:1 OPC and GGBFS blend was affected by the temperature at which the test was carried out; the blend generated a lower total heat at 20^0C but higher at 50^0C than the pure OPC, (Figure 3.46). He stated that the use of 50^0C temperature was justified as it was well within the range of temperatures that had been recorded in normal structural concretes where OPC or blended cements had been used. The peak temperatures recorded in structural applications of high strength concrete have been as high as 90^0C, (see pages 54-58), despite the use of low heat binders. This therefore raises some doubts as to whether the effect of Portland cement replacement in high strength concrete is as significant as that monitored for normal strength concrete.

Pulverised fuel ash (PFA) is often cheaper than Portland cement and this factor in combination with its technical advantages, summarised below, tend to encourage its use in high strength concrete wherever feasible.

1. A good-quality PFA will generally permit the water content of a concrete mix to be reduced without loss of workability. Partial cement replacement with PFA may also reduce the rate of slump loss experienced with superplasticised concrete. More supporting data are however needed before this can be substantiated.

2. PFA contributes to the strength of concrete at later ages of curing than 28-days. Concretes with 10 per cent cement replacement by PFA may have higher 90-day strengths than the corresponding OPC concretes.

3. The use of PFA to partially replace Portland cement has been shown to reduce the temperature rise during hydration of normal strength concrete. An investigation is, however, required to assess whether the effect of partial cement replacement in high strength concrete is equally significant.

(II) Ground granulated blast-furnace slag (GGBFS).

In the production of iron, the blast furnace is continuously charged from the top with iron oxide sources (ore, pellets, sinter, etc.), fluxing stone (limestone and dolomite), and fuel (coke). Two products are obtained from the furnace: molten iron that collects in the bottom of the furnace (hearth) and liquid blast-furnace slag floating on the pool of iron. Both are periodically tapped from the furnace at a temperature of about 1500^0C. The slag consists primarily of silica and alumina (which come from the iron ores), combined with the calcium and magnesium oxides (which come from the fluxing stone), and with some impurities from the coke charged into the blast-furnace. The major oxides, silica, alumina, lime, and magnesia, constitute 95 per cent or more of the total oxides. The cementitious action of a slag is dependent to a large extent on the glass content, although other factors will also have some influence. Slowly cooled slags are predominately crystalline and therefore do not possess significantly cementitious properties. To maximise hydraulic (cementitious) properties, the molten slag must therefore be chilled rapidly as it leaves the blast furnace. Rapid "quenching" or chilling minimises crystallization and converts the molten slag into fine aggregate-sized particles, composed of predominantly noncrystalline, glassy, material. "Quenching" of the molten slag with water, as shown in Figure 3.47[160], is the most common process, and the product is referred to as granulated blast-furnace slag (GGBFS). A newer granulation process, shown in Figure 3.48, involves the use of a pelletizer. The resulting product, referred to as pelletized blast-furnace slag, may also have a high glass content, and can be used as a cementitious material, or in the larger particle sizes, as lightweight aggregate. The granulated or pelletized blast-furnace slag that is to be used as a cementitious material is then dewatered, dried, and ground to a similar fineness to Portland cement.

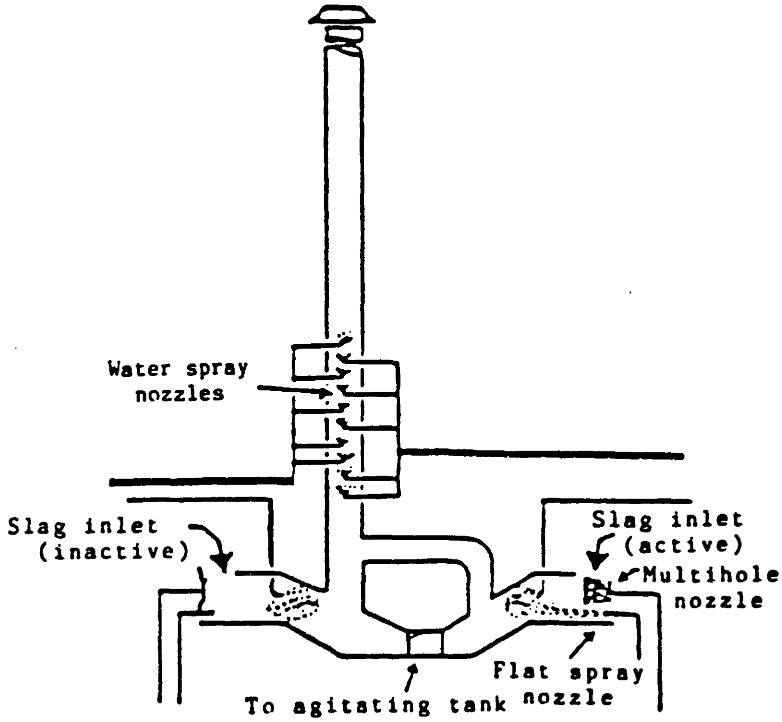


Figure 3.47: Typical configuration of blast-furnace slag water granulator showing the steam condensing tower[160].

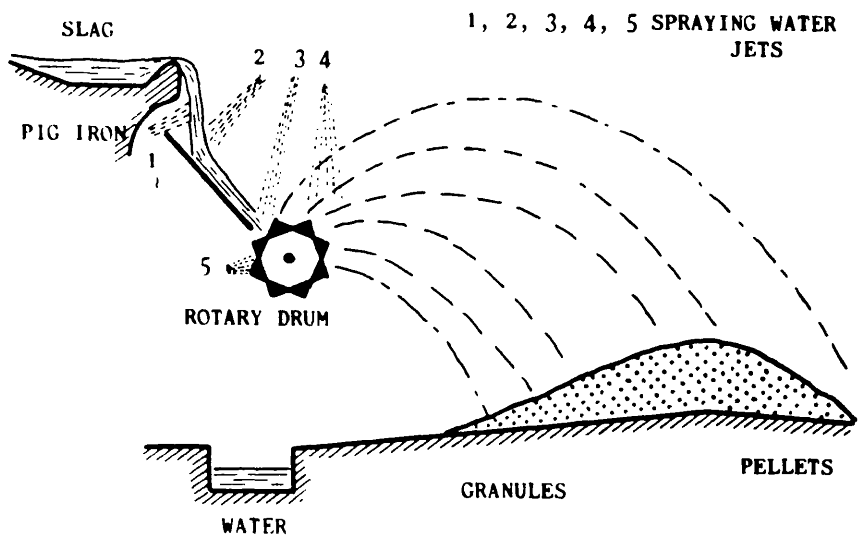


Figure 3.48: Blast-furnace slag pelletization process, using a minimum of water usually applied at the vibrating feed plate[147].

According to ACI Committee 363[1], GGBFS shows particular promise for high strength concrete. However, although GGBFS has been used in Europe for a number of years for the production of normal strength concrete, data are limited on its use in high strength concrete.

Meusel and Rose[162] have reported that as the GGBFS portion of the cementitious material in non-superplasticised concrete is increased there is an increase in slump. Limited investigations at CANMET[161] have shown that superplasticisers are as effective in pelletized blast-furnace slag concrete as they are in normal Portland cement concrete, in spite of different dosage requirements.

Parrott[163] observed that substitution of GGBFS for Portland cement reduced 28-day compressive strengths with high strength concrete mixes of given proportions or a given workability, but at 90-days all mixes achieved strengths of about 100 N/mm^2; water-binder ratios were 0.26 to 0.28 and no superplasticisers were used. Mphonde[164] achieved similar strengths at 28-days with superplasticised GGBFS concretes in which the water binder ratio was 0.27, and the level of cement replacement was 50 per cent.

Nakamura[165] investigated the effectiveness of GGBFS of different fineness in the production of high strength concrete. He showed that concrete with the ordinary GGBFS (364 m^2/kg Blaine) could not reach the strength level of the control, even at the age of 91-days, (Figure 3.49). However, the concrete with the very fine GGBFS (715 m^2/kg Blaine) showed comparable strengths to the control mix at the age of 7-days and from 28-days onward resulted in higher strengths, even at the substitution rate of 70 per cent.

Another factor which can affect the performance of GGBFS in concrete is the water-binder ratio. Meusel and Rose[162] showed that the percentage of strength gain achieved with GGBFS is greater in concrete mixtures which have high water-binder ratios, (Figure 3.50). The same trend was also noted by Malhotra[166] when pelletized slag was used.

Only limited literature has been found on the effect of partially replacing Portland cement by GGBFS on the temperature rise of high strength concrete. Bamforth's study[93], (see

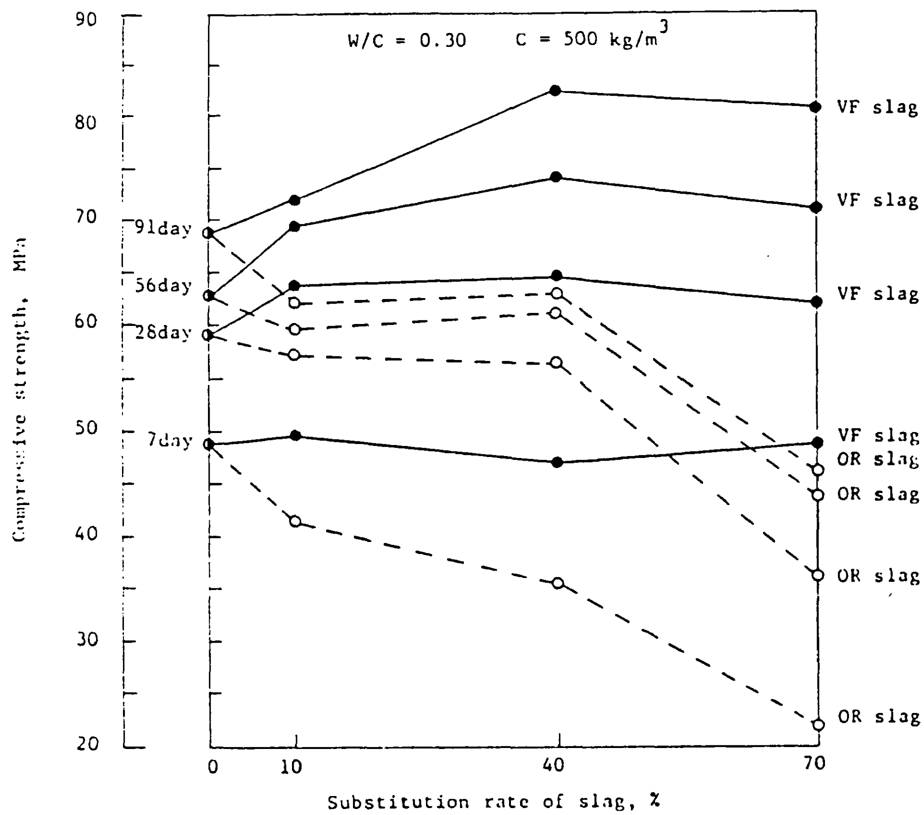


Figure 3.49: Effect of ordinary (OR) and very fine (VF) GGBFS on the compressive strength of concrete(165).

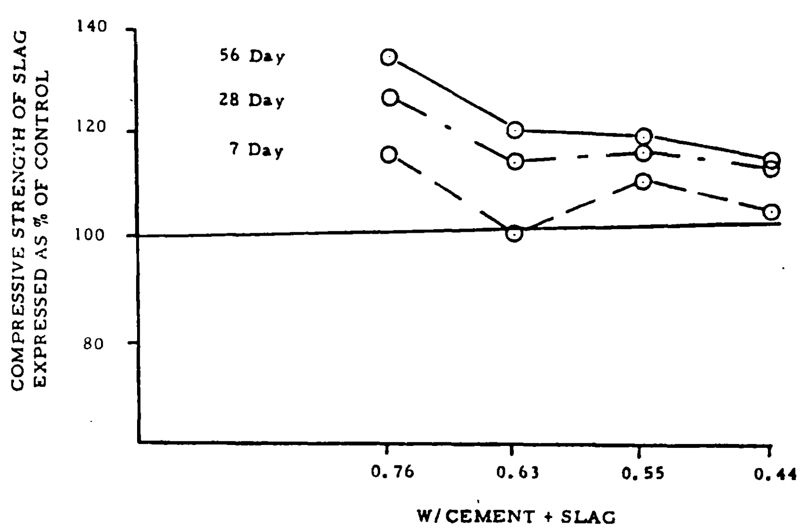


Figure 3.50: Effect of water-binder ratio on the percentage strength gain achieved with GGBFS(162).

pages 39 and 122), also included the effect of partial cement replacement using GGBFS at levels up to 75 per cent for normal strength concretes with a total cement content of 400 kg/m^3,(Figure 3.51). This shows that, at a nominal casting temperature of 15^{0}C, the effect of a 25 per cent replacement level was to marginally reduce the early rate of hydration, but to increase the temperature rise after 8-days, indicating that the GGBFS has a potentially higher level of hydration than Portland cement. At higher levels of replacement the temperature rise was reduced both in the short and long term indicating a reduction at 8-days of up to about 40 per cent for the 75 per cent replacement mix compared with the control concrete. It would appear, therefore, that the ratio of Portland cement to GGBFS is critical in determining the heat generating characteristics of the GGBFS itself; the higher ratio, the greater the temperature rise attributable to the GGBFS in the long term.

The only available adiabatic temperature data for high strength concrete made using GGBFS, (Figure 3.52)[(165)], indicates that even 40 per cent cement replacement by GGBFS of ordinary fineness may increase the adiabatic temperature rise of concrete.

Ground granulated blast-furnace slag (GGBFS), like PFA, is often cheaper than Portland cement and according to ACI Committee 363[(1)], it shows particular promise for high strength concrete production. However, there have been very few investigations on its use in high strength concrete.

Areas that need further research are:

1. *Workability:* Data from normal strength concrete show that GGBFS permits the water content to be reduced without loss of workability. Quantification of this effect for high strength concrete is required.

2. *Strength:* It has been shown that concrete with a compressive strength of as high as 100 N/mm^2 can be produced with cement replacement level by GGBFS of 50 per cent. However, studies are required that will quantify the contribution of GGBFS to strength, especially at very low water-binder

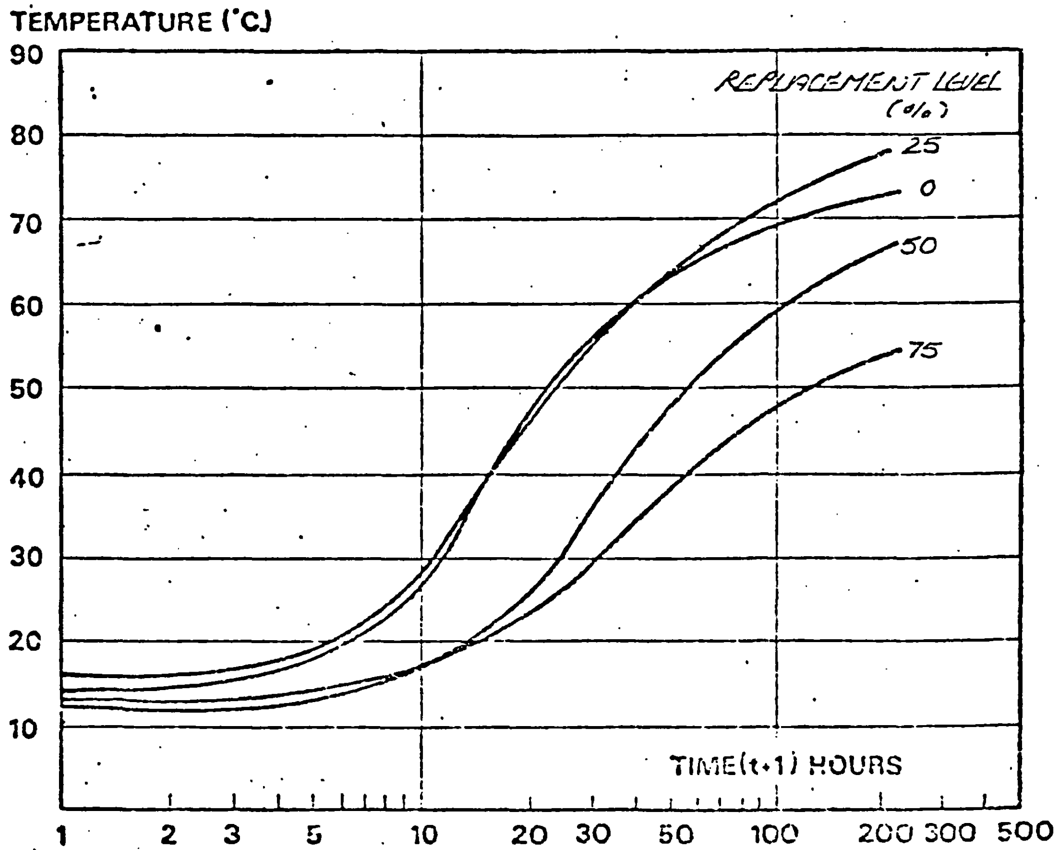


Figure 3.51: The effect of partially replacing cement with GGBFS on the adiabatic temperature of concrete(93).

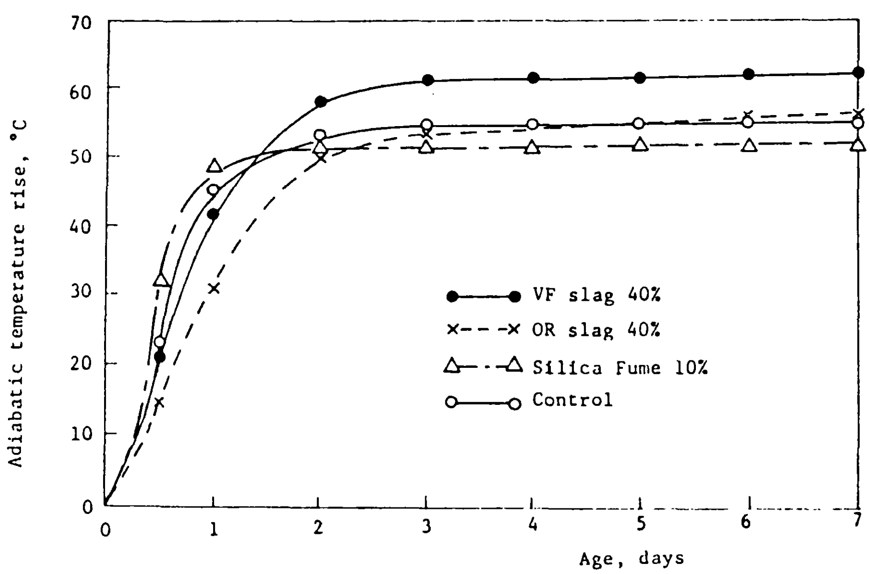


Figure 3.52: Adiabatic temperature rise of high strength concrete made using ordinary (OR) and very fine (VF) GGBFS(165).

ratios.

3. *Adiabatic temperature rise:* The use of GGBFS, like PFA, to partially replace Portland cement has been shown to reduce the temperature rise during hydration of normal strength concrete. An investigation is, however, required to assess whether the effect of partial Portland cement replacement in high strength concrete is equally significant.

(III) Condensed silica fume (CSF).

CSF is a by-product resulting from the reduction of high-purity quartz with coal in electric arc furnaces in the production of silicon and ferrosilicon alloys. The fume, which has a high content of amorphous silicon dioxide and consists of very fine spherical particles, is collected from the gases escaping from the furnaces. CSF is also collected as a by-product in the production of other silicon alloys such as ferrochromium, ferromanganese, ferromagnesium, and calcium silicon. Few published data are available on the properties of these, except that CSF from ferrochromium has properties somewhat similar to those obtained from ferrosilicon; the use of these CSFs should be avoided unless data on their performance in concrete are available. The silica content of the by-product silica fume collected is related to the manufacture of silicon alloys as shown in Table 3.8[167].

The specific gravity of a typical CSF is about 2.2 as compared to 3.1 for normal Portland cement. CSF consists of very fine vitreous particles with a surface area on the order of 20,000 m^2/kg when measured by nitrogen adsorption techniques. The particle-size distribution of a typical CSF shows most particles to be smaller than one micrometer (1 μm) with an average diameter of about 0.1 μm, which is approximately 100 times smaller than the average cement particle. CSF, because of its extreme fineness and high active silica content as compared to other pozzolanic materials, (Table 3.9), is a highly effective pozzolanic material[167].

It is generally accepted that the use of CSF in high strength concrete, because of its

Ferrosilicon alloy/metal*	SiO_2 content of silica fume
50 percent ferrosilicon	72 to 77 percent
75 percent ferrosilicon	84 to 88 percent
Silicon (98 percent)	93 to 98 percent

*Ferrosilicon alloys are produced with nominal silicon contents of 50, 75, and 90 percent. When the silicon content reaches 98 percent, the product is called silicon metal rather than ferrosilicon. As the silicon content increases in the alloy, the SiO_2 content will increase in the silica fume. Silica fume from grades 75 percent and higher are being sold in the United States for use in concrete.

Table 3.8: The silica content of the by-product condensed silica fume collected from the manufacture of silicon alloys(167).

Typical oxide compositions (% by weight)

oxide	pfa	ggbs	csf	opc
SiO_2	48	36	97	20
Al_2O_3	27	9	2	5
Fe_2O_3	9	1	0.1	4
MgO	2	11	0.1	1
CaO	3	40	-	64
Na_2O	1	-	-	0.2
K_2O	4	-	-	0.5

Typical physical properties

	pfa	ggbs	csf	opc
specific gravity	2.1	2.9	2.2	3.15
particle size range (microns)	10-150	3-100	0.01-0.5	0.5-100
specific surface area (m^2/kg)	350	400	20,000	350

Table 3.9: Typical oxide compositions and physical properties of OPC, PFA, GGBFS and CSF.

extreme fineness, reduces the workability and requires a consequent addition of superplasticiser[167,168,169], (Figure 3.53). The addition of water to counteract the reduction of workability would produce an unacceptable strength penalty in most concretes. However, the use of a superplasticiser in concretes that do not contain CSF can produce significant strength gains as indicated earlier, and thus although some of the highest strengths reported have been obtained using CSF in superplasticised concretes, the relative contributions of CSF and the superplasticiser are often obscured[32].

Some workers have, however, obtained conflicting effects on workability. For example, Loland and Hustad[170] reported a reduction in water demand for lean mixes incorporating up to 10 per CSF, but did not offer an explanation.

Yogendran[169] investigated the variation of dosage of superplasticiser at constant slump for concretes with water-binder ratio of 0.34, and 0.28, (Figure 3.54). This shows that, at the 0.34 water-binder ratio, the amount of superplasticiser required to maintain the slump increases linearly from 10 to 30 per cent replacement, while at 5 per cent replacement no superplasticiser is required. At the water-binder ratio of 0.28, however, the amount of superplasticiser required at five and ten per cent replacement was nearly the same as the control mix. Overall, it appears that the efficiency of the superplasticiser increased in the presence of CSF. A similar conclusion was reached by Hjorth[171] and Sellevold and Radjy[172].

Two teams from France (one from Laboratoire Central des Ponts et Chaussees, Paris[173], and the other from the Institute National des Sciences Appliquees, Toulouse[174]) have shown that above a certain superplasticiser dosage CSF has a water reducing effect. Both claim that the use of ultrafine particles, i.e. grain size smaller than that of cement, facilitates the production of low water-binder ratio concrete by the filler effect: the grains fill the voids between those of cement thus reducing the water requirement. Buil et al[130] determined the optimum addition of CSF by workability measurements on mortar. CSF and superplasticiser were added to the mortar mix, at a constant superplasticiser-CSF ratio of 6 per cent, without any other modification of the mix. They concluded that the optimum CSF-cement ratio was 0.40, (Figure 3.55). Seki et al[175] showed that the

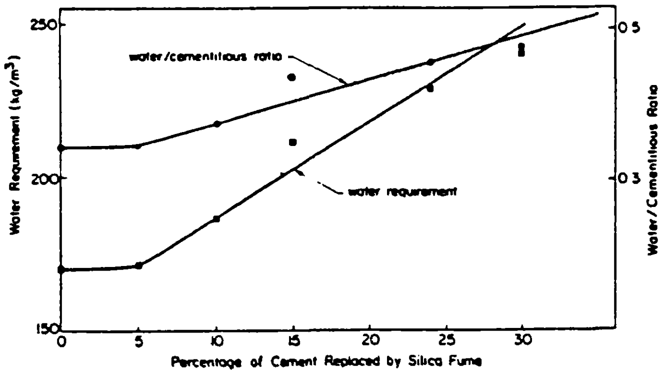


Figure 3.53: Variation of water requirement for CSF mixes at constant slump(169).

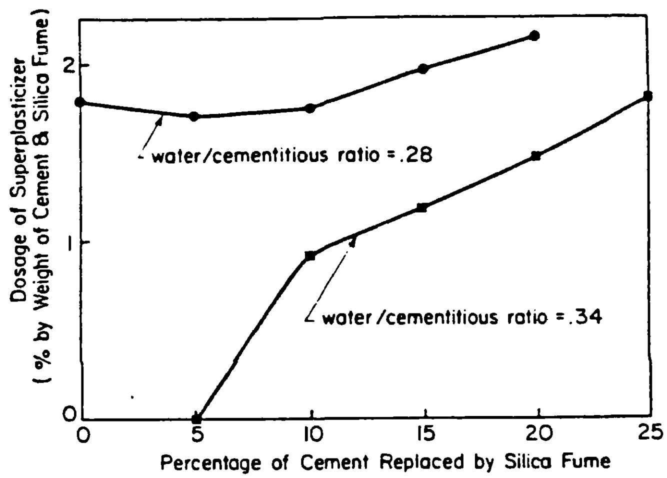


Figure 3.54: Variation of superplasticiser dosage for constant slump with various levels of cement replacement by CSF(169).

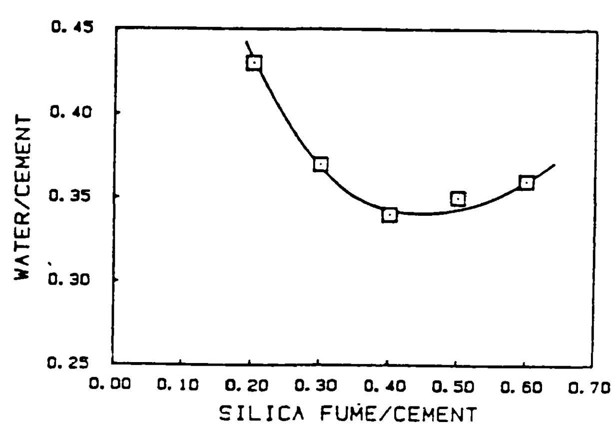


Figure 3.55: Water-cement ratio versus CSF-cement ratio for constant workability(130).

optimum cement replacement level by CSF in concrete is 0.26. This improvement would not have been achievable by the addition of cement, which would have necessarily reduced workability with a constant paste content, or increased the paste volume for the same workability.

Tachibana et al[138] showed that the electric energy consumed during mixing decreased by incorporating CSF in the high strength concrete mixes, (Figure 3.56), despite these having similar slumps after 3.5 minutes of mixing. He suggested that this is due to a better dispersion of cement particles by the use of CSF. This was more prominent for the low water-binder ratios used. De Larrard[176] also suggested that a small addition of CSF prevents the sedimentation of the cement and thus facilitates its flow. This is probably to be linked with the improvement in concrete pumpability, frequently observed with low CSF proportions.

Sellevold and Nilsen[177] have stated that the "static" and "dynamic" behaviour of CSF concrete do not relate in the same way as for OPC concrete, i.e. the slump measure does not predict the response to vibration in the usual way. For practical purposes they recommended that the slump should be increased by 20 to 30 mm for a CSF concrete to obtain the same workability as that for normal concrete. Because of this behaviour of CSF concrete, workers[169,178,179] have described CSF concrete as "sticky" or "plastic".

Another physical effect of CSF on concrete is the increased internal cohesion of fresh concrete. It is possible to design CSF concretes with essentially no bleeding or segregation. As a result, local areas of weakness such as bleed-water channels and voids under coarse aggregate particles can be eliminated. The transition zone between cement paste and coarse aggregate particles is an especially critical region in most concretes. It is frequently the weakest part because of bleed-water voids, yet it is under the greatest stress because of the elastic mismatch between the cement paste and the relatively stiff aggregate material. The reduction of bleeding in fresh concrete brings about significant improvements in the density of the transition zone and thus in the mechanical behaviour of the hardened concrete[180]. The strength of the transition zone can be further enhanced by the pozzolanic reaction. The reactivity of various pozzolans in cement paste or mortar

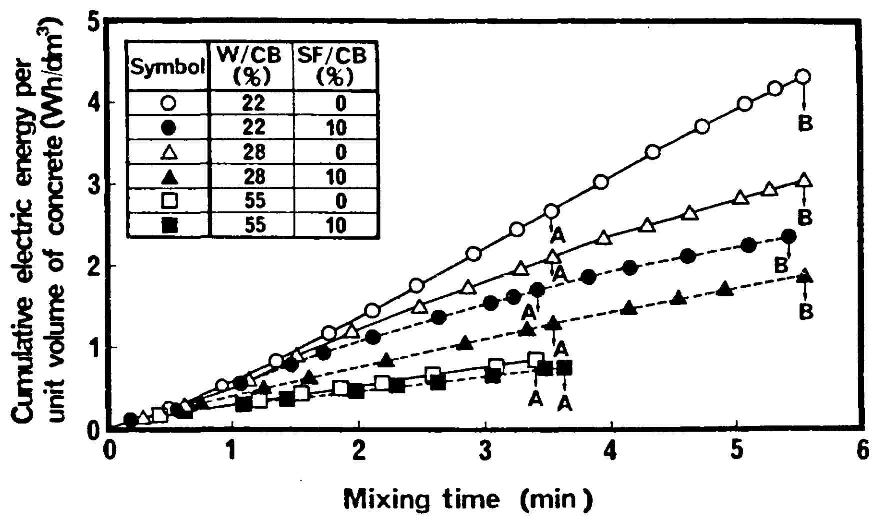


Figure 3.56: Cumulative energy consumed during mixing of concrete with various binders[138].

can be evaluated by measuring the calcium hydroxide content at different ages of the pastes. Differential Thermal Analysis (DTA), Thermogravimetric Analysis (TGA), and X-ray Diffraction (XRD) methods have been used. The difference in $Ca(OH)_2$ content for mature plain cement paste and that of cement/CSF pastes with different CSF content can be regarded as the amount consumed in the pozzolanic reaction, as shown in Figure 3.57.

In some proprietary formulations(181), concrete incorporating large percentages of CSF in combination with a very high superplasticiser dosage, and having a water-binder ratio of less than 0.25, has been produced. The resulting concrete had very low permeability, and compressive strength of the order of 150 N/mm^2. According to Bache(181), dense packing is the basis for the superiority of this type of concrete. The extremely fine CSF particles are packed in the spaces between the densely packed cement particles and normally packed coarse and fine aggregates.

CSF, because of its high surface area and high content of amorphous silica, reacts more quickly than ordinary pozzolans. According to Mehta(182), the pozzolanic reaction may begin as early as 2-days after cement hydration. Sellevold and Radjy(172) stated that the "main pozzolanic effect" of CSF in concrete takes place between the ages of 3 and 28-days for curing at 20^{0}C.

The use of CSF has been shown by many workers(184,183,185) to lead to a refinement of the pore structure of pastes. For example, Mehta and Gjorv(183) found that although CSF did not decrease the total porosity, CSF paste contained only about 10 per cent pores larger than 0.1 μm while the control contained more than 50 per cent, (Figure 3.58). A comparison of their data for cement pastes cured for 7, 28 and 90-days also showed that the effect of pore refinement became more pronounced over time. This indicates that although pore refinement may initially result from the physical filling of voids with CSF particles, the subsequent chemical or pozzolanic reaction definitely plays a part. Sellevold et al(184) reached the same conclusion from their experiments comparing the effects of CSF with those of the inert filler $CaCO_3$. Mercury intrusion porosimetry data showed, (Figure 3.59), that the $CaCO_3$ reduced the pore size to some degree due to physical effects, but the combination of physical and chemical mechanisms made the CSF more effective in

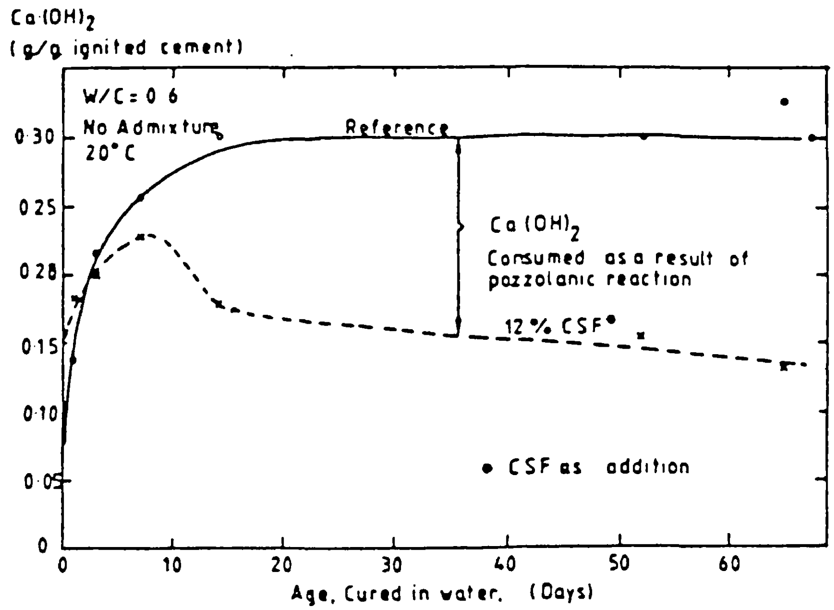


Figure 3.57: Calcium hydroxide contents of a white cement with and without CSF at 20_0C[184].

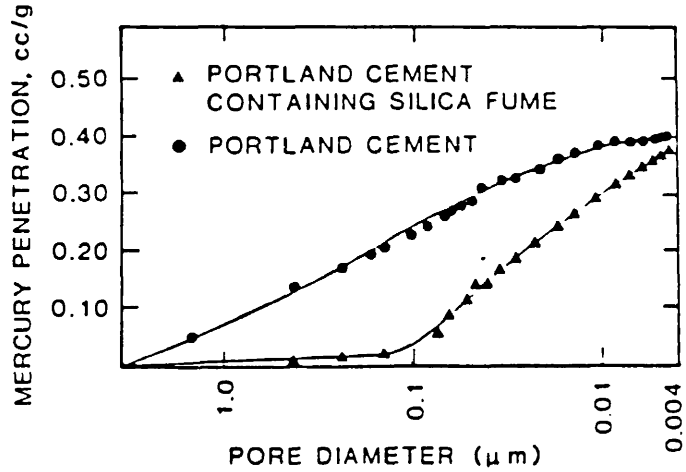


Figure 3.58: Pore size distribution in Portland cement pastes with and without CSF[182].

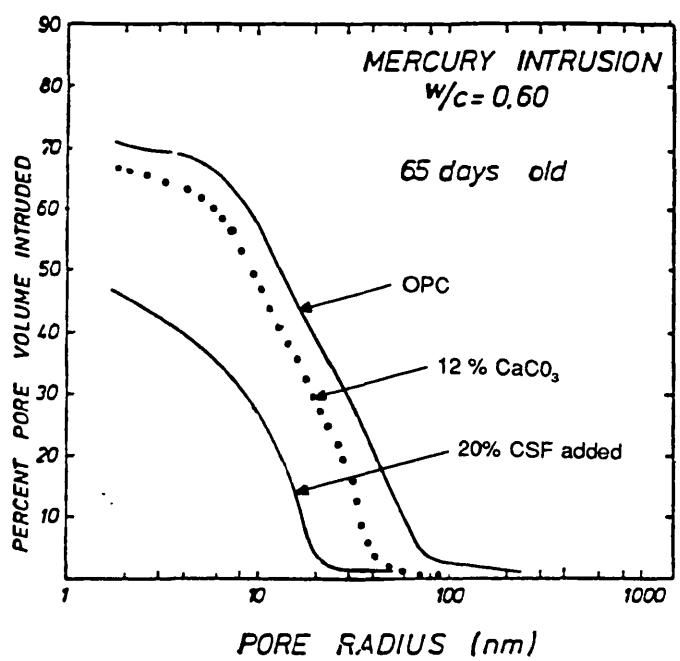
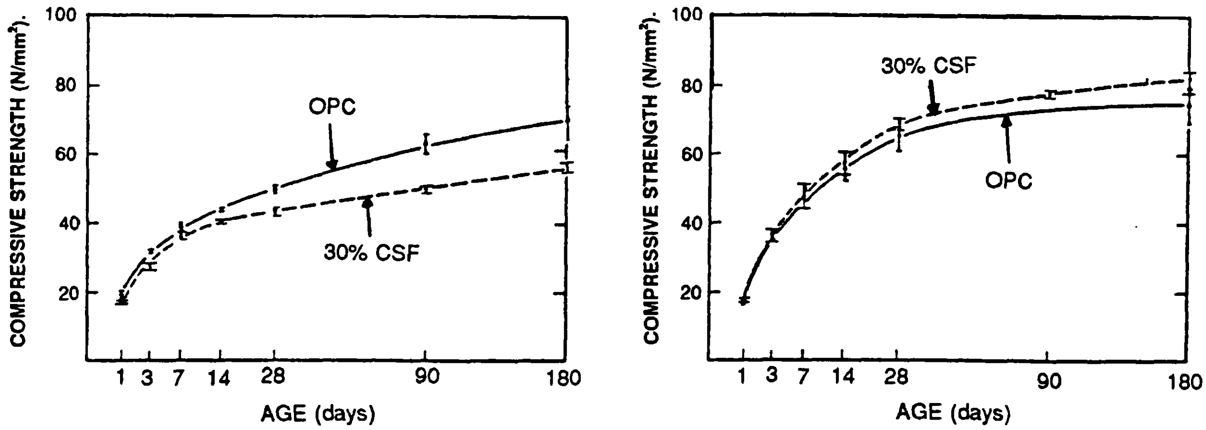


Figure 3.59: Mercury intrusion in mature white cement pastes with and without $CaCO_3$ and CSF[184].

reducing pore size.

Cheng-yi and Feldman(186) investigated the influence of CSF on the strength development of pastes and cement mortars containing 0-30 per cent cement replacement by CSF, (Figure 3.60). This shows that with no CSF the paste is stronger than the mortar throughout, having a strength of 68 N/mm^2, compared to 55 N/mm^2 for mortar, at 180-days. The mixes containing 30 per cent CSF show the reverse trend. After seven days curing, the curves representing mortar and paste begin to diverge; the mean strength of mortar after 180-days curing is 82 N/mm^2 compared to 74 N/mm^2 for paste. They concluded that although the sand is denser and stronger than cement paste, the strength of mortar is lower than that of paste owing partly to a weak sand-cement bond. The addition of CSF to the mortar appeared to improve that bond. A preferential deposition of $Ca(OH)_2$ in the interfacial zone (less than 50 microns) around aggregates in concrete and fibres in paste had been observed.

Scrivener et al(187,188) also reported similar strengths in pastes of similar water-binder ratio, regardless of the CSF content, (Table 3.10). The effect of CSF on the total porosity of the pastes and the paste component of the concretes was similar and so changes to the porosity could not account for the increase in the strength of a CSF concrete over a corresponding OPC concrete. The backscattered electron image analysis results indicated that a major influence of CSF in concrete is associated with the densification of the microstructure at the transition zone, resulting in a much lower porosity. This was quite different from the microstructure of high strength concretes containing no CSF but of similar water-binder ratio. The transition zone of ordinary Portland cement mixes was much more porous than the bulk matrix, even after 180-days of water curing. The influence of the CSF on the transition zone was already apparent at 1-day, indicating that the influence of CSF originates from processes taking place in the early stages of microstructural development, due to the modification of the nature of the fresh concrete, i.e. less bleeding and more compact packing of small CSF particles around the aggregates, thus reducing the formation of water-filled spaces around the aggregates in the fresh concrete. However, the presence of CSF had very little effect on the strength of the concretes at 1-day. This suggests that the improved bonding at the interface, leading to



(a) PASTES

(b) MORTARS

Figure 3.60: Compressive strength of cement pastes and mortars containing 0 and 30 per cent CSF(186).

	Concretes			Pastes	
Silica fume (%)	0	0	15	0	15
Cement content (kg/m^3)	400	495	407	–	–
Water:binder ratio	0·40	0·33	0·33	0·33	0·33
Total porosity at 28 days (volume %)	–	35	38	34	37
Compressive strength at 28 days (N/mm^2)	62·7	77·9	107·6	84·5	85·6

water-reducing effect — inherent effect

Table 3.10: Composition and properties of concretes and cement pastes made with and without CSF(187).

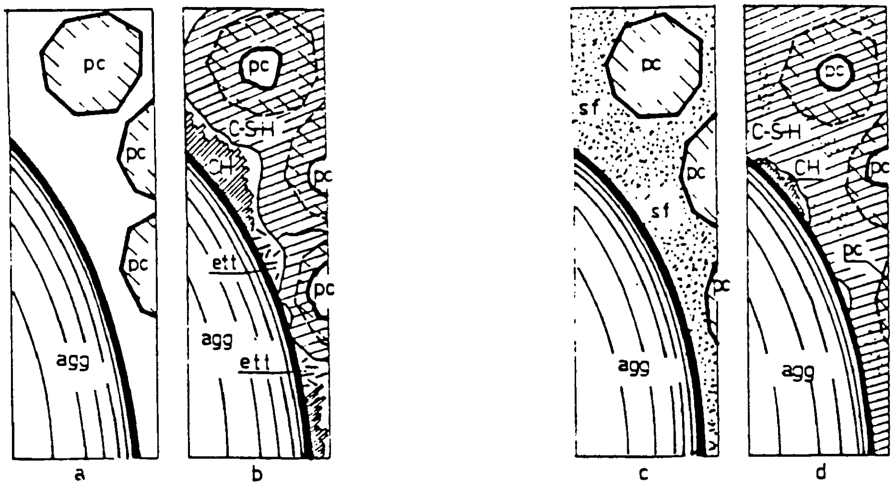
improved strength in the mature concrete, only develops with time as the CSF particles react to produce C-S-H.

Goldman and Bentur[189] provided a schematic description of the effects involved in the formation and reduction in porosity of the transition zone in systems with and without CSF, (Figure 3.61). From this, it can be seen that the effect of the CSF might be divided into two stages. The first is a filler effect leading to reduction in porosity of the transition zone in the fresh concrete and providing the infrastructure needed for a strong transition zone. The second stage is formation of bonds between the densely packed particles in the transition zone, and this is achieved through the pozzolanic reaction. The increase in the magnitude of the inherent effect from 1 to 28-days may be associated with this reaction. The marked advance of the pozzolanic reaction at this time period can be seen readily from the reduction in the $Ca(OH)_2$ content, (Figure 3.62), of the CSF curves. Thus the aggregate-matrix bond improvement induced by CSF is probably the result of a combined filler and pozzolanic effect. It was however difficult to resolve the relative significance of each of the two.

Detwiler and Mehta[180] used carbon black as a mineral admixture in the same dosage as CSF to facilitate the separation of the chemical effects of CSF from the purely physical effects of introducing numerous fine particles into the cement paste system. The carbon black is similar to CSF in terms of particle size characteristics, but it is not pozzolanic. They found that:

1. At early ages, all of the concretes of a given water-binder ratio had essentially the same strength, (Figure 3.63). Apparently the pozzolanic reaction had little effect on the strength of CSF concrete at ages up to 7-days. It was clear that physical mechanisms were primarily responsible for the strength improvement.

2. The carbon black and plain cement mixes showed comparable strengths at both 7 and 28-days, even though the carbon black mixes contained 10 per cent less cement than the comparable plain cement mixes. Apparently,



PC = Portland cement grains,
CSH = calcium silica hydrate gel

SF = CSF particles,
ETT = ettringite.

CH = $Ca(OH)_2$

Figure 3.61: Schematic description of the formation of the transition zone in concretes with and without CSF[(189)].

a) Fresh concrete without CSF showing the water filled space around the aggregate surface, due to bleeding and inefficient cement grain packing at the boundary.
b) Mature system in (a) showing filling of the transition zone with $Ca(OH)_2$, and CSH, and the remnants of porous pockets and zones, some of them filled with needle-like material.
c) Fresh concrete with CSF showing the CSF particles filling the space around the aggregate that was occupied by water in the OPC concrete in (a).
d) Less porous transition zone in the mature system of (c).

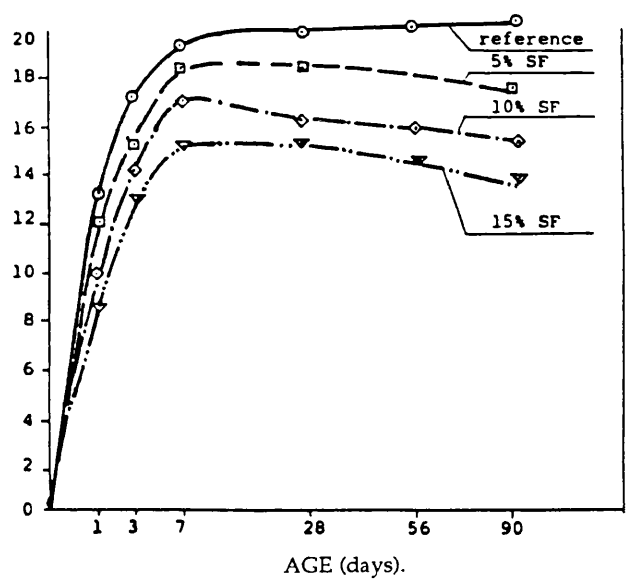


Figure 3.62: $Ca(OH)_2$ contents in pastes with and without CSF[(189)].

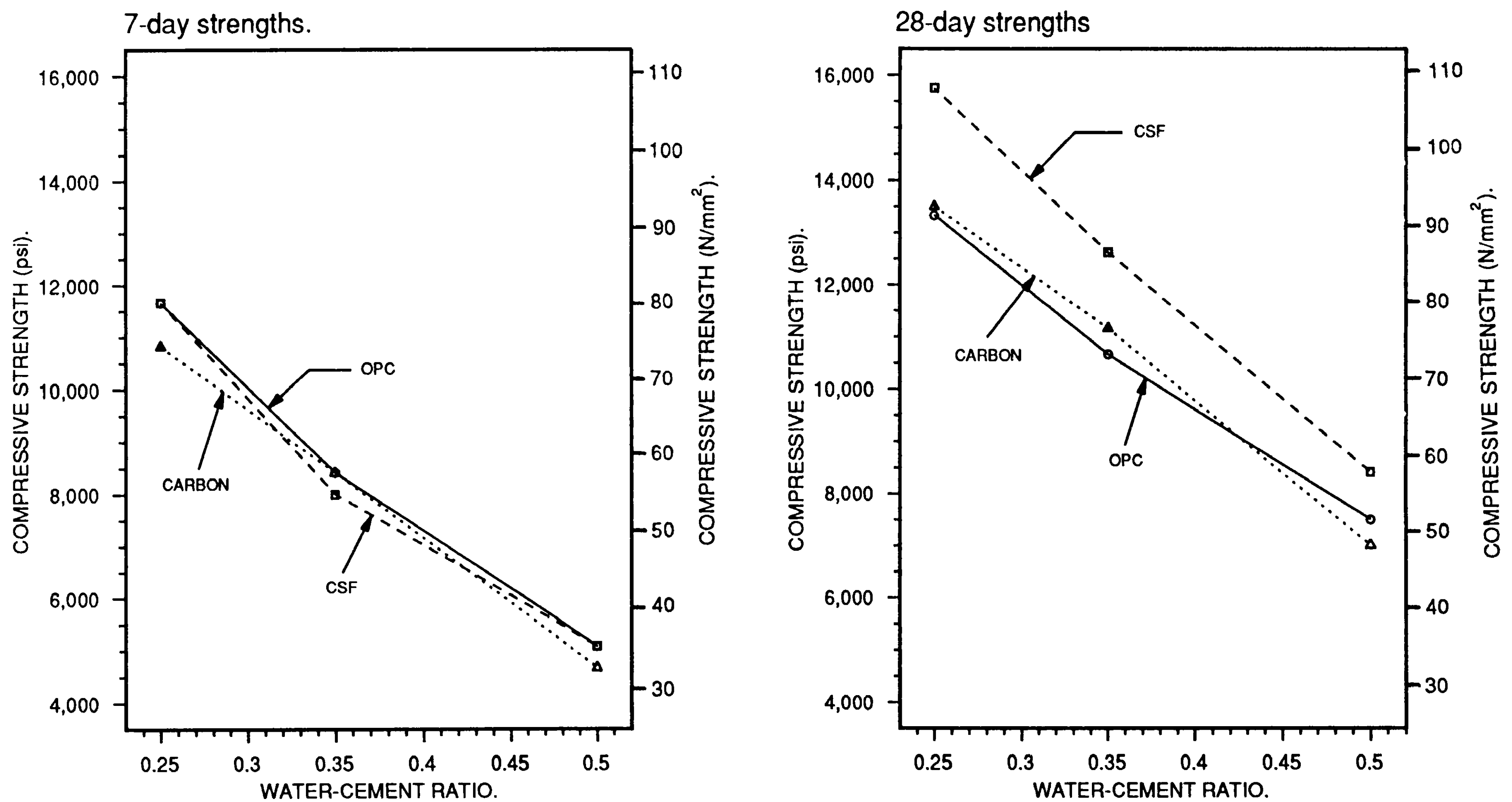


Figure 3.63: Compressive strength versus water-cement ratio for concretes at the ages of 7 and 28-days(180).

grain refinement and/or the improved packing of the hydration products compensated for the reduced cement content.

3. Between the ages 7 and 28-days, the CSF concrete showed the greatest improvement in strength due to the combination of cement hydration and the pozzolanic reaction.

Other investigations indicate that the effect of CSF on strength is either small[190] or negative[132,169], the results being somewhat dependent on the specific binder and superplasticiser that were used[190]. The results of Yogendran et al[169] indicate that the effect of CSF on strength may also be influenced by the water-binder ratio, and the level of cement replacement. For concretes with a water-binder ratio of 0.34, (Figure 3.64), the compressive strength of concrete incorporating CSF increased compared to the control mix up to 15 per cent replacement; subsequently it decreased. However, for concretes with a water-binder ratio of 0.28, the strength at 28, 56, and 91-days of the CSF concretes decreased compared to control concretes even at 5 per cent replacement, (Figure 3.65). Malhotra and Carette[168] also suggested that CSF may perform more efficiently in superplasticised concretes having very high water-binder ratios. They advocated that more supporting data were needed.

There are two methods of using CSF in concrete:

(1) As a partial replacement for cement,

and, (2) as an additive.

The first method has gained a lot of support and resulted in CSF being included as part of the binder since it has been shown to possess cementitious properties in the prescence of $Ca(OH)_2$. Data by Sellevold et al[184], (Figure 3.66), indicate that about 24 per cent of CSF would consume all the $Ca(OH)_2$ produced in a cement paste of water-cement ratio of 0.6. Sarkar and Aitcin[191], however, showed that for low water-binder ratios not all the CSF particles were consumed despite partial cement replacement by CSF of only 6 per cent.

Cheng-yi and Feldman[186] have reported that for pastes containing 30 per cent CSF, the

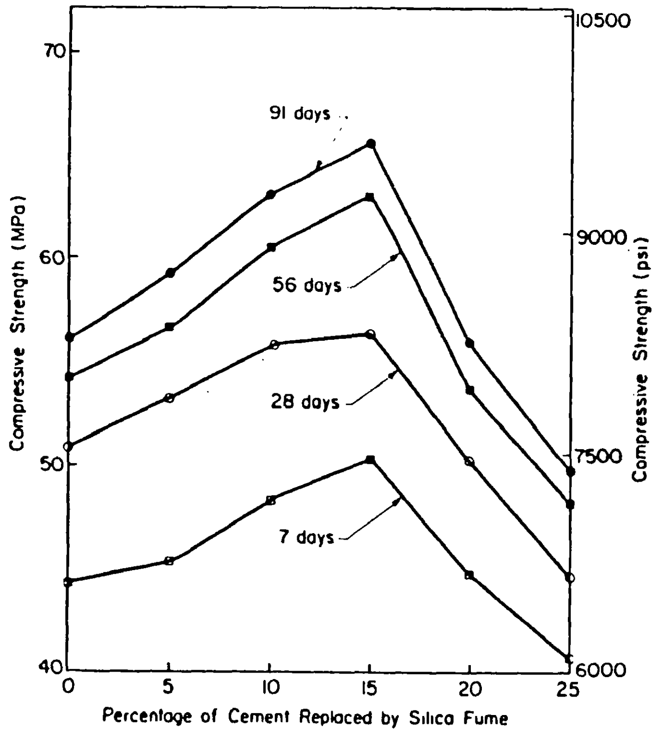


Figure 3.64: Effect of partial cement replacement by CSF on the compressive strength of concrete with water-binder ratio of 0.34[169].

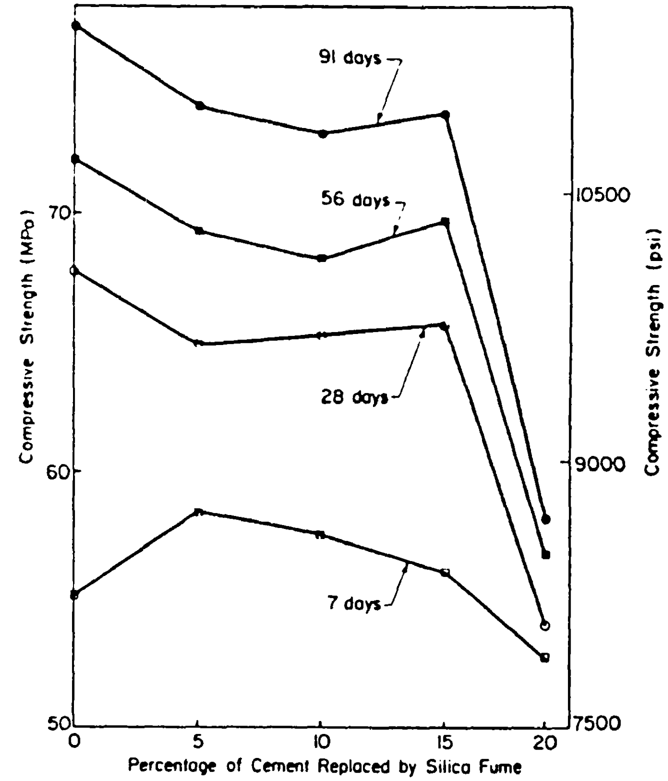


Figure 3.65: Effect of partial cement replacement by CSF on the compressive strength of concrete with water-binder ratio of 0.28[169].

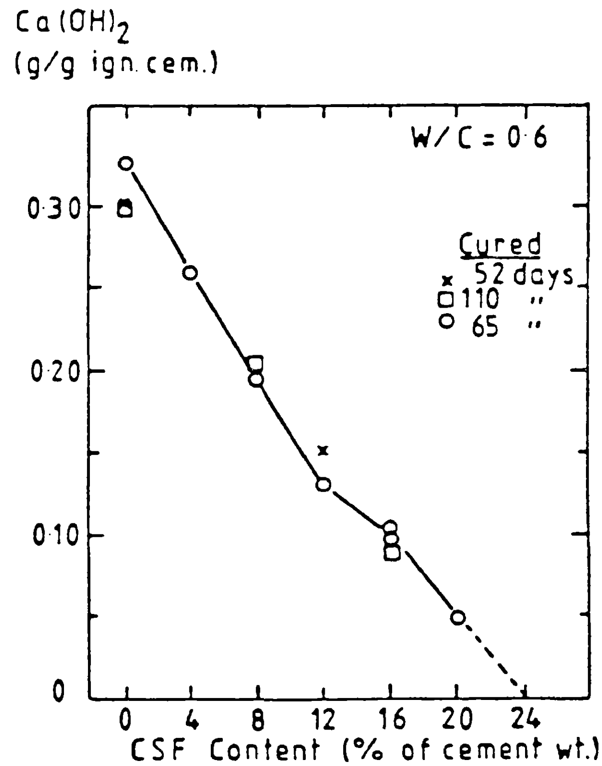


Figure 3.66: Calcium hydroxide contents of mature pastes with various CSF contents(184).

$Ca(OH)_2$ was completely consumed in 14-days. They also observed higher $Ca(OH)_2$ content per gramme of ignited cement at higher water-solids ratios for plain OPC mixes. This probably means there is more $Ca(OH)_2$ available for the pozzolanic reaction at higher water-solids ratio than lower water-solids ratio. It is not therefore surprising that ACI Committee 226[(167)] called for the strength efficiency factors when the water-cement ratio is less than 0.55 to be investigated.

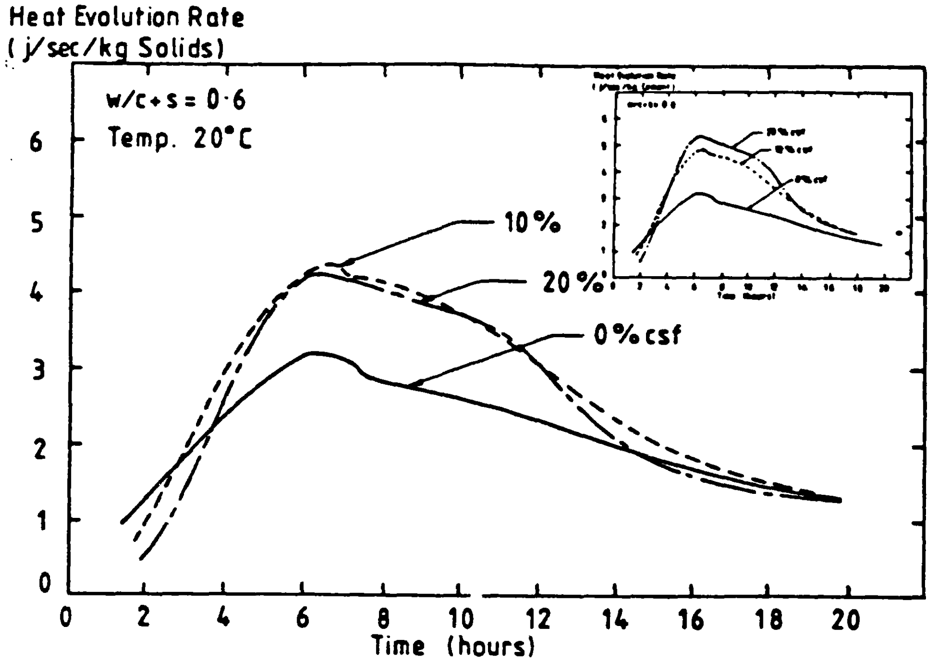
Contradictory results exist about the contribution of CSF to the heat of hydration. Tank[(192)] observed greater heat of hydration with increasing CSF contents in pastes, (Figure 3.67). He attributed the higher heat evolution rate to the enhancement and acceleration of the C_3S hydration resulting from the use of CSF. The acceleration of the cement hydration has also been reported by others[(184,193,194)]. Sellevold and Nilsen[(184)] also reported that adiabatic measurements of the heat development of CSF concrete indicated that the CSF generated 1 to 2 times as much heat as compared to cement. In contradiction to these, Helland[(195)] showed that addition of CSF to concrete with low water-cement ratio contributed very little to the heat of hydration, (Figure 3.68).

It may therefore be concluded that the use of CSF in high strength concrete may result in:

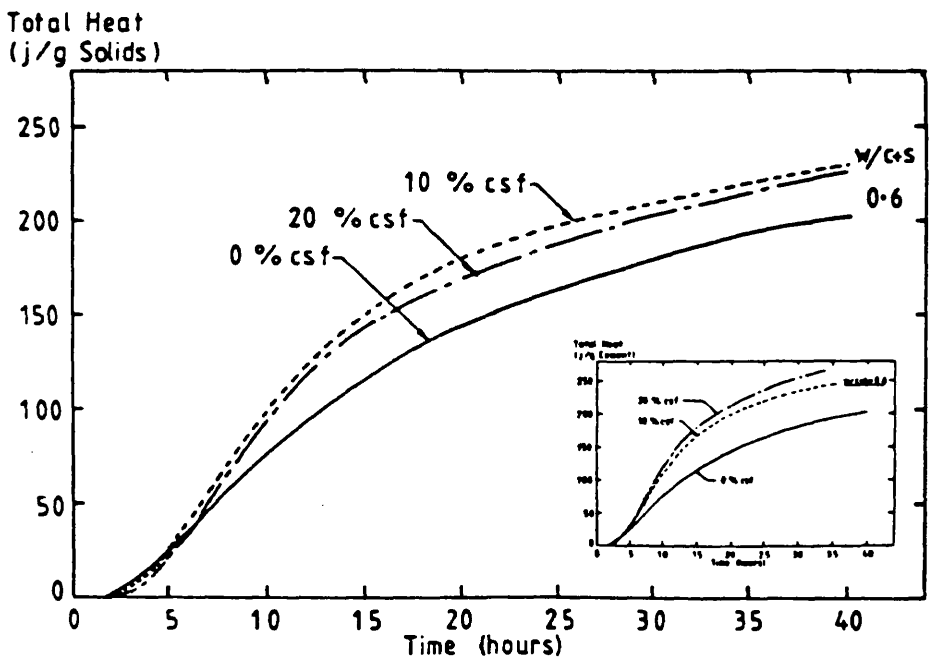
1. A reduction of the workability up to a certain superplasticiser dosage, above which the CSF can have a water reducing effect.

2. Improvement of the compressive strength. This is probably mainly due to the aggregate-matrix bond improvement resulting from the combined filler and pozzolanic effect of CSF. There are however indications that the strength contribution of CSF is affected by the specific binder, superplasticiser, and water-binder ratio. More supporting data are, however, needed before this can be substantiated.

3. Increase in the heat of hydration. However, contradictory results exist, and therefore more data are required on the adiabatic temperature rise of CSF

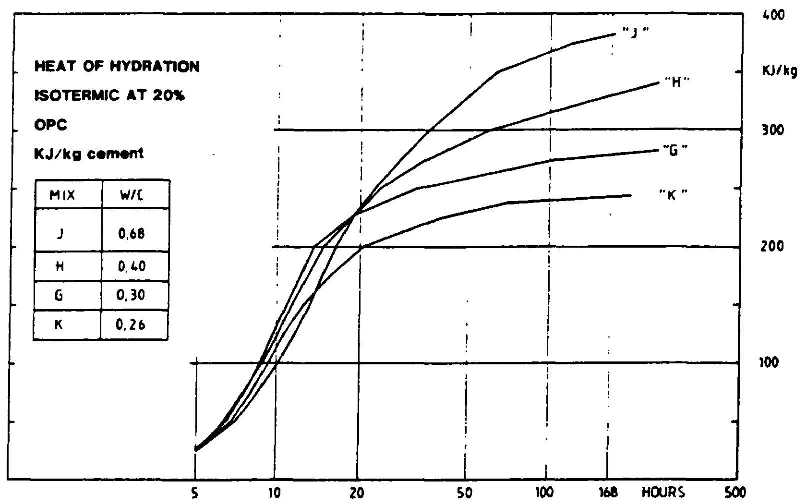


(a) Heat evolution rate

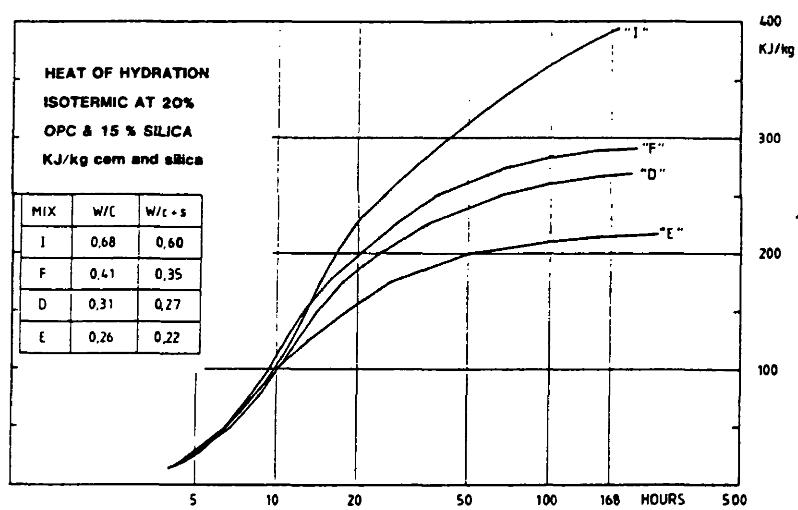


(b) Total heat of hydration

Figure 3.67: Effect of CSF on the hydration of OPC (isothermal calorimetry at $20^0C)$[192].



(a) OPC MIXES.



(b) OPC + 15% CSF MIXES.

Figure 3.68: The effect of water-cement ratio on the heat of hydration of OPC and CSF concrete[195].

concretes.

(IV) Combinations of CSF with PFA or GGBFS.

CSF has been used in combination with PFA, e.g., in the concrete mixture for the Pacific First Center, and with GGBFS for Scotia Plaza, (see Table 2.2). However, there are only limited comprehensive studies on the effects of these combinations on the fresh and hardened properties of concrete, particularly high strength concrete.

Carrette and Malhotra[196] investigated the effect of CSF addition on the strength development of normal strength concrete, i.e. with water-binder ratios higher than 0.40, with 30 per cent Class F (low calcium) PFA, (Figure 3.69). This shows that the use of CSF increased the compressive strength of the concrete at all ages as compared to the control concrete which in this case was a (70% OPC + 30% PFA) mix. They also showed that the compressive strength resulting from the partial replacement of cement by PFA was overcome by the addition of 10 per cent CSF for concretes with water-(cement + PFA) ratios from 0.40 to 0.60, while 15 to 20 per cent was required for higher water-(cement + PFA) ratios.

The mix designs with a combination of GGBFS and CSF that have been used for the construction of Scotia Plaza are shown in Table 2.2. Ryell[61] observed that GGBFS contents up to 30 per cent of the binder gave strength increases in high strength concrete mixes tested at 7-days and later, (Figure 3.70). It was noted, however, that while GGBFS is beneficial to the strength of the concrete at later ages, the lower strengths at early ages with the higher proportions of GGBFS can work to the contractors disadvantage on a fast truck schedule. The lower GGBFS contents, e.g., 10 per cent by mass of total cementitious material, have been used to ensure high in place strengths at early ages for winter concreting operations.

Two other workers[138,157] have also reported on the strength development of concretes made using a combination of GGBFS and CSF. Both Djellouli et al[157] and Tachibana et

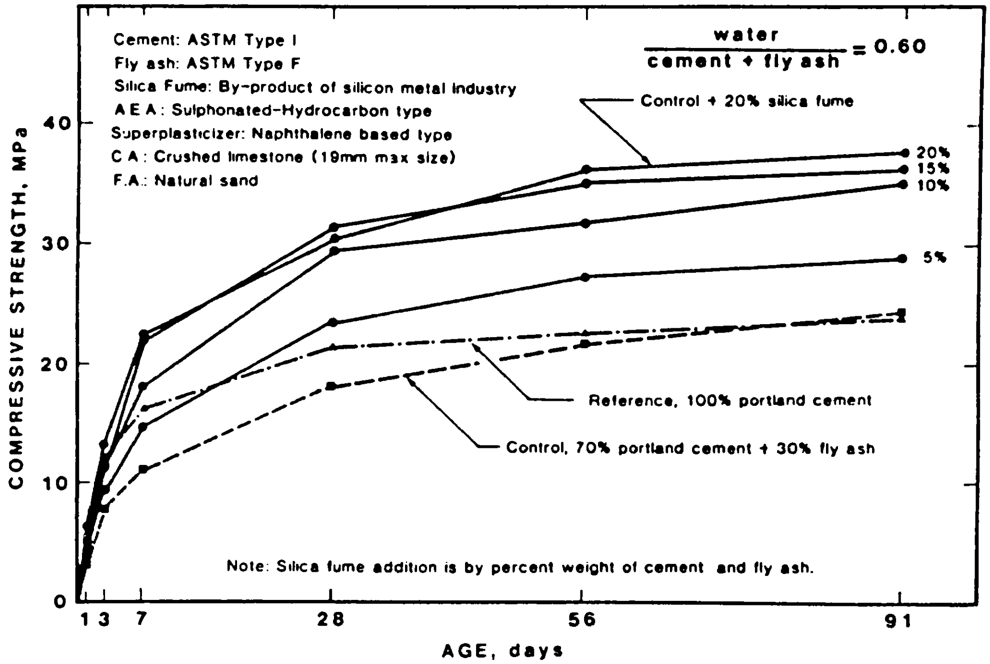


Figure 3.69: The effect of CSF addition on the strength development of PFA concrete[196].

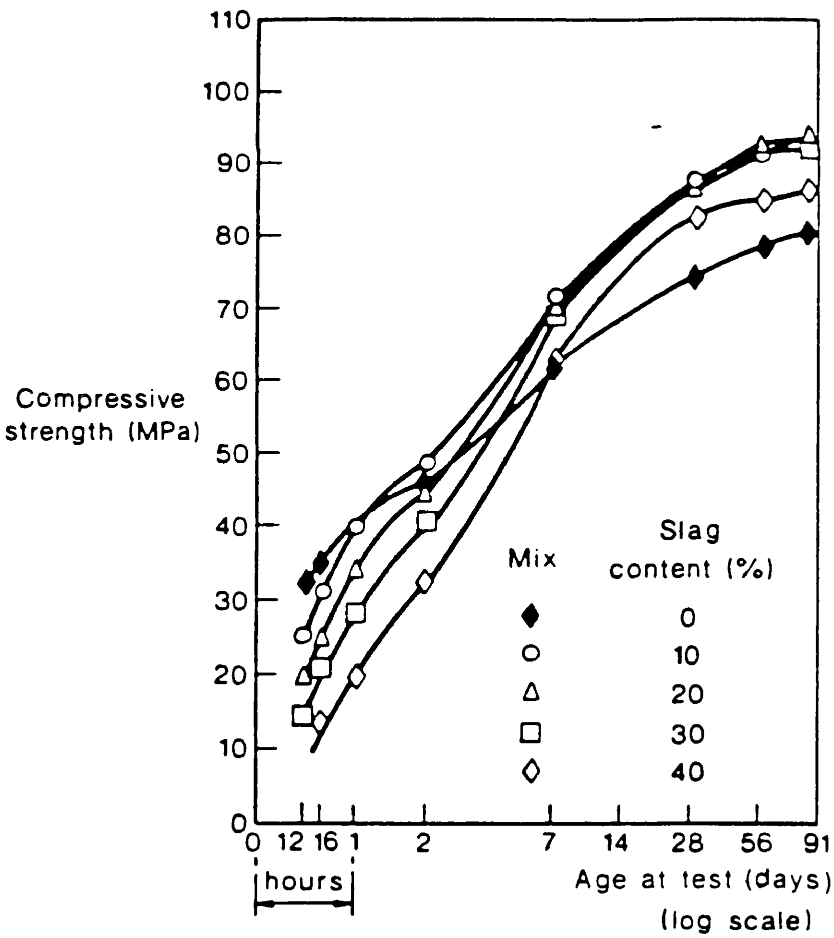


Figure 3.70: Strength development of the CSF/GGBFS used for the construction of Scotia Plaza. CSF was used at the cement replacement level of 8 per cent[61].

al's[138] results, (Figures 3.71 and 3.72a), show that partial cement substitution by GGBFS does not equal the strength of the corresponding CSF concrete even after 91-days. This is in contradiction to Ryell's findings. The only significant difference in the mixes used seems to be the water-binder ratio. Ryell's mixes were with a much higher water-binder ratio than those of the other two workers.

Tachibana et al[138] also investigated the adiabatic temperature rises of the high strength concretes made using combinations of GGBFS and CSF, (Figure 3.72b). It is unfortunate that although they investigated the adiabatic temperature rise of OPC concrete with similar properties, they did not investigate its compressive strength development. For 55 per cent cement replacement by GGBFS in combination with 10 per cent CSF, the adiabatic temperature rise, as compared to CSF concrete, is reduced by 9.8^{0}C, but the 28-day strength is also reduced significantly by 30 N/mm^2.

According to Djellouli et al[157], combinations like PFA/CSF and GGBFS/CSF appear to be promising in the production of high strength concrete from an economical point of view. Better knowledge of their effect on workability, strength development and adiabatic temperature rise is required.

3.2.7 Conclusions.

High strength concrete has been produced using a wide range of quality materials based on the results of trial mixtures. The state of knowledge regarding material selection can be summarised as follows:

1. *Cement:* It has been advocated that the selection of both type and brand of cement are extremely important[8,31,104]. However, it appears that high strength concrete can, in general, be produced with any type of cement when a compatible superplasticiser is added to the mix[1].

2. *Water:* The usual requirement is for the water to be of potable quality[1].

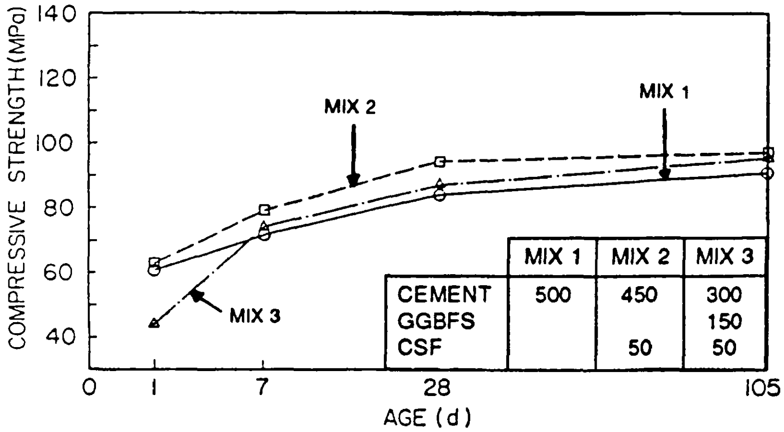


Figure 3.71: Strength development of concretes with water-cement ratio = 0.27[157].

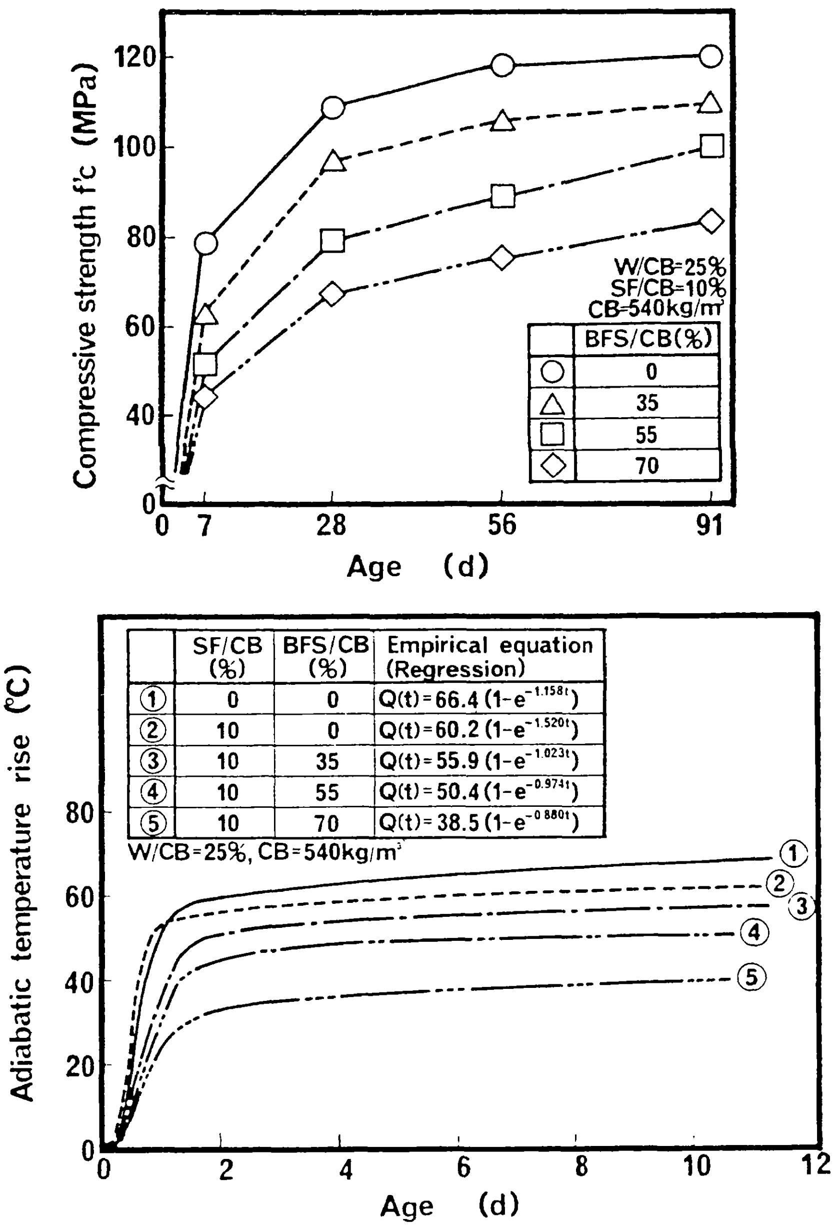


Figure 3.72: Effect of GGBFS on the compressive strength and adiabatic temperature rise of OPC-CSF concrete mixes[138].

3. *Chemical admixtures:* The use of a water-reducer, retarding water-reducer, high range water-reducer, or a combination of these becomes necessary to efficiently use all cementitious materials and to maintain the lowest practical water-cement ratio[1]. Factors to be considered when evaluating an admixture are: cement and mineral admixture compatibility, water reduction, setting times, workability, time of addition, and addition rates. Air-entraining admixtures have not generally been used in North America because of the accompanying strength loss and because the type of application - caissons, interior columns, and shear walls - normally does not require them. However, air-entrainment must be considered for concretes used in highway structures and offshore oil platforms because of their exposure. The question of whether high strength concrete needs a proper air entrained pore system to be frost resistant continues to be a relevant one with conflicting reports in the literature[2]. The conflict concerns laboratory test results and not recorded field performance, of which there is little. The greatest present need is therefore to relate field performance to laboratory test methods.

4. *Coarse aggregate:* Ideal coarse aggregate properties seem mostly to relate to strength, aggregate-mortar bond characteristics and mixing water requirements. The use of a strong, 100 per cent crushed, clean irregular coarse aggregate, with a minimum of flat or elongated particles has been recommended. It has also been recommended that the maximum size should be kept to a minimum, at about 10 mm, since the compressive strength of concrete increases gradually as the maximum size decreases[141,142,143]. There are indications that this effect may be reduced by the use of a superplasticiser, in combination with a good quality crushed aggregate and the use of a mineral admixture[59,105]. More data are however needed before this can be substantiated.

5. *Fine aggregate:* The use of a coarse sand, with an FM of about 3.0, in high strength superplasticised concrete mixes may be advantageous in reducing the "stickiness" and the water demand[11,104,108]. The increased water demand resulting from the use of finer sands can, however, be counteracted in high strength concrete by increasing the superplasticiser dosage. The data are limited and inconclusive.

6. *Mineral admixtures:* The use of a good quality PFA or GGBFS in the production of high strength concrete has been recommended wherever economically feasible. These materials will, in general, improve the workability[151,161,162], contribute to the strength development at late ages (56 to 91-days)[149], and may reduce the temperature rise of normal strength concrete[89,93]. The use of CSF has also been recommended because of the resulting improved strengths. This effect may however be influenced by the binder, superplasticiser[190], water-binder ratio and the level of cement replacement[169]. Although it has generally been accepted that the use of CSF in concrete reduces the workability[167,168,169], there are indications that this effect may be reversed above a certain superplasticiser dosage[173,174]. In general, the technical advantages of PFA, GGBFS, CSF and combinations of these when used in high strength superplasticised concrete mixes require quantification.

Once the materials to be used have been selected there remains the problem of proportioning them to produce concrete having the specified properties. This is discussed in the following section.

3.3 CONCRETE MIX PROPORTIONING.

The proportioning of a concrete mixture is the determination of the quantities of the ingredients which, when mixed together and properly cured, will produce concrete having the desired plastic properties (workability, finishability, etc.) and the required characteristics in the hardened state (strength, durability, etc.) at the lowest possible cost[58]. The heterogeneous nature of concrete and concrete materials creates numerous variables which influence the properties of fresh and hardened concrete. Quoting Cordon[197]:

> "Scientifically trained men of the profession have, in the past, developed sophisticated procedures based on mathematical application of laboratory test results and physical characteristics of the concrete materials. This has resulted in a great deal of mysticism through the years. Much like the

secret recipe of the master chef, the magic formula for proportioning concrete mixes has been sought by one and all."

Some of the methods that have been suggested for design of normal strength concrete mixes are first reviewed so as to provide the historical background that preceded the establishment of guidelines for the design of high strength concrete mixes. Mix proportioning for high strength concrete is then reviewed.

3.3.1 Historical background to mix proportioning for normal strength concrete.

The following are some of the methods that have at some time been suggested for proportioning concrete ingredients.

(I) Nominal proportions.

The widespread use of concrete as a construction material necessitated rationalisation of the mix design and production process and eventually led to the use of mixes of fixed proportions, which generally ensured that concrete of adequate strength was made. These mixes had the virtue of simplicity and, under normal circumstances, had a comforting margin of strength above that specified. Concrete was proportioned by volume, and strength was controlled by varying the cement content. For example, CP 114 (1957)[198] recommended aggregate-cement ratios of 3:1 for a rich mix, 4.5:1 for a medium mix, and 6:1 for a lean mix. However, the following problems were associated with this method of proportioning:

(i) Even an experienced man who knew his materials and could often produce good results with this method, would occasionally fail because this nominal volume method disregarded the water content of the aggregates. In addition, if aggregates were from different sources or if the gradation, shape, or surface texture of the aggregates varied, the characteristics of the resulting concrete would change[199].

(ii) As long as aggregates were batched by volume, proportioning by volume was reasonable; but conversion to weight proportions was required for weigh-batching and this led to ambiguities because of the choice of density measurements available. (A 6:1 mix by volume might be 6.2:1 by weight if the loose bulk density data were used, or 6.7:1 if the compacted bulk density data were accepted.)[200]

(iii) A rather widespread rigidity regarding the fine-coarse aggregate ratio added considerably to the problems associated with nominal mixes, but subsequent relaxation of this ratio of 1:2 led to improvements and allowed a wider range of materials to be used satisfactorily. Fine-coarse ratios from about 1:3 to about 1:1 were frequently accepted, but lack of understanding of the need for this adjustment by very many users of concrete still caused practical difficulties[200].

Nominal proportioning is now limited to small jobs where suitable proportions have been established by experience or observation[199].

(II) Standard mixes.

The publication of CP 116[201] in 1965 introduced a set of mixes which were specified in terms of dry weights of aggregates per bag of cement, which displaced nominal mixes and which allowed flexibility in terms of types of aggregate used, degree of workability chosen and level of quality control exercised on site. These standard mixes avoided the ambiguities of the nominal mixes and were useful "off the shelf" sets of proportions that allowed the desired concrete to be produced with the minimum of preparatory work. However, they were again, by definition, conservative in terms of cement content, and they therefore produced uneconomic concrete stronger than required[200].

(III) Proportioning by maximum density of aggregates.

The requirement that the aggregate occupies as large a relative volume as possible is in the first instance an economic one, the aggregate being cheaper than the cement paste, but there are also technical reasons why too rich a mix is undesirable. These are:

(i) Stickiness and loss of workability will be increased as higher amounts of cement are incorporated into the mixture, as has been explained in Section 2.3.3.

(ii) The maximum temperature desired in the concrete element may limit the quantity or type of cement in the mixture, (c.f. Section 2.3.4).

(iii) The cement content may be limited because of the danger of alkali silica reaction.

(iv) Higher cement contents increase the creep and shrinkage of concrete.

Even ignoring the above reasons, an optimum cement content must be determined; higher cement contents do not necessarily mean higher strength.

Generally accepted theories of proportioning concrete before about 1920, were based on the assumption that the greatest strength and imperviousness of concrete would be obtained with mixtures of maximum density[(197)]. Experimentation with percentages of fine and coarse aggregates to produce the maximum density and therefore the minimum volume of voids resulted in the development of curves such as the one shown in Figure 3.73. It was found, however, that the aggregate proportions that give the maximum density made a somewhat unworkable mix[(199)]. This has been attributed[(117)] to the following two workability requirements that the "Maximum Density Theory" does not consider:

(i) an excess of paste above that required to fill the voids in sand,

and, (ii) an excess of mortar (sand plus cement) above that required to fill the voids in the coarse aggregate.

Methods devised later also denied that maximum density was necessary or even desirable[(197)]. For example, the introduction by Professor Duff Abrams in 1918 of the relation between strength and water-cement ratio[(202)] emphasized the importance of the quality of the paste rather than the density of the concrete. As a result, the method of

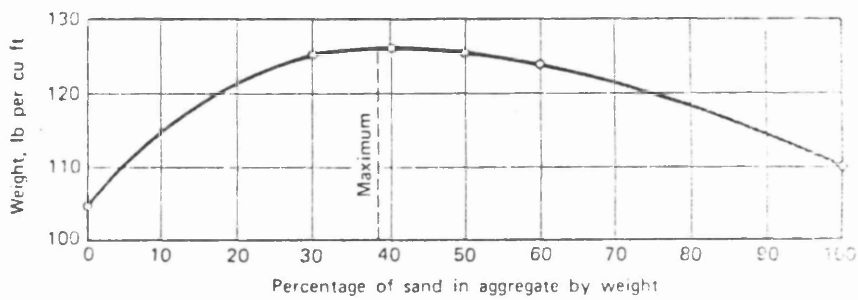


Figure 3.73: Unit weight of dry-rodded fine and coarse aggregates mixed in various proportions. (For illustration only, to show trends; values will differ for different aggregates.)[199].

proportioning by maximum density of aggregates is not used today. However, the basis of the theory, i.e. minimum void content and therefore minimum volume of cement paste, seems an attractive approach to optimising the mix proportions for high strength concrete mixtures, because of the importance of minimising the high cement contents that have generally been used for these. It may be possible to overcome the workability problem by increasing the water-reducer or superplasticiser dosages. This therefore needs further investigation.

(IV) Proportioning by surface area of aggregates.

Since the required amount of cement paste in concrete is affected by the amount of surface to be coated, the surface area of the aggregates seems to be a logical basis for proportioning. Taking for simplicity a sphere of diameter D as representative of the shape of the aggregate we have the ratio of the surface area to volume of 6/D. This ratio of the surface of the particles to their volume, or when the particles have a constant specific gravity, to their weight, is called the "specific surface". For particles of a different shape, a coefficient other than 6/D would be obtained but the surface area is still inversely proportional to the particle size, as shown in Figure 3.74(203).

In the case of graded aggregate, the grading and the overall specific surface are related to one another, although of course there are many grading curves corresponding to the same specific surface. If the grading extends to a larger maximum aggregate size, the overall specific surface is reduced and the water requirement decreases(117), (c.f. page 97). Having chosen the maximum size of aggregate and its grading, we can express the total surface area of the particles using the specific surface as a parameter, and it is the total surface of the aggregate that determines the water requirement or the workability of the mix. Mix design on the basis of the specific surface of the aggregate was first suggested by Edwards(204) as far back as 1918.

The application of surface area calculations was, however, found to break down for aggregate particles passing the 150 μm sieve, and for cement. These particles, and also

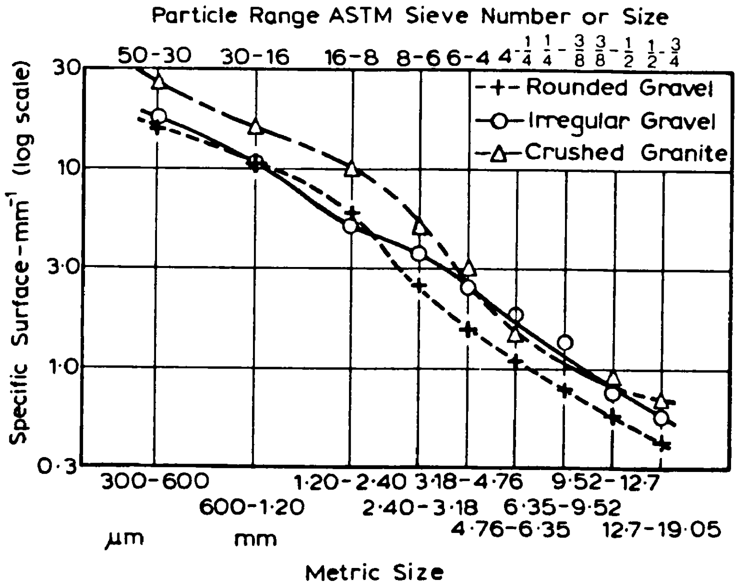


Figure 3.74: Relation between specific surface and particle size(203).

Particle size fraction	ASTM sieve No.	Relative surface area	Murdock's surface index[3.19]
76·2–38·1 mm	3–1½ in.	½	½
38·1–19·05 mm	1½–¾ in.	1	1
19·05–9·52 mm	¾–⅜ in.	2	2
9·52–4·76 mm	⅜–$^3/_{16}$ in.	4	4
4·76–2·40 mm	$^3/_{16}$ in.–8	8	8
2·40–1·20 mm	8–16	16	12
1·20 mm–600 µm	16–30	32	15
600–300 µm	30–50	64	12
300–150 µm	50–100	128	10
smaller than 150 µm	smaller than 100		1

Table 3.11: Relative values of surface area and surface index(205).

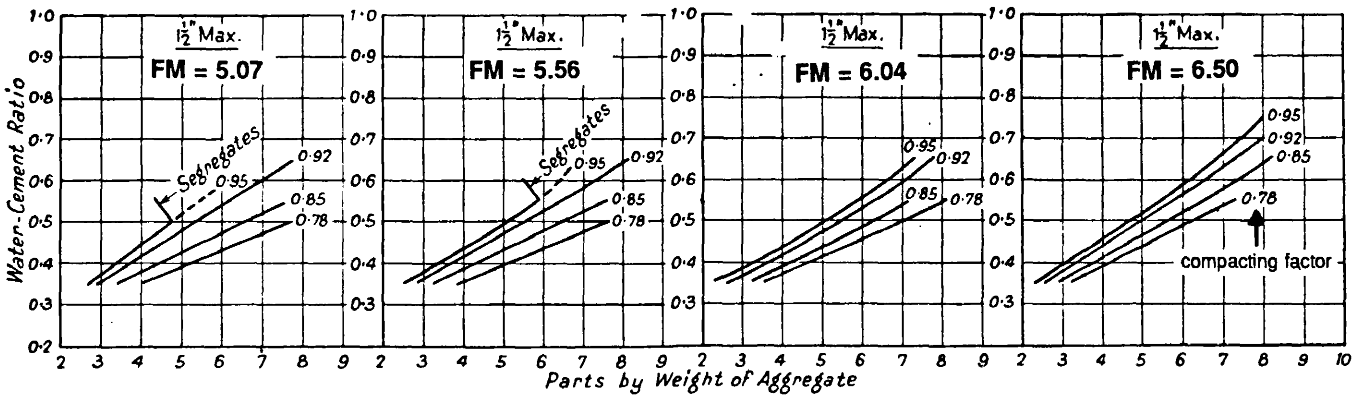
some larger sand particles, appear to act as a lubricant in the mix and do not seem to require wetting in quite the same way as coarse particles[117], (c.f. the effect of the proportion of PFA particles smaller than 45 μm on the amount of water required for a given degree of workability, page 116). Because of this the specific surface overestimates the effect of fine particles and gives a somewhat misleading picture of the workability to be expected. An empirical surface index was therefore suggested by Murdock[205] and its values as well as those of the specific surface are given in Table 3.11. The overall effect of the surface area of an aggregate of given grading is obtained by multiplying the percentage weight of any size fraction by the corresponding coefficient, and summing all the products. According to Murdock, this surface index should further be modified by an angularity index, which is again empirically determined.

A simpler index number, that has been more widely used, is the fineness modulus (FM), which, as has been mentioned on page 108, is roughly proportional to the average particle size of a given aggregate; that is, the coarser the aggregate, the greater the FM. Charts were developed to show the relationship between water-cement ratios, mix proportions by weight, and the workability as measured, for example, by the compacting factor. Figure 3.75[206] shows these relationships for each of four FMs and two maximum aggregate sizes. Similar charts were developed for other types of aggregates such as, for example, crushed stone. The charts are useful, not only in deciding the proportions of a mix for a certain water-cement ratio, but also for assessing the effect of grading on the workability of the mix.

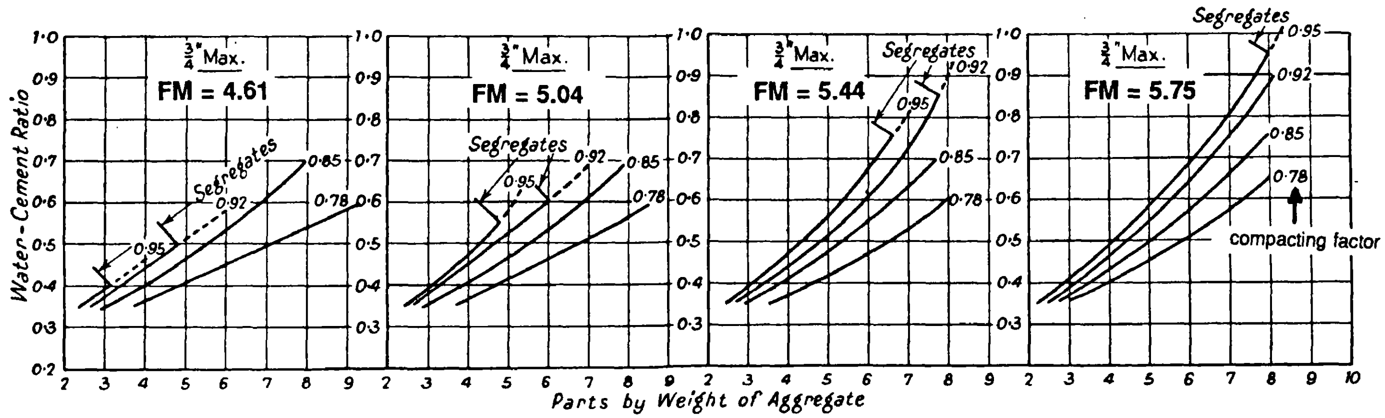
It seems then that the grading and therefore the surface area of the aggregate is an important factor in determining the workability of the mix. However, the design of the mixes on the basis of the specific surface of the aggregate is not universally recommended because:

(i) although the specific surface area can be determined using a water permeability method[207], no simple field test is available, and a mathematical approach is made difficult by the variability in the shape of different aggregate particles[117].

(ii) the behaviour of "semi-liquid" mixture of granular materials is still



(a) 1-1/2 in. (38.1 mm) maximum aggregate size.



(b) 3/4 in. (19.1 mm) maximum aggregate size.

Figure 3.75: Relationships between water-cement ratios, mix proportions by weight, and the workability as measured by the compacting factor for aggregate mixtures with various fineness moduli[206].

imperfectly understood. Especially, the exact role played by the finer particles has by no means been ascertained[117].

(iii) this method does not consider the fact that the amount of paste required in concrete is also affected by the number of voids to be filled[199].

(V) Proportioning by voids-cement ratio and mortar voids.

In 1922, Talbott and Richart[199] published a paper explaining a new way of concrete proportioning called the voids-cement ratio method, and presenting a mortar-voids system for applying this method. Spaces not occupied by aggregate or cement were considered voids; their volume was the sum of the mix's water and entrapped air. The voids-cement ratio, expressed in cubic feet of voids per hundredweight, was approximately proportional to the water-cement ratio because of the relatively insignificant amount of space occupied by air in concrete.

To produce concrete of a particular strength, a voids-cement ratio for this strength must be determined. This process involved preparing a number of trial mortar mixes. Several batches were blended, each with a different ratio of sand to cement. Water was added gradually to each, and the volume of voids for each different proportion of sand, cement, and water was determined. This step was accomplished by weighing a known volume of mortar, using the specific gravities of the constituents of the mortar and the known weight and volume of the sample produced, to calculate the volume of voids. Each resulting sample was subsequently tested for strength.

The test produced raw data from which four different curves could be drawn for each water content. These were:

1. the relationship of strength to the voids-cement ratio,
2. the voids-cement ratio to the sand-cement ratio,
3. the sand-cement ratio to the voids-per-unit volume of mortar,

and, 4. the sand-cement ratio to the water-per-unit volume of mortar.

Once these curves had been developed, a final estimate of proportions could be made.

First, it was necessary to assume, from previous experience, the amount of water needed to produce the desired consistency or slump in the concrete. Referring to the series for that water content, proportioning for the desired concrete strength could begin. Knowing this, determination of the voids-cement ratio from Curve No. 1 was possible. Once the voids-cement ratio was established, the sand-cement ratio could be determined from Curve No. 2. With the sand-cement ratio known, the voids-per-unit volume of mortar could be determined from Curve No. 3, and the water-per-unit volume of mortar from Curve No. 4. At this point, the amount of water-per-unit volume of mortar and the sand-cement ratio were known, and the volume of sand and cement, both per unit volume of mortar, could be established. The only unknown remaining was the volume of coarse aggregate to be used in a unit volume of concrete. This amount could only be arrived at by trial or from past experience. The "voids-cement ratio" method of proportioning was complicated and involved the preparation of a large number of trial mixes and the amount of coarse aggregate was still subject to trial or experience. For these reasons, the voids-cement ratio and the mortar-voids methods were used very little outside of the laboratory. The research done by Talbott and Richard in developing this method, however, increased the general knowledge of concrete qualities and behaviour. For instance, the voids-cement ratio to strength curves explain air-entrained concrete's lower strength, despite controlled, constant water-cement ratio.

(VI) Proportioning by void content of coarse aggregate.

In 1942, National Crushed Stone Association researchers Goldbeck and Gray(199), building on the original work of Talbott and Richart, published a method of proportioning based on void content of coarse aggregate. Although this method is no longer used, it included a table of recommended bulk volumes of coarse aggregate per unit volume of concrete when the FM of sand and the maximum size of coarse aggregate were known, (Table 3.12). This table, applicable to all cement contents and to crushed or rounded aggregates, was later incorporated into the ACI standard as the absolute volume method of proportioning, which is now described.

Maximum size of aggregate, in.	Volume of dry-rodded coarse aggregate* per unit volume of concrete for different fineness moduli of sand			
	2.40	2.60	2.80	3.00
3/8	0.50	0.48	0.46	0.44
1/2	0.59	0.57	0.55	0.53
3/4	0.66	0.64	0.62	0.60
1	0.71	0.69	0.67	0.65
1 1/2	0.75	0.73	0.71	0.69
2	0.78	0.76	0.74	0.72
3	0.82	0.80	0.78	0.76

*Volumes are based on aggregates in dry-rodded condition as described in ASTM C 29 for Unit Weight of Aggregate.

These volumes are selected from empirical relationships to produce concrete with a degree of workability suitable for usual reinforced construction. For less workable concrete such as required for concrete pavement construction they may be increased about 10 percent. When placement is to be by pump, they should be reduced about 10 percent.

Table 3.12: Recommended volume of coarse aggregate per unit volume of concrete, (ACI 211.1-89)[48].

(VII) Development of ACI Standard and Building Research Establishment's report for proportioning normal concrete mixes.

The American Concrete Institute, being responsible to all organisations of the concrete profession and industry in the USA, was obligated to formulate a standard method for proportioning concrete mixtures. The first successful attempt to produce an ACI standard was completed by Committee 613 in 1944, about 24 years after the relation between strength and water-cement ratio was introduced by Professor Duff Abrams(202). Complete agreement was never achieved among the committee members, but after much discussion and compromise, ACI-613-44(208) was published. Some members of the committee never agreed and insisted on a minority report in the discussion. In 1954 the report was revised under the chairmanship of Walter H. Price(209). This included the use of air-entrainment and the dry rodded volume of coarse aggregate per unit volume of concrete concept for estimating coarse aggregate content, (Table 3.12). The fact that there were no major revisions of the basic concepts of that original report is a tribute to the thoroughness of the work of the original committee. After serving the institute for 17 years ACI Standard 613 was replaced by ACI Standard 211.1(210) in 1970, which again did not change the basic concepts of the original committee report. This has also undergone revisions(211-217), the most notable one being in 1981(213), which added recommendations on the use of PFA, GGBFS and chemical admixtures. Recently, in 1991, ACI Committee 211 also proposed to include proportioning with CSF(217).

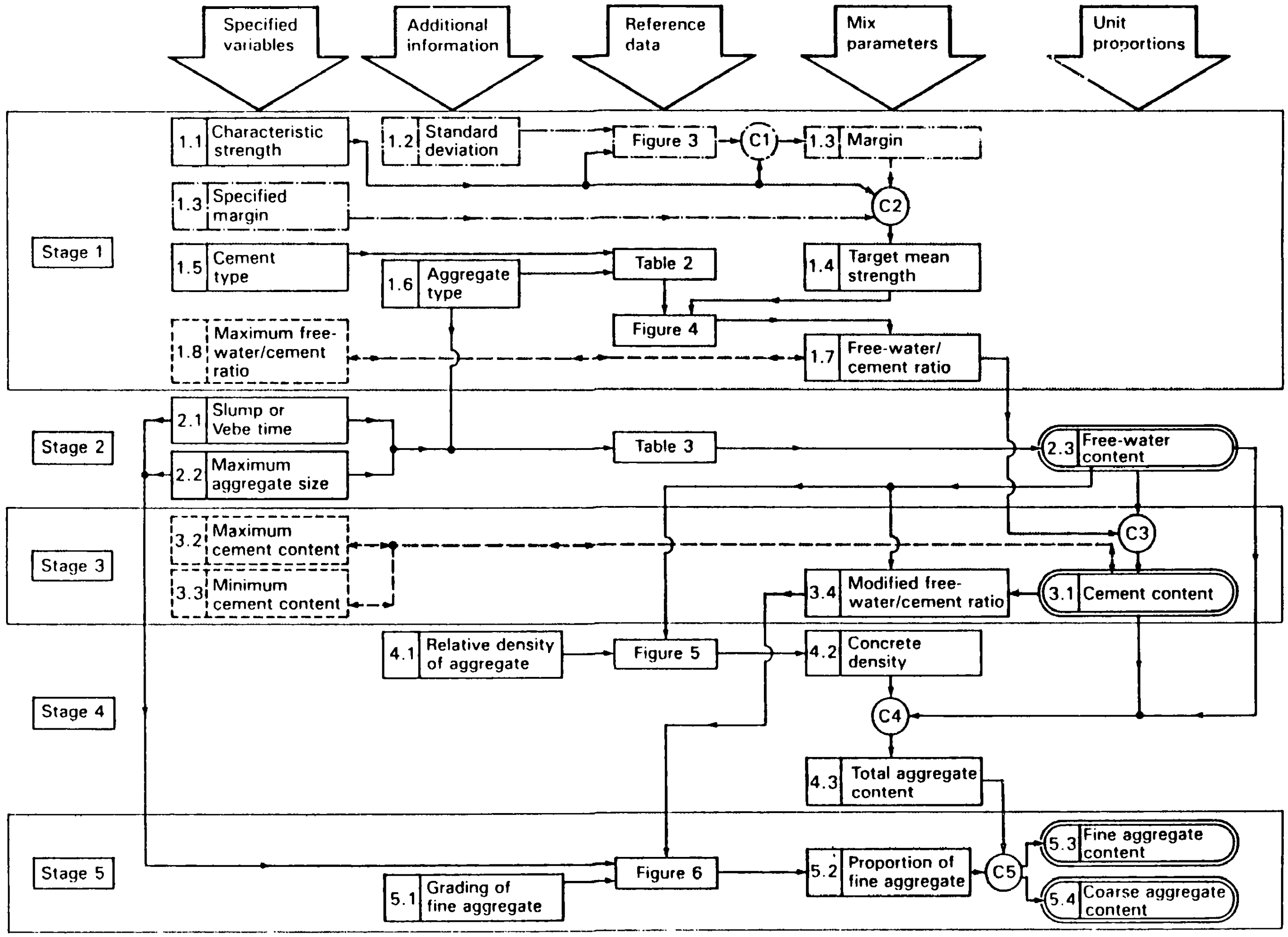
In Britain, the standard method for proportioning concrete mixtures was first published in 1950 and was known as "Road Note No. 4: Design of Concrete Mixes"(218). It was revised in 1975(219) and, more recently, in 1988 and has been published by the Building Research Establishment, the title having been changed to: "Design of Normal Concrete Mixes"(47).

The current ACI and BRE procedures follow approximately similar steps leading to the trial mix proportions. They both provide an estimation of the correct mix proportions based upon compilations of a large number of published data, referred to as "reference data", which appear in the form of figures or tables, and upon a knowledge of the properties of the aggregate to be used, which may involve preliminary testing. The stages of the British method for concrete mixtures proportioning can be outlined as:

Stage 1:	deals with strength leading to the free water-cement ratio.
Stage 2:	deals with workability leading to the free water content.
Stage 3:	combines the results of Stages 1 and 2 to give the cement content.
Stage 4:	deals with the determination of the total aggregate content.
Stage 5:	deals with the selection of the fine and coarse aggregate contents.

These stages are also shown in Figure 3.76[47], showing in more detail the required flow process. Stages 4 and 5 are the ones that differ from one procedure to another, and are therefore now described in more detail:

(i) *Road Note No. 4 method*[218]*:* Tables, such as Table 3.13, are used in Stage 4 to determine the aggregate-cement ratio required to give four degrees of workability, with predefined combined coarse-fine aggregate gradings, shown in Figure 3.77. Having chosen the aggregate-cement ratio, the next step is to fix the ratio of fine to coarse aggregate. This is linked with the question of stability. Since this property depends upon the amount and fluidity of the mortar, low cement contents need more sand, and high cement contents need less sand. With high water contents and therefore high workability, in a lean mix the mortar will be too fluid unless more sand is added; with a dry mix less sand is required. Curve No. 1 represents the coarsest grading which is comparatively workable and can, therefore, be used for mixes with a low water-cement ratio or for rich mixes; it is, however, necessary to make sure that segregation does not take place. At the other extreme, Curve No. 4 represents a fine grading; it will be cohesive but not very workable. In particular, an excess of material between 1.20 and 4.76 mm (No. 16 and 3/16 in.) test sieves will produce a harsh concrete, which, although it may be suitable for compaction by vibration, is difficult to place by hand. If the same workability is to be obtained using aggregates with grading Curves Nos. 1 and 4, the latter would require a considerably higher water content; this would mean a lower strength if both concretes are to have the same aggregate-cement ratio or, the concrete made with the



Items in dashed boxes and with two-way arrows are optional limiting values that may be specified. C = calculation
Items in chain-dotted boxes are alternatives

Figure 3.76: Flow chart of mix design procedure, (Building Research Establishment's report)[47].

Degree of workability		Very low				Low				Medium				High			
Grading curve No.		1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
Water/cement ratio by weight	0·35	4·0	3·9	3.5	3·2	3·4	3·3	3·2	2·9	2·9	2·8	2·6	2·5	2·7	2·5	2·3	2·3
	0·40	5·3	5·3	4·7	4·3	4·5	4·5	4·2	3·8	3·8	3·8	3·7	3·4	3·5	3·5	3·3	3·1
	0·45	6·5	6·5	5·9	5·3	5·6	5·6	5·3	4·8	4·6	4·7	4·6	4·3	4·1	4·4	4·3	4·0
	0·50	7·7	7·7	7·1	6·3	6·7	6·6	6·3	5·7	5·4	5·7	5·5	5·1	4·8	5·2	5·1	4·8
	0·55	–	–	8·1	7·3	7·6	7·6	7·2	6·6	6·2	6·5	6·3	5·8	×	5·9	6·0	5·5
	0·60			–	–	–	–	–	7·4	7·0	7·3	7·1	6·6	×	×	6·7	6·2
	0·65								8·1	7·8	8·1	7·8	7·2	×	×	7·3	6·9
	0·70								–	–	–	–	7·9	×	×	–	7·4
	0·75												–	×	×	–	8·0
	0·80													×	×	–	–

– Indicates that the mix was outside the range tested.
× Indicates that the mix would segregate.
These proportions are based on specific gravities of approximately 2·5 for the coarse aggregate and 2·6 for the fine aggregate.

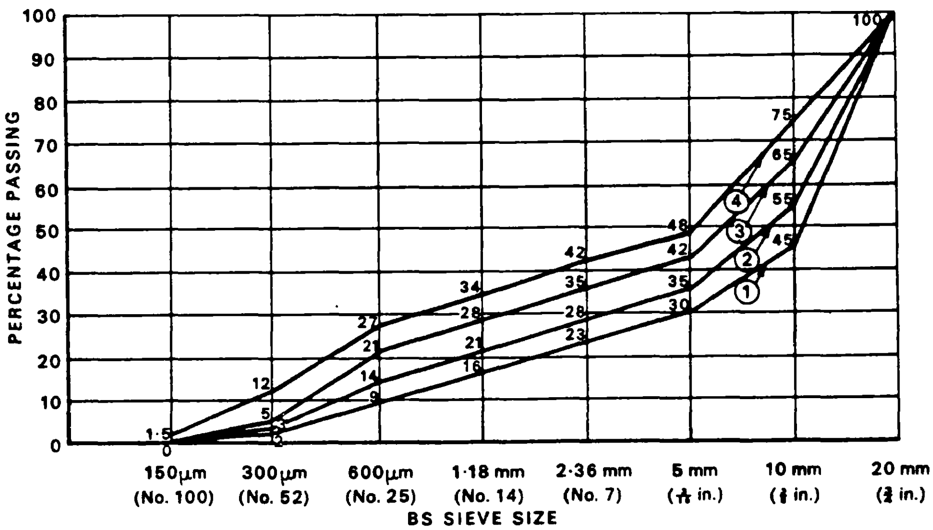
(a) Irregular aggregate.

Degree of workability		Very low				Low				Medium				High			
Grading curve No.		1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
Water/cement ratio by weight	0·35	4·5	4·5	3·5	3·2	3·8	3·6	3·2	3·1	3·1	3·0	2·8	2·7	2·8	2·8	2·6	2·5
	0·40	6·6	6·3	5·3	4·5	5·3	5·1	4·5	4·1	4·2	4·2	3·9	3·7	3·6	3·7	3·5	3·3
	0·45	8·0	7·7	6·7	5·8	6·9	6·6	5·9	5·1	5·3	5·3	5·0	4·5	4·6	4·8	4·5	4·1
	0·50	—	—	8·0	7·0	8·2	8·0	7·0	6·0	6·3	6·3	5·9	5·4	5·5	5·7	5·3	4·8
	0·55	—	—	—	8·1	—	—	8·2	6·9	7·3	7·3	7·4	6·4	6·3	6·5	6·1	5·5
	0·60	—	—	—	—	—	—	—	7·7	—	—	8·0	7·2	×	7·2	6·8	6·1
	0·65					—	—	—	8·5	—	—	—	7·8	×	7·7	7·4	6·6
	0·70					—	—	—	—	—	—	—	—	×	—	7·9	7·2
	0·75													×	—	—	7·6
	0·80													×	—	—	—
	0·85													×	—	—	—
	0·90																

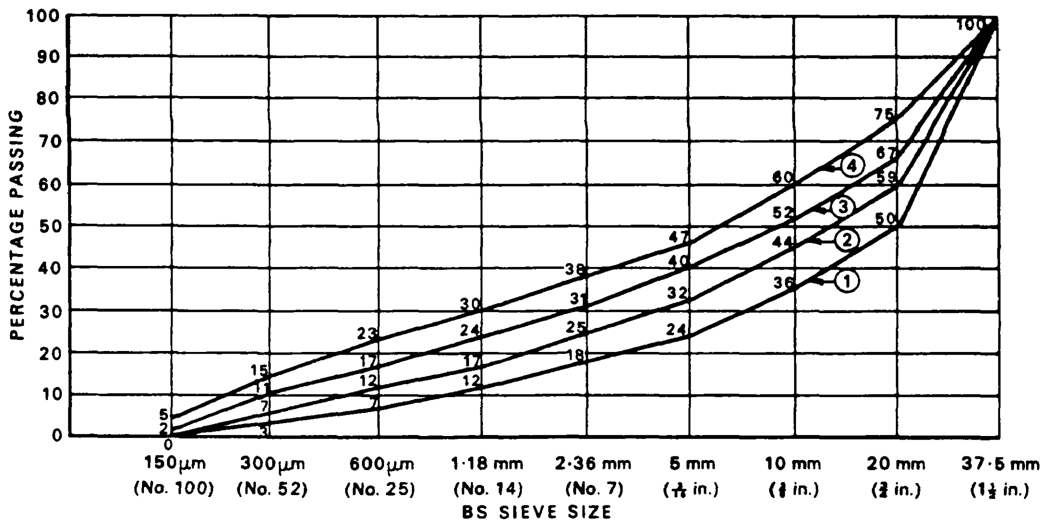
— Indicates that the mix was outside the range tested.
× Indicates that the mix would segregate.
These proportions are based on specific gravities of approximately 2·5 for the coarse aggregate and 2·6 for the fine aggregate.

(b) Rounded aggregate.

Table 3.13: Aggregate-cement ratio (by weight) required to give four degrees of workability with different aggregate gradings(218).



(a) 3/4 in. (19.1 mm) maximum aggregate size.



(b) 1.5 in. (38.1 mm) maximum aggregate size.

Figure 3.77: Road Note No. 4[218] type grading curves.

fine aggregate would have to be considerably richer, i.e. each cubic metre would contain more cement than when the coarser grading is used(117).

The change between the extreme gradings is progressive. In the case of gradings lying partly in one zone, partly in another, i.e. when too many intermediate sizes are missing, there is a danger of segregation. If, on the other hand, there is an excess of middle-sized aggregate the mix will be harsh and difficult to compact by hand and possibly even by vibration. For this reason, it is preferable to use aggregate with gradings similar to type rather than totally dissimilar ones(117).

In practice, the use of separate fine and coarse aggregate means that a grading can be made up to conform exactly with a type grading at one intermediate point, generally the 5 mm (3/16 in.) size. Good agreement can usually also be obtained at the ends of the curve, i.e. at 150 um (No. 100) sieve and the maximum size used. If coarse aggregate is delivered in single-size fractions, as is usually the case, agreement at additional points above 5 mm (3/16 in.) can be obtained, but for sizes below this blending of two or more sands is necessary(117).

(ii) *The BRE method(47):* Stage 4 in the more recent BRE method(47) requires an estimate of the density of the fully compacted concrete which is obtained from Figure 3.78. Using this the total aggregate content is determined from the equation:

$$\text{Total aggregate content} = D - C - W$$

where: D = the wet density of the concrete (kg/m^3),

C = the cement content (kg/m^3),

and, W = the free-water content (kg/m^3).

Stage 5 involves deciding how much of the total aggregate should consist of the sand. Figure 3.79 shows recommended values for the proportion of fine aggregate depending on the maximum size of aggregate, the workability level, the grading of the fine aggregate and the free water-cement ratio. Unlike the Road Note No. 4, the BRE method does not use combined aggregate grading curves. While the 1975 edition(219) used the grading zones for fine aggregate given in BS 882: 1973(220), the

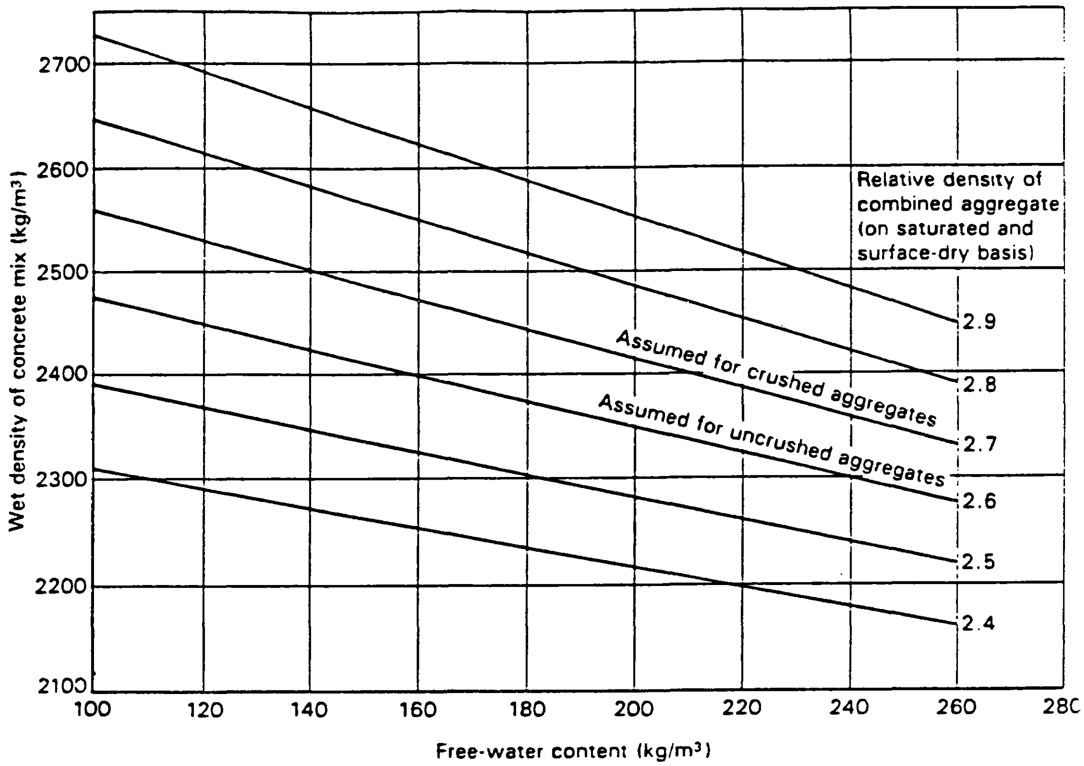


Figure 3.78: Estimated wet density of fully compacted concrete (Building Research Establishment's report)[47].

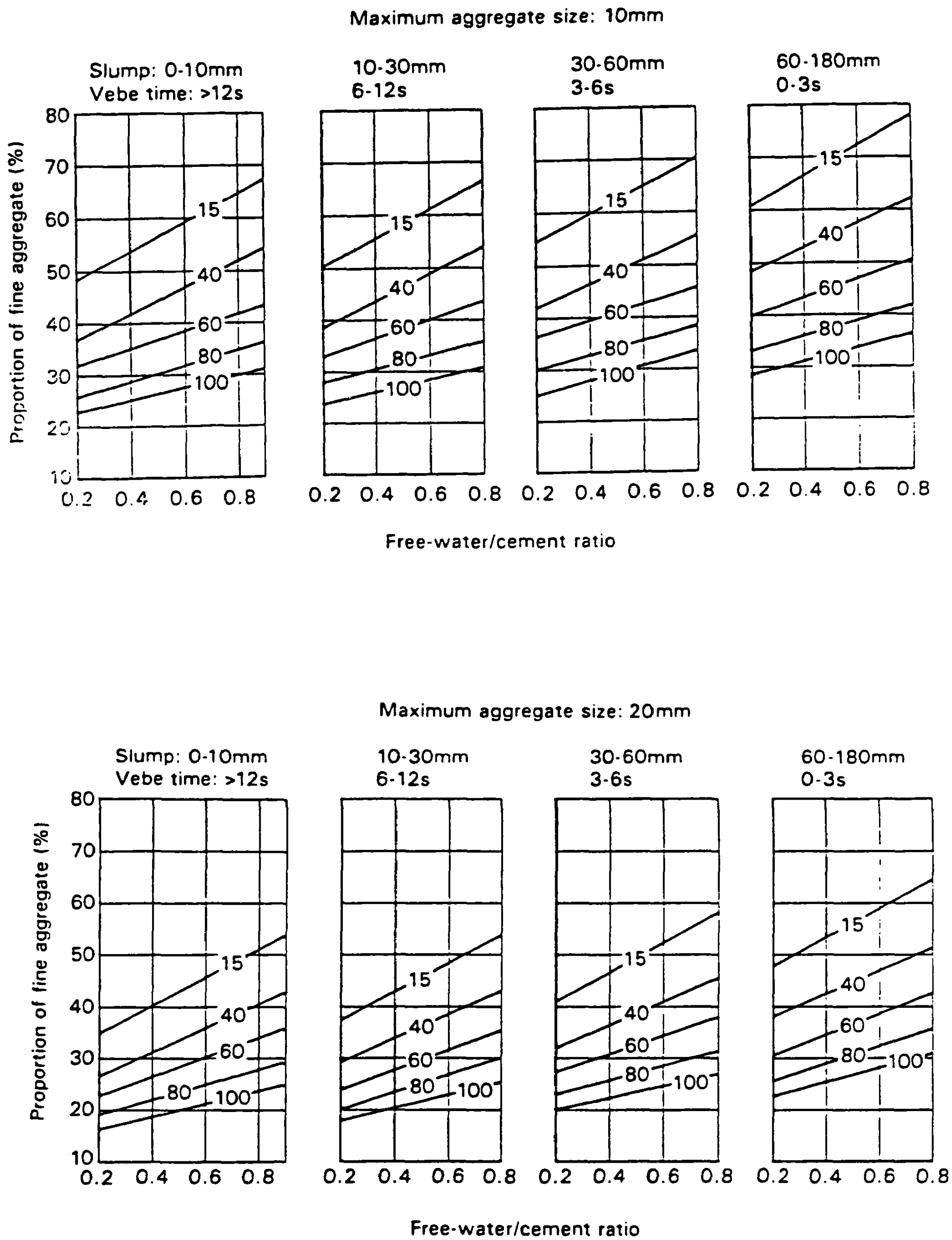


Figure 3.79: Recommended proportions of fine aggregate according to percentage passing a 600 μm sieve (Building Research Establishment's report)[47].

1988 edition[47] uses instead the percentage of fine aggregate passing the 600 um test sieve; the higher the percentage, the finer the fine aggregate. However, the fine aggregates should still comply with the coarse, medium, or fine grading requirements of BS 882: 1983[221]. The best proportion of fines to use in a given mix will depend on the shape of the particular aggregate, the actual grading of the fine aggregate and the use to which the concrete is to be put. However, adoption of a proportion obtained from Figure 3.79 will generally give a satisfactory concrete in the first trial mix which can then be adjusted as required for the exact conditions prevailing.

(iii) The ACI method[48]: Unlike the British methods that first determine the total aggregate content, the ACI method[48] first provides an estimate of the coarse aggregate content. According to ACI Committee 211, aggregates of essentially the same maximum size and grading will produce concrete of satisfactory workability when a given volume of coarse aggregate, on a dry-rodded basis, is used per unit volume of concrete. Appropriate values for this aggregate volume are given in Table 3.12. It can be seen that, for equal workability, the volume of coarse aggregate in a unit volume of concrete is dependent only on its maximum size and the fineness modulus of the fine aggregate. After having determined the coarse aggregate content, all the ingredients of the concrete have been estimated except the fine aggregate content, which can therefore be determined by difference.

The BRE report[47] recommends that where there is more appropriate information available related to local materials, this can be used instead of the typical data. A trial mix is then made, but because of the assumptions made at this stage in the design it is probable that this trial mix will not completely comply with the requirements. If necessary it is possible, from the trial mix results and information given in the current ACI and BRE procedures, to adjust the mix proportions and to use these for actual production or to prepare a revised trial mix.

According to Shacklock[222], the process of mix design can be broken down into two stages:

1. an estimation of the correct mix proportions based upon either published

data or past experience and upon a knowledge of the properties of the aggregate to be used, which may involve some preliminary testing;

2. small-scale trial mixes, usually in a laboratory, using the aggregate in a condition of known moisture content.

From this simple breakdown, which applied reasonably well in the 1950s, two contrary trends have emerged and are worth consideration. The large majority of structural concrete used in the United Kingdom nowadays is produced in either ready mixed concrete depots or precast concrete factories. In these instances a continuity of work is maintained and the experience gained while producing concrete for one contract automatically provides data for use with the next. Therefore, in (1) above, the emphasis is naturally placed upon past experience rather than published data, and (2) becomes largely unnecessary once a depot or factory has become established. The contrary trend is shown by the ever-increasing range of new materials developed; for example, lightweight aggregates and admixtures. In this instance the emphasis has often to be placed on published data since little past experience may exist, and small-scale trial mixes are probably essential. Further, in this latter trend, the data needed for mix design cannot be standardized because the new materials themselves often have widely differing properties. Both these trends do, however, add up in one respect: the "standardized" mix design methods, such as those published by BRE and ACI, are no longer as important as they used to be.

These mix design methods have been developed for proportioning normal strength concrete and do not apply for the design of high strength concrete mixes since:

1. the relationship between compressive strength and free water-cement ratio is defined for water-cement ratios of 0.30 to 0.90 and 0.40 to 0.82 in the Building Research Establishment's Report(47) and ACI standard(48) respectively.
2. An important stage of the mix designs is the determination of the free water content required for a certain workability.
3. The use of water-reducers or superplasticisers is not included in the BRE method(47).
4. Attempting to design a concrete mix using Table 3.14 (Table 3 from the Building Research Establishment's Report(47)) with a water-cement ratio of 0.30 and a slump

Slump (mm)		0-10	10-30	30-60	60-180
Vebe time(s)		> 12	6-12	3-6	0-3
Maximum size aggregate (mm)	Type of aggregate				
10	Uncrushed	150	180	205	225
	Crushed	180	205	230	250
20	Uncrushed	135	160	180	195
	Crushed	170	190	210	225
40	Uncrushed	115	140	160	175
	Crushed	155	175	190	205

Table 3.14: Approximate free-water contents (kg/m^3) required to give various levels of workability, (Building Research Establishment's report)[47]..

of 60-180 mm, using 10 mm crushed aggregate results in a cement content of 835 kg/m^3.

According to Mehta and Aitcin[46]:

> "The data from the ACI tables has, in some instances, been successfully extrapolated to determine the mix proportions for concretes with moderately high strength (40 to 60 N/mm^2, 0.4 to 0.3 water-cement ratio). However, the ACI tables are not applicable for very high strength (above 60 N/mm^2), high-slump (above 175 mm), superplasticised concrete mixtures with less than 0.3 water-cement ratio. Concretes of this type contain substantial amounts of unhydrated cement particles and a compact microstructure with strong transition zone. Their strength characteristics are highly sensitive not only to the aggregate type but also to very small changes in the water content of the concrete mixture. Also, there is no longer any direct proportionality between the water-cement or the water-binder ratio and strength since, at a given water-binder ratio, the strength can be affected significantly by the type and amount of mineral admixture used. In conclusion, since the aggregate type and the types and amounts of admixtures can have a great influence on the strength characteristics of high strength concrete mixtures - the factors which are not taken into consideration by the ACI 211 method - a different approach is needed for determining the mix proportions."

Much design work has been undertaken since the publication of the first edition of the Road Note No. 4 method[218]. Some of this work has, as with the previously described mix design methods, been based upon a consideration of aggregate grading and particle packing. Theories and equations, some very complex, have been derived in attempts to represent good concrete properties; none to date has become widely accepted, partly because complex theories require so many supporting data - for example, Method IV requires a knowledge of the specific surface characteristics of the aggregates - that they are not economically justified. Two of these mix design procedures, however, provide a more complete understanding of the basic factors affecting the properties of concrete and are therefore now described.

(VIII) Rational concrete mix design.

Between 1960 and 1968, Hughes published several papers[(223-227)] showing the existence of an optimum specific volume of coarse aggregate (a_0) which he defined as that quantity of coarse aggregate which gives maximum compactability of the concrete for given materials and given amounts of both water and cement. He determined this using first the compacting factor test and later showed that the same value was obtained using the Vebe test. The relations between compacting factor, or Vebe time, and the specific volume ratio of coarse aggregate, for constant specific volume ratios of cement and water are shown in Figure 3.80. If the specific volume ratio of coarse aggregate (a) exceeds the optimum (a_0), interference produced by contact between the particles, especially the coarse ones, becomes the dominant factor. If, however, the value of (a) is less than the optimum (a_0), changes in the total surface area of the aggregate have more effect on the workability than corresponding changes in particle interference. The optimum aggregate combination may be considered to occur when the interstices of the coarse aggregate are just sufficiently large to readily accept the finer particles, especially the coarser fraction of the fine aggregate. This implies that particle interference varies a good deal within a relatively small range of values of (a), hence any change in the total surface area of the aggregate is also small. The particle interference for given fine and coarse aggregates increases as some function of (a). The particle interference for given values of (a), but for different aggregates, depends upon their relative sizes. For this reason he derived two grading indices. They are the *grading modulus* and the *mean equivalent diameter.*

(i) The grading modulus.

The surface area is a very important characteristic of the aggregate grading. The simplest method of measuring the surface area of a single grain is to determine first the size, and secondly the shape. The former is given by the surface area of the "equivalent-size sphere", which is defined as that sphere which just passes the same

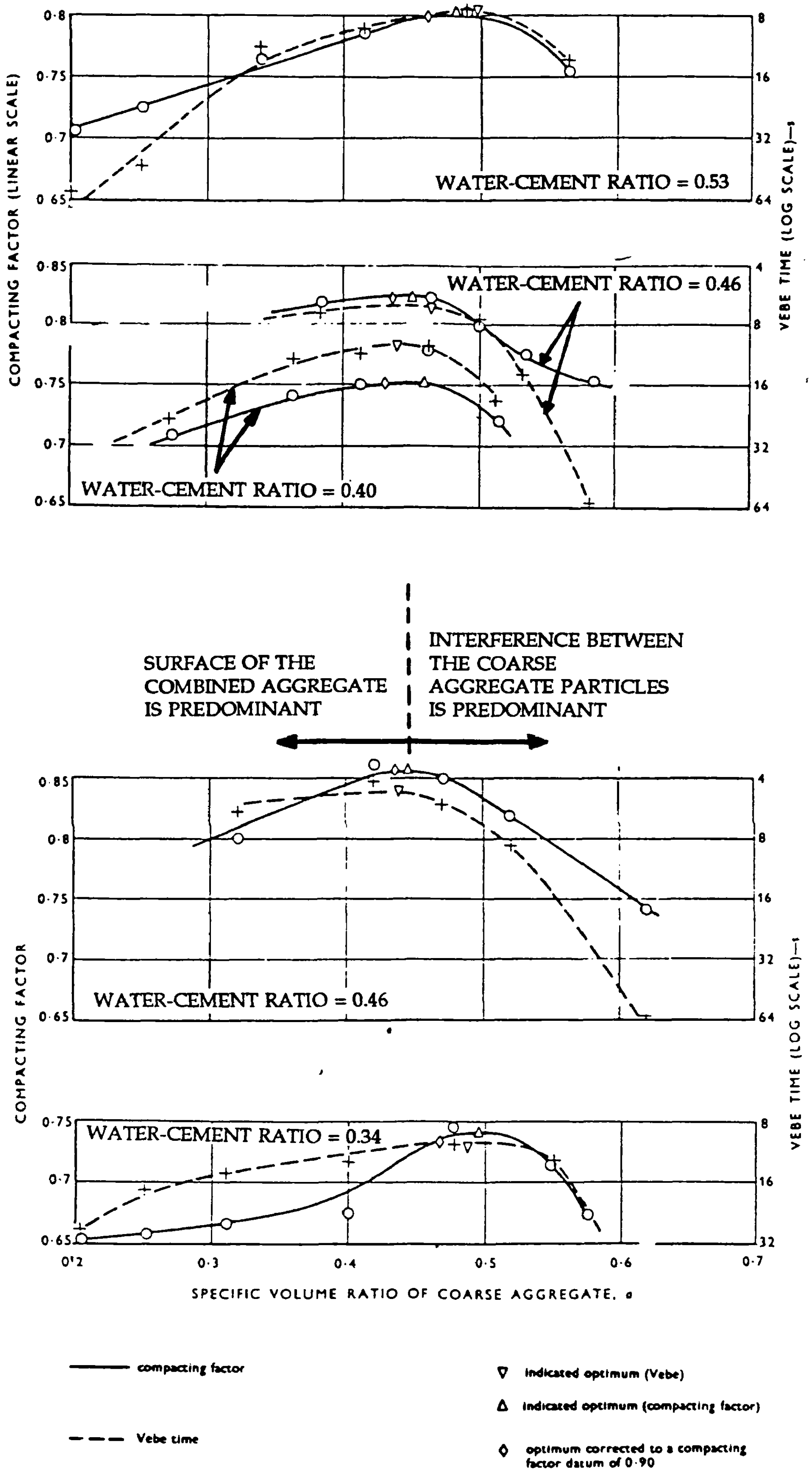


Figure 3.80: Relations between compacting factor, or Vebe time, and the specific volume ratio of coarse aggregate, for constant specific volume ratios of cement and water(224).

size of sieve as the actual grain. The latter can be indicated by an "angularity factor", which is given by the ratio of the surface area of the grain to the volume of an equivalent-size sphere. It is the surface area per unit apparent volume (G) of a collection of equivalent-size spheres which is of concern here. The value of (G) for spheres of constant diameter (D) is given by:

$$G = \frac{\pi \cdot D^2}{\pi \cdot \frac{D^3}{6}} \quad or \quad \frac{6}{D}$$

The diameter (D) varies for all natural aggregates, the actual variation being most conveniently estimated from the sieve analysis. It is assumed for convenience that the volumetric proportion of aggregate with particle diameter smaller than a given size varies as the logarithm of the size. Let D_1 and D_2 be the diameters of the smallest and the largest equivalent-size spheres of a particular group, i.e. the size group that just passes sieve size D_2 and is just retained on sieve size D_1, then:

$$G = \frac{\int_{D_1}^{D_2} \frac{6}{D} \cdot d(\ln D)}{\int_{D_1}^{D_2} d(\ln D)}$$

which simplifies to:

$$G = \frac{6\left(\frac{1}{D_1} - \frac{1}{D_2}\right)}{\ln\left(\frac{D_2}{D_1}\right)}$$

This value is referred to as the "grading modulus" of the aggregate. Table 3.15 gives the values of G for all the B.S. sieves from 1-1/2 in. to No. 200. The grading modulus of an aggregate is obtained by multiplying the percentage retained on each sieve by the appropriate value given in the table. The sum of all such products, divided by 100, is the grading modulus of the aggregate.

(ii) The equivalent mean diameter.

The mean particle size of the aggregate is a further important characteristic of the grading. He considered for convenience that the equivalent mean diameter for each size group retained on sieve D_1 to be the average of D_1 and D_2, where D_1 and D_2 are as previously defined. The equivalent mean diameters for the various size groups are included in Table 3.15. Using this table and the sieve analysis of the aggregate, its equivalent mean diameter can be obtained in the same way as for the grading modulus.

B.S. sieve	Aperture: inches	Grading modulus: sq. in./cu. in.			Equivalent mean diameter: inches		
1½"	1·5000	4·9	5·7		1·2500	1·1250	
1"	1·0000	6·9			0·8750		
¾"	0·7500	9·9	11·6		0·6250	0·5625	
½"	0·5000	14·0			0·4375		
⅜"	0·3750						
¼"	0·2500	19·7	23·0		0·3125	0·2813	
3/16"	0·1875	28·4			0·2388		
5	0·1320	37·8	46·0		0·1598	0·1412	
7	0·0949	52·4			0·1135		
8	0·0810	68	76		0·0880	0·0805	
10	0·0660	82		91	0·0735		0·0712
12	0·0553	99	108		0·0607	0·0567	
14	0·0474	117			0·0514		
16	0·0395	141	152		0·0435	0·0405	
18	0·0336	163		183	0·0366		0·0355
22	0·0275	197	214		0·0306	0·0286	
25	0·0236	235			0·0256		
30	0·0197	279	304		0·0217	0·0201	
36	0·0166	329		372	0·0182		0·0176
44	0·0139	396	439		0·0153	0·0141	
52	0·0116	475			0·0128		
60	0·0099	559	614		0·0108	0·0100	
72	0·0083	663		734	0·0091		0·0088
85	0·0070	786	856		0·0077	0·0072	
100	0·0060	924			0·0065		
120	0·0049	1,110	1,210		0·0055	0·0051	
150	0·0041	1,340		1,442	0·0045		0·0045
170	0·0035	1,585	1,710		0·0038	0·0036	
200	0·0030	1,845			0·0033		

Table 3.15: Grading moduli and equivalent mean diameters of the various aggregate size groups[223].

Using the above indices he assumed that the particle interference, for given values of (a), is related to a function of:

$$G_a D_b$$

where: G_a is the grading modulus of the coarse aggregate,

and, D_b is the equivalent mean diameter of the fine aggregate.

He used G_a to emphasize the finer particles of the coarse aggregate and D_b to emphasize the coarser fraction of the fine aggregate. He then plotted the optimum specific volume of coarse aggregate (a_0) against $G_a D_b$ for three different coarse aggregates, (Figure 3.81), and noted that, while there was appreciable scatter, all three curves possessed a similar shape. Further, if (a_0) was made equal to the "loose" specific bulk volume (a_B) of the respective coarse aggregates, i.e. the ratio of the gross apparent volume of the aggregate to the volume of the container, the curves gave a constant value of nearly 0.2 for $G_a D_b$.

The first mix design procedure, based on this optimum aggregate content, had to satisfy the following conditions:

1. The specific volumes of the constituents of a concrete mix, i.e. coarse aggregate (a), fine aggregate (b), cement (c), and the water (w), are the ratios of the gross apparent volume of each constituent to the total volume of the fully compacted concrete. Therefore:

$$a + b + c + w = 1. \qquad \text{Equation 1.}$$

2. The second condition relates strength to the cement-water ratio:

$$c/w = \text{constant, say } Q. \qquad \text{Equation 2.}$$

Hughes provided Figure 3.82 from which this can be determined.

3. The workability of concrete, for given materials, cement-water ratio, and aggregate grading, depends only upon the richness of the mix. It is therefore possible to select a given (a+b)/c ratio and then check the workability of the resulting concrete. Let the third condition, therefore, be given by the equation:

$$(a+b)/c = \text{constant, say } R. \qquad \text{Equation 3.}$$

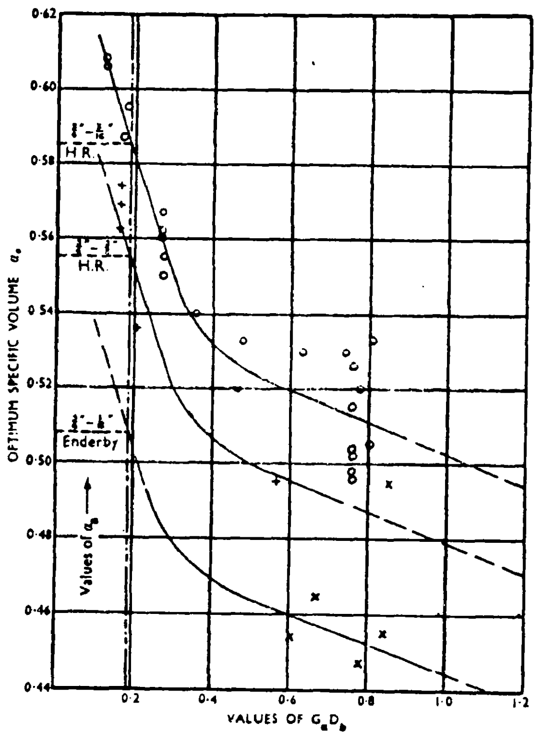


Figure 3.81: (a_0) plotted against G_aD_b for three different coarse aggregates(223). If (a_0) is made equal to the "loose" specific bulk volume (a_B) of the respective coarse aggregates, the curves give a constant value of nearly 0.2 for G_aD_b.

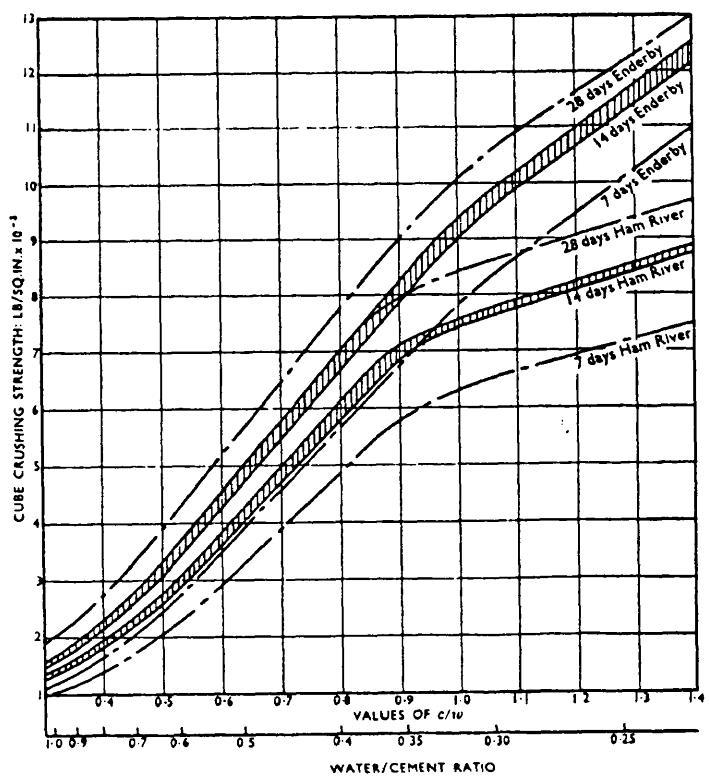


Figure 3.82: Compressive strength versus water-cement and cement-water ratio(223).

4. The optimum coarse aggregate proportion, as given by Figure 3.83, provides a condition which is easily applied as the following equation:

$$a = \text{constant, say } a_0. \qquad \text{Equation 4.}$$

The method involved the solving of equations 1 to 3 for c, i.e.

$$c = Q/(1+Q+Q.R) \qquad \text{Equation 5.}$$

Once the values of c/w and (a+b)/c are determined, the value of c can be obtained by direct substitution in Equation 5. All the specific volumes could then be readily calculated. Equation 3, i.e. the aggregate-cement ratio, is the only condition in the foregoing analysis which has not been accurately determined to satisfy its basic requirements, which is to provide a satisfactory workability for the designed mix. Later, he determined the relation between the Vebe time, the cement-water ratio and the cement content, (Figure 3.84), and even provided adjustments to the cement content for different cements and for the different grading indices of the aggregates.

This procedure has never attracted a lot of attention, maybe because at the time there already existed the Road Note No. 4 standard method for proportioning concrete mixtures[218], which was a much simpler procedure. However, similar ideas to those used by Hughes, i.e. the particle interference and the optimum aggregate combination being when the interstices of the coarse aggregate are just sufficiently large to readily accept the finer particles, have recently been used by Dewar in his approach to mix proportioning, which is therefore described next.

(IX) Dewar's theory of particulate mixtures.

Dewar[228,229] has, in the past 6 years, been developing mathematical models to assist in explaining particle interference and to enable computerization of void estimations and mix proportioning. This procedure is still under development and a monograph justifying the various formulae and providing detailed experimental evidence from tests of aggregates, mortars and concretes, has not yet been published. Therefore, only the main principles

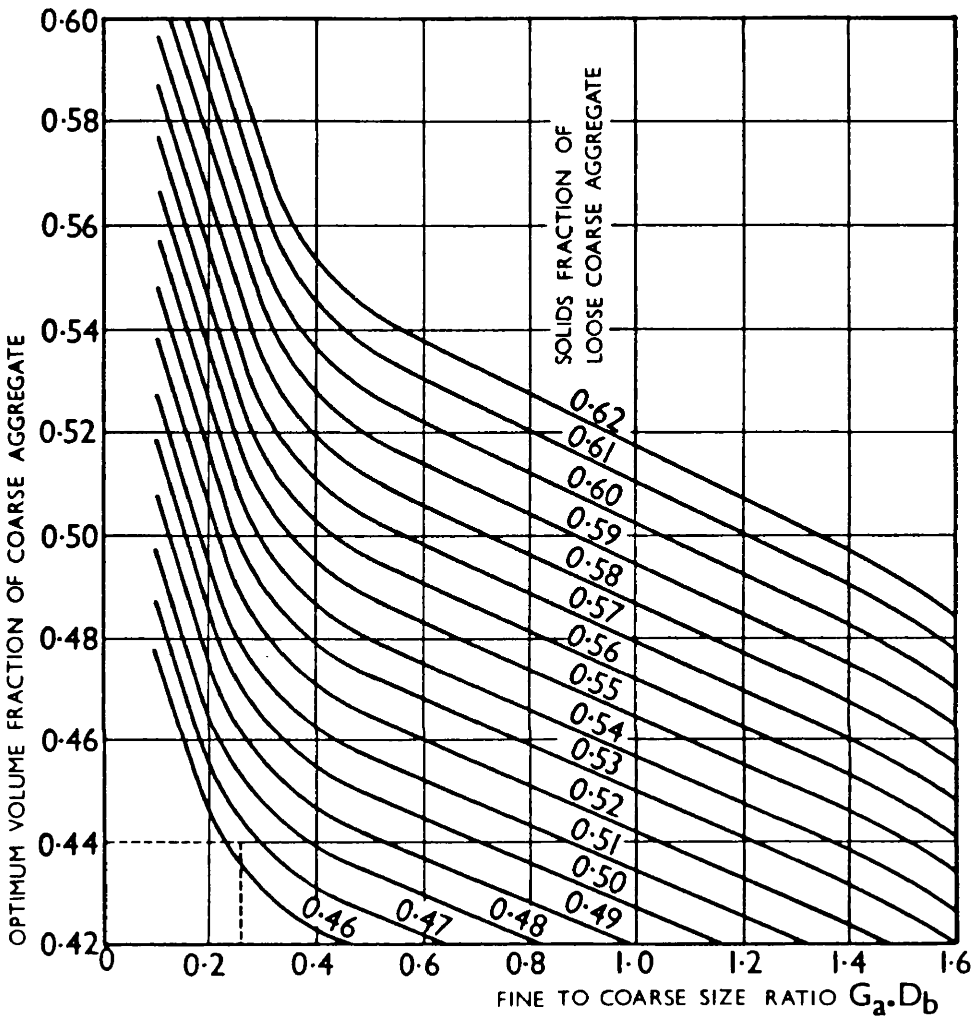


Figure 3.83: Chart for determination of optimum volume fraction of coarse aggregate (a_0). Data required for use of chart are grading moduli of both coarse aggregate and fine aggregate, and solids fraction of loose coarse aggregate[225].

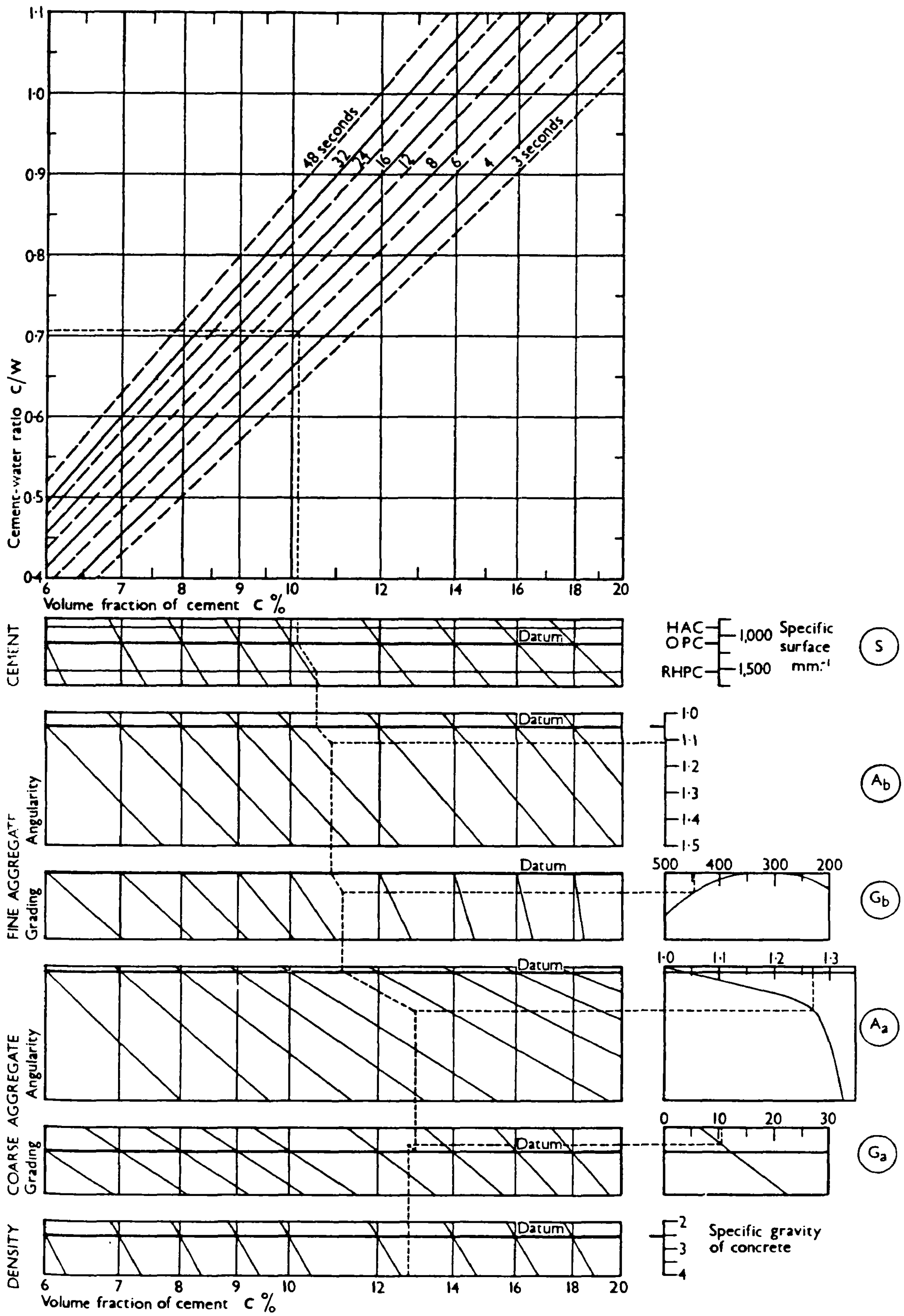


Figure 3.84: Relationships between the Vebe time, the cement-water ratio and the cement content[225].

used for modelling are described here.

Dewar considers that three parameters are sufficient to characterise any particulate component, in order to predict its effect on the structure of fresh concrete. These are:

1. *Relative density* which provides a relationship between mass and volume.
2. A grading index, *the mean particle size,* to enable size comparisons to be made. This is defined for single sized particles as the logarithmic mean of the boundary sieve sizes, A and B, between which the particles would be retained in a standard grading test.
3. *Aggregate voidage or voids ratio,* i.e. voids-solids ratio by volume, which accounts for: (a) distribution of sizes about the mean size, (b) particle shape, and, (c) texture. Aggregate voidage he determined in a loosely packed condition. Although the denser condition achieved by rodding the aggregate was considered to be inappropriate for the usual condition of the aggregate in concrete, it may, however, be relevant for rolled lean or very low workability vibrated concretes.

The visual general model for concrete is that shown in Figure 3.85, i.e. in concrete there is a coarse component in a fine component matrix, and there exists particle interference between them, which is explained as shown in Figure 3.86. The procedure followed in order to formulate the theory is in three steps, the voidage of the mixture in each step being inferred from a standard consistence test, i.e. the Vicat test described in BS 4550: Part 3[(230)], or a workability test, i.e. the slump test described in BS 1881: Part 102[(231)].

STEP 1: The voids ratio of the paste.

The voidage of the paste can not be assessed directly from tests of dry materials due to relatively large interparticle forces. Instead, the behaviour can be inferred from tests of pastes in which water and some residual air replaces air alone as the medium. The Vicat test of a cement paste is therefore used in place of bulk density test of a dry powder. Using values from this test, he defined the voids ratio (u) of powders as:

$$u = \frac{1}{100 - a}(RD_p \times SC + a)$$

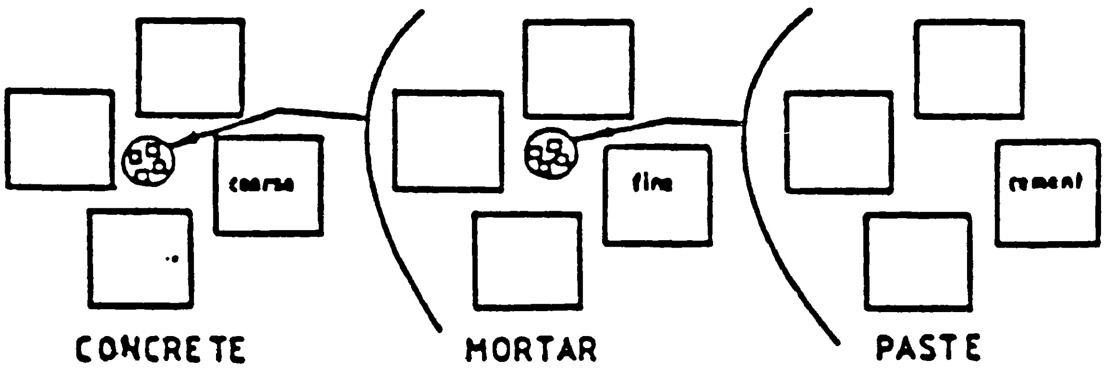
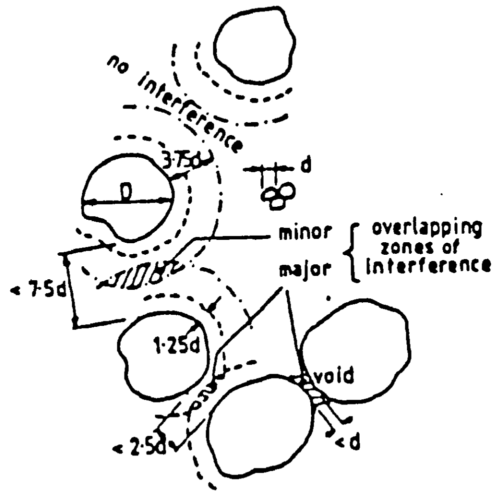
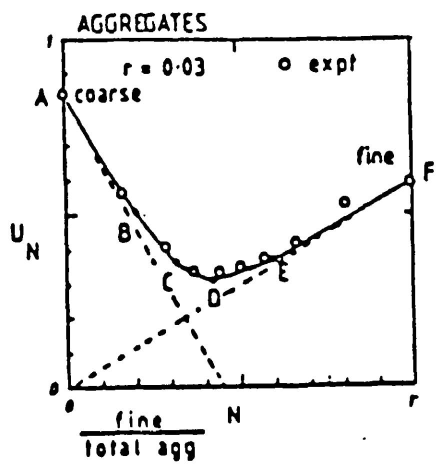


Figure 3.85: The visual general model for concrete, i.e. in concrete there is a coarse component in a fine component matrix(228).





Change Point	Condition at change point	Coarse particle separation (md)
A	Coarse particles alone, in contact	0
B	Available voids between coarse particles are just filled with fine particles.	0.25d
C	Separation of coarse particles equals mean size of fine particles.	d
D	Zones of major interference, surrounding coarse particles, are just in contact.	2.5d
E	Zones of minor interference, surrounding coarse particles, are just in contact.	7.5d
F	Fine particles alone	–

Note: d is mean size of fine component.

Condition in transition stage	Transition stage
Coarse particle structure dilated slightly by presence of fine particles; voids only partially filled by fine particles.	AB
High voidage between adjacent coarse particles; major interference with packing of coarse particles.	BC
Fine and coarse particles mutually interfering with packing; overlapping zones of major interference with packing of fine particles.	CD
Coarse particles interfering generally with packing of fine particles; overlapping zones of minor interference	DE
Coarse particles far enough apart for interference to be localised.	EF

Figure 3.86: Particle interference between coarse and fine aggregate(228).

where: RD_p = the relative density of the powder,

SC = the percentage water by mass of cement in the standard consistence test,

and, a = the percentage air, assumed to be 1.5%.

As mentioned above, no justification has yet been given for the use of this formula.

STEP 2: Application to mortars.

The water and air content of mortar of 50 mm slump can be considered identical to the voidage of the dry particle mixture. Again, no explanation has yet been given for chosing the slump value to be 50 mm. An example of a voids ratio diagram, i.e. how the voids ratio varies with increasing sand-cement ratio, is shown in Figure 3.87. Batch weights (kg/m^3) for cement, sand and water for points A to F are given in Table 3.16. Naturally, only a small part of this table is of practical interest.

STEP 3: The model for concrete.

Concrete is treated as a mixture of coarse aggregate with any one of the full range of mortars obtained in the previous section. Then the chosen mortar is combined with coarse aggregate, so that for each point A-F for mortar there are further points a-f in the overall voids ratio diagram for concrete, (Figure 3.88). Of these final points, d and e are the most important. Point d in most cases will be the lowest voids ratio, i.e. (water + air)/solids. It will also be the point to the left of which lower values of mortar content will lead to segregation and to the right of which, higher values of mortar content will progressively lead to over-cohesion and sometimes higher water contents, (Table 3.17).

As has been mentioned above, this procedure is still under development. A further development that it requires is the specification of the water-cement ratio required for attaining the specified compressive strength. At present it optimizes the proportions for a concrete of 50 mm slump so as to achieve minimum water content, but not necessarily minimum water-cement ratio.

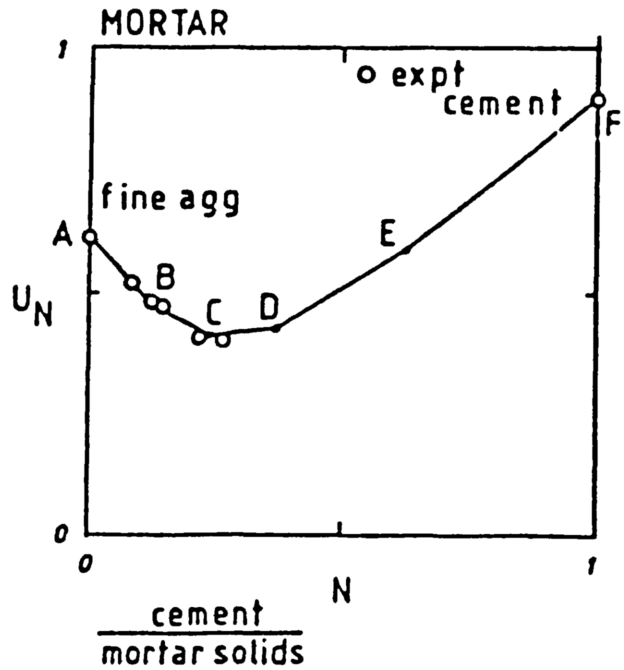


Figure 3.87: An example of a voids ratio diagram for mortar, i.e. how the voids ratio varies with increasing sand-cement ratio(228).

	Cement	Water	Sand
A	0	363	1605
B	264	302	1542
C	540	273	1387
D	829	281	1127
E	1226	351	613
F	1642	454	0

Note Assuming 2% air. Based on 50mm. slump.

Table 3.16: Batch weights (kg/m^3) for cement, sand and water for points A to F, shown in Figure 3.87(228).

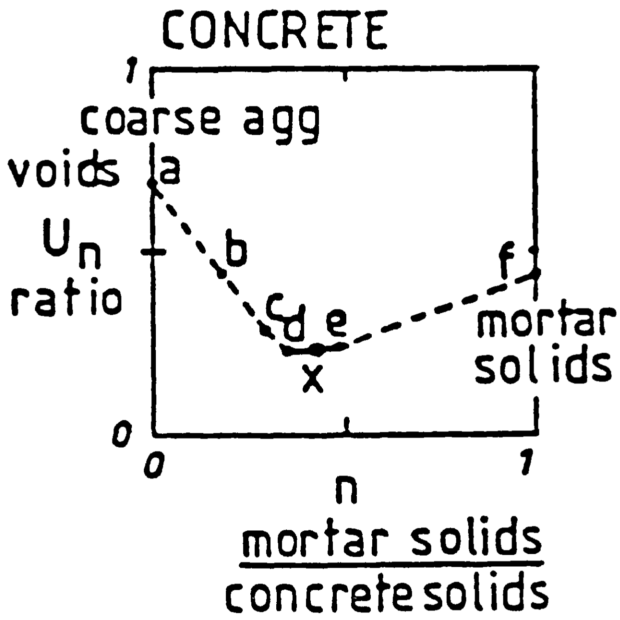


Figure 3.88: An example of a voids ratio diagram for concrete, i.e. how the voids ratio varies with increasing mortar-concrete ratio(228).

	cement	water	fine aggregate	coarse aggregate
Bx	152	180	788	1191
Cx	286	157	669	1258
Dx	397	164	524	1290
Ex	549	190	284	1331

Note

50mm slump.

Table 3.17: Batch weights (kg/m^3) for cement, fine and coarse aggregate, and water for points a to f, shown in Figure 3.88(228).

3.3.2 Mix proportioning of high strength concrete mixes.

Proper proportioning is more critical for high strength concrete since optimum performance is required from all materials used[102]. As we have seen earlier in the chapter, usually, specially selected pozzolanic and chemical admixtures are employed, and the attainment of a low water-binder ratio is considered essential. Quoting Albinger and Moreno[11]:

> "Selecting the proportions of a high strength concrete mixture is a combination of art and science. Because of the innumerable types of gradings of aggregates, chemistries of various cements, PFAs, and chemical admixtures, and the subsequent interaction of any combination of these materials, arriving at the optimum combination is often a matter of trial and error. Certainly basic concrete technology can be applied, but as in blending blue and yellow to make green, many combinations must be tried to attain the desired mix."

Since there are no generally accepted mix design procedures for high strength concrete, many trial batches have often been required to generate the data that enables the researcher to identify optimum mix proportions. However, two mix design procedures have been suggested, although they have not been widely accepted. These are first reviewed, and then some guidelines for the selection of mix proportions are discussed.

3.3.2.1 Mix design methods for high strength concrete.

Only four years after the publication of Road Note No. 4[218], Erntroy (one of the authors of Road Note No. 4) and Shacklock[232] published the first guidelines for the design of high strength concrete mixes. Their guidelines were, however, for only Portland cement mixes with no chemical or mineral admixtures. The next guidelines were published by Mehta and Aitcin[46,233] in 1990 and these consider the use of both chemical and mineral admixtures. These mix design procedures are described below:

(I) Erntroy and Shacklock, (1954).

Erntroy and Shacklock[232] realised that the properties of high strength concrete, (regarded by them as concrete with a 28-day compressive strength above 40 N/mm^2), depend on factors additional to those considered by Road Note No. 4[218]. They considered the main difference to lie in the fact that the workability of the mix and the type and maximum size of aggregate (assumed to have a sufficiently high ceiling of strength), as well as the strength requirement, influence the selection of the water-cement ratio. It follows that, in addition to the type of coarse aggregate, either the aggregate-cement ratio or the workability has to be known in order to choose the water-cement ratio for a required strength. They therefore prepared empirical graphs relating compressive strength to an arbitrary "reference number" for concretes made with irregular gravel and crushed granite coarse aggregates. These graphs are reproduced in Figures 3.89 and 3.90 for mixes with ordinary Portland cement, and in Figures 3.91 and 3.92 for mixes with rapid hardening Portland cement.

Having obtained the reference number for the desired strength, the water-cement ratio to give the required workability is found with the aid of Figures 3.93 and 3.94 for aggregates with a maximum size of 3/4 in. (19.1 mm) and 3/8 in. (9.5 mm) respectively.

The aggregate-cement ratio can now be found from Tables 3.18 and 3.19. The values given in these tables were obtained with aggregates containing 30 per cent of material passing the 3/16 in. (4.8 mm) sieve, and therefore suitable adjustments must be made for other gradings.

It must be noted that their investigation included only two types of cement (an ordinary and a rapid hardening Portland cement) and two types of coarse aggregate of two maximum sizes, 3/4 in. (19.1 mm) and 3/8 in. (9.5 mm), in combination with only one type of natural sand. As with other methods, quoting the authors:

> "Owing to the appreciable effects of the quality of the constituent materials, the design is not likely to be so precise and every effort should

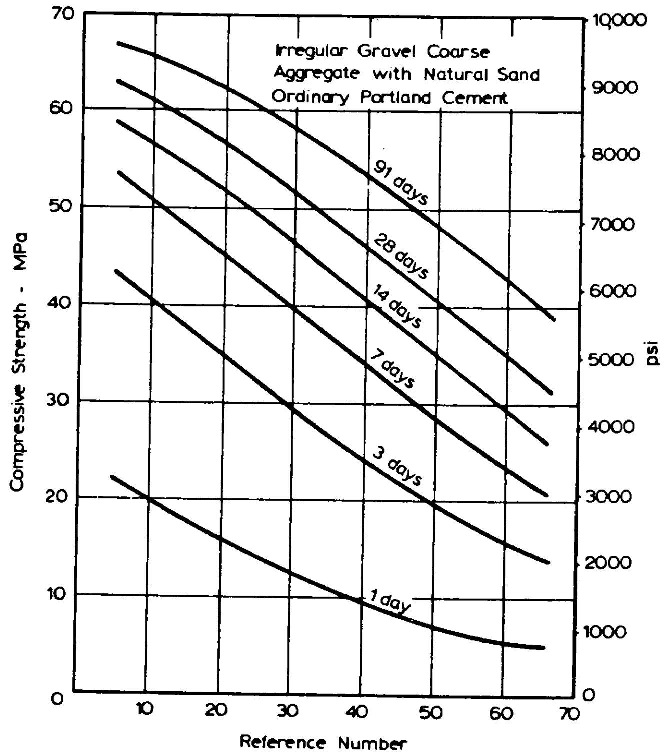


Figure 3.89: Relation between compressive strength of 4 in. (102 mm) cubes and "reference number" for mixes containing irregular gravel, natural sand, and ordinary Portland cement[232].

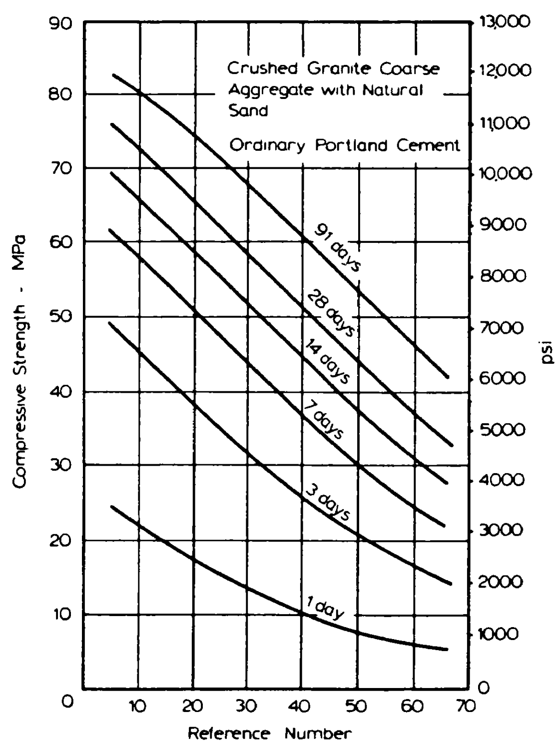


Figure 3.90: Relation between compressive strength of 4 in. (102 mm) cubes and "reference number" for mixes containing crushed granite, natural sand, and ordinary Portland cement[232].

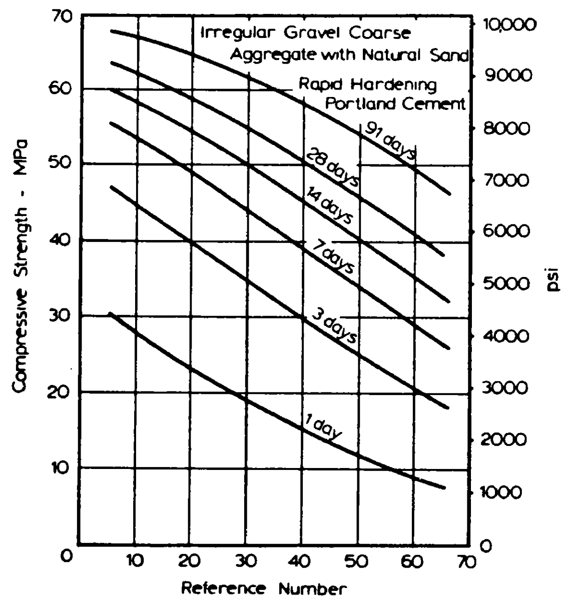


Figure 3.91: Relation between compressive strength of 4 in. (102 mm) cubes and "reference number" for mixes containing irregular gravel, natural sand, and rapid hardening Portland cement(232).

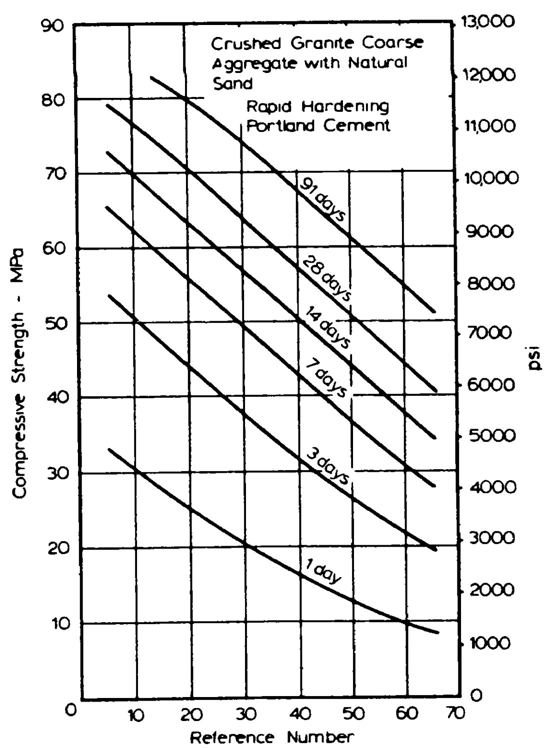


Figure 3.92: Relation between compressive strength of 4 in. (102 mm) cubes and "reference number" for mixes containing crushed granite, natural sand, and rapid hardening Portland cement(232).

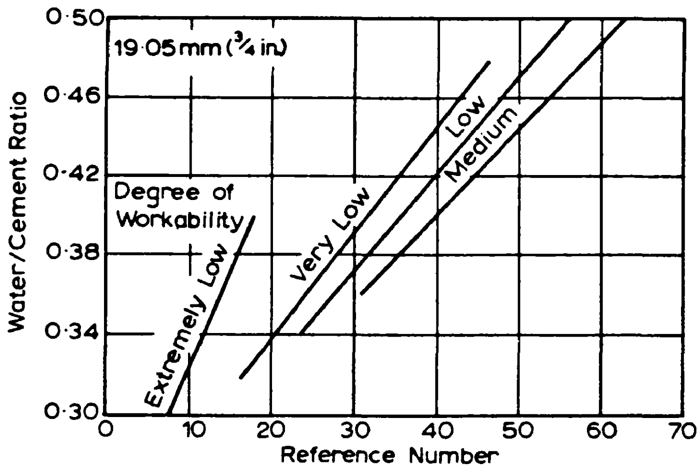


Figure 3.93: Relation between water-cement ratio and "reference number" for 3/4 in. (19.1 mm) maximum aggregate size(232).

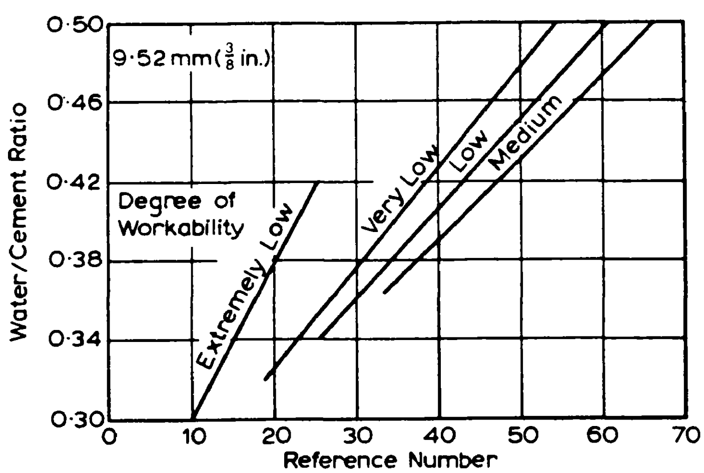


Figure 3.94: Relation between water-cement ratio and "reference number" for 3/8 in. (9.5 mm) maximum aggregate size(232).

EL = "extremely low" VL = "very low", 0 - 25 mm slump
L = "low", 25 - 50 mm M = "medium", 50 - 100 mm

Type of coarse aggregate*		Irregular gravel								Crushed granite							
Maximum size of aggregate		19·05 mm (¾ in.)				9·52 mm (⅜ in.)				19·05 mm (¾ in.)				9·52 mm (⅜ in.)			
Degree of workability†		EL	VL	L	M	EL	VL	L	M	EL	VL	L	M	EL	VL	L	M
Water/cement ratio by weight	0·30	3.0	—	—	—	2·4	—	—	—	3·3	—	—	—	2·9	—	—	—
	0·32	3·8	2·5	—	—	3·2	—	—	—	4·0	2·6	—	—	3·6	2·3	—	—
	0·34	4·5	3·0	2·5	—	3·9	2·6	—	—	4·6	3·2	2·6	—	4·2	2·8	2·3	—
	0·36	5·2	3·5	3·0	2·5	4·6	3·1	2·6	—	5·2	3·6	3·1	2·6	4·7	3·2	2·7	2·3
	0·38	—	4·0	3·4	2·9	5·2	3·5	3·0	2·5	—	4·1	3·5	2·9	5·2	3·6	3·0	2·6
	0·40	—	4·4	3·8	3·2	—	3·9	3·3	2·7	—	4·5	3·8	3·2	—	4·0	3·3	2·9
	0·42	—	4·9	4·1	3·5	—	4·3	3·6	3·0	—	4·9	4·2	3·5	—	4·4	3·6	3·1
	0·44	—	5·3	4·5	3·8	—	4·7	3·9	3·3	—	5·3	4·5	3·7	—	4·8	3·9	3·3
	0·46	—	—	4·8	4·1	—	5·1	4·2	3·6	—	—	4·8	4·0	—	5·1	4·2	3·6
	0·48	—	—	5·2	4·4	—	5·4	4·5	3·8	—	—	5·1	4·2	—	5·5	4·5	3·8
	0·50	—	—	5·5	4·7	—	—	4·8	4·1	—	—	5·4	4·5	—	—	4·7	4·0

* Natural sand used in combination with both types of coarse aggregate.

Table 3.18: Aggregate-cement ratio (by weight) required to give four degrees of workability with different water-cement ratios using ordinary Portland cement[232].

Type of coarse aggregate*		Irregular gravel								Crushed granite							
Maximum size of aggregate		19·05 mm (¾ in.)				9·52 mm (⅜ in.)				19·05 mm (¾ in.)				9·52 mm (⅜ in.)			
Degree of workability†		EL	VL	L	M	EL	VL	L	M	EL	VL	L	M	EL	VL	L	M
Water/cement ratio by weight	0·32	2·6	—	—	—	—	—	—	—	2·9	—	—	—	2·5	—	—	—
	0.34	3·4	2·2	—	—	2·8	—	—	—	3·6	2·4	—	—	3·2	—	—	—
	0·36	4·1	2·7	2·3	—	3·5	2·4	—	—	4·3	2·9	2·4	—	3·9	2·5	—	—
	0·38	4·8	3·2	2·8	2·3	4·2	2·9	2·4	—	4·9	3·4	2·9	2·4	4·5	3·0	2·5	—
	0·40	5·5	3·7	3·2	2·7	4·9	3·3	2·8	2·3	5·5	3·9	3·3	2·7	5·0	3·4	2·9	2·4
	0·42	—	4·2	3·6	3·0	—	3·7	3·1	2·6	—	4·2	3·6	3·0	5·5	3·8	3·2	2·7
	0·44	—	4·6	4·0	3·4	—	4·1	3·5	2·9	—	4·7	4·0	3·3	—	4·2	3·5	3·0
	0·46	—	5·0	4·3	3·7	—	4·5	3·8	3·2	—	5·1	4·3	3·6	—	4·6	3·8	3·2
	0·48	—	5·5	4·7	4·0	—	4·9	4·1	3·5	—	5·5	4·6	3·9	—	5·0	4·1	3·4
	0·50	—	—	5·0	4·3	—	5·2	4·4	3·7	—	—	4·9	4·1	—	5·3	4·4	3·7

* Natural sand used in combination with both types of coarse aggregate.

Table 3.19: Aggregate-cement ratio (by weight) required to give four degrees of workability with different water-cement ratios using rapid hardening Portland cement[232].

be made to determine the suitability of the proposed mix proportions by means of trial mixes."

Table 3.20 compares the water-cement ratios and maximum 28-day compressive strengths considered by this method to those of ACI and BRE. Even with this method, that is specifically for high strength concrete mixes, the maximum 28-day strength is not more than 75 N/mm^2, and this can only be achieved at an extremely low workability, i.e. zero slump. Mixes with approximately 100 mm slump and 28-day compressive strength of 60 N/mm^2 require cement contents as high as 600 kg/m^3, (Table 3.21).

This empirical extension of the existing normal strength concrete mix designs, that are themselves based on compilations of a large number of experimental data, becomes complicated as the production of high strength concrete nowadays usually employs pozzolanas and water-reducers or superplasticisers. These have not been covered by this mix design procedure as at the time pozzolanas were not widely used, and superplasticisers had not yet been developed. Also, the data for the mix design of high strength concrete cannot be standardised because of:

(i) the widely differing properties of these new materials, as already mentioned earlier for normal strength concrete, and,

(ii) the availability of different concrete making materials in different regions.

Because of these, the empirical extension of the mix design procedure for normal strength concrete mixes becomes even less precise for high strength concrete mixes.

(II) Mehta and Aitcin, (1990).

Mehta and Aitcin[46,233] have also developed a step by step procedure, which is essentially similar to the ACI procedure for proportioning normal concrete mixtures. Briefly, concrete mixtures are classified into five strength grades, with 28-day average compressive strengths of 65, 75, 90, 105, and 120 N/mm^2. To insure low shrinkage and creep, it is assumed that the total cement paste content in the 65-120 N/mm^2 strength range shall remain fixed at 35 per cent by volume, and strength shall be controlled by controlling the

MIX DESIGN METHOD	Water-binder ratio.	Mineral admixture			Chemical admixtures	Maximum 28-day compressive strength
		PFA	GGBFS	CSF		
ACI	0.41 - 0.82	YES	YES	NO[1]	YES	6,000 psi (41.4 N/mm^2)
BRE	0.30 - 0.90	YES	YES	NO	NO	80 N/mm^2
ERNTROY & SHACKLOCK	0.30 - 0.50	NO	NO	NO	NO	75 N/mm^2

(1) Recently, in 1991, ACI Committee 211 proposed to include proportioning with CSF.

Table 3.20: Water-binder ratios and maximum 28-day compressive strengths considered by various mix design methods.

MIX DESIGN METHOD	Water-binder ratio	Slump	28-day compressive strength	Aggregate/cement ratio	Cement
ACI	0.41	3 to 4 in. (75 - 100 mm)	6,000 psi (41.4 N/mm^2)	2.6	878 lb/yd^3 (521 kg/m^3)
BRE	0.30	60 - 180 mm	80 N/mm^2	1.9	750 kg/m^3
ERNTROY & SHACKLOCK	0.36	50 - 100 mm	58 N/mm^2	3.1	596 kg/m^3

Table 3.21: Mixes designed with various mix design methods.

quality of the cement paste and not by increasing the cement paste-aggregate ratio. As a first approximation, the fine-coarse aggregate ratio is assumed to be 2:3 by volume. Since the consistency of superplasticised concrete mixture need not be governed by the water content (at a given water content the consistency can be controlled by changing the superplasticizer dosage), the strength can be related to the water content and the content of portland cement and mineral admixtures. Depending on the desired rate of strength development and the local availability of high quality PFA, GGBFS, and CSF, the procedure offers the concrete manufacturer the options of using either: (i) Portland cement alone, (ii) Portland cement in combination with PFA or GGBFS, (iii) Portland cement in combination with a mixture of CSF and PFA or GGBFS. Using the preceding assumptions, the authors have been able to calculate the mix proportions shown in Table 3.22, that they recommend for use as a first trial batch. No guidelines have, however, been given for adjusting these proportions to account for the variability in physical properties of concrete making materials available in different regions.

3.3.2.2 Guidelines for the selection of mix proportions for high strength concrete mixes.

Since the two mix design methods discussed above do not account for the variability in physical properties of concrete making materials, many trial batches have often been required to generate the data that enables the researcher to identify optimum mix proportions. These have varied widely not only because of the different material characteristics, but also depending upon such factors as the strength level required, test age and type of application. In addition, economics, structural requirements, manufacturing practicality, anticipated curing environment, and even the time of year have affected the selection of mix proportions[1].

(I) Water-cement and water-binder ratio.

The relationship between water-cement ratio and compressive strength, has been found to

Strength Grade	Avg. Strength MPa	Option	Cementitious Materials PC	FA or BFS	CSF	Total* Water	Coarse Agg.	Fine Agg.	Total Batch	W/C
A	65	1	534	--	--	160	1050	690	2434	0.30
		2	400	106	--	160	1050	690	2406	0.32
		3	400	64	36	160	1050	690	2400	0.32
B	75	1	565	--	--	150	1070	670	2455	0.27
		2	423	113	--	150	1070	670	2426	0.28
		3	423	68	38	150	1070	670	2419	0.28
C	90	1	597	--	--	140	1090	650	2477	0.23
		2	447	119	--	140	1090	650	2446	0.25
		3	447	71	40	140	1090	650	2438	0.25
D	105	--	--	--	--	--	--	--	--	--
		2	471	125	--	130	1110	630	2466	0.22
		3	471	75	42	130	1110	630	2458	0.22
E	120	--	--	--	--	--	--	--	--	--
		2	495	131	--	120	1120	620	2486	0.19
		3	495	79	44	120	1120	620	2478	0.19

+The mix proportions are for non-air-entrained concrete, although 2% entrapped air is assumed.
*Total water includes the water in the superplasticizing admixture, the dosage of which may range from 10 to 20L/m^3, depending on consistency and strength requirements.

Table 3.22: Calculated mix proportions for the first trial batch, (kg/m^3)[46].

extend to higher-strength concretes[32], (Figure 3.95). A U.S. Army investigation[149] concluded that the single most important variable in achieving high strength concrete is the water-cement ratio. When pozzolanic materials are used in concrete, a water-(cement plus pozzolan) ratio, or water-binder ratio, by weight has been considered in place of the traditional water-cement ratio by weight. The relationship between strength and water-binder ratio can be influenced by the cement, mineral admixture, (Figure 3.96)[45], and aggregate, but it is not greatly affected by water-reducers and superplasticisers[32]. However, the use of superplasticisers greatly aids the production of workable concrete with a low water-binder ratio.

(II) Cement content.

The cement content of a high strength mixture has best been determined, as with selecting the type of cement, (see page 66), by the fabrication of trial batches[1]. Cement contents in high strength concrete test programmes have ranged from 390 to 560 kg/m^3.

ACI Committee 363[1] recommended that, in evaluating optimum cement contents, trial mixes should be proportioned to equal consistencies, allowing the water content to vary according to the water demand of the mixture. Albinger and Moreno[11] stated that, for any particular combination of materials, an optimum cement content exists beyond which the strength does not continue to increase and the mix becomes too "sticky" to handle. Freedman[44] found that the 28-day compressive strength did not increase for cement contents above 8.5 to 10 sacks/yd^3 (474 to 557 kg/m^3), (Figure 3.97). Day[143] reported that the mix used for the Collins Place was a "maximum strength" design in that cement contents higher than the 520 kg/m^3 used did not give a higher strength. He stated, however, that if different admixtures, e.g., PFA, were used they might give higher strengths with higher cementitious contents.

The Chicago Committee on High-Rise Buildings[8] suggested trial batches using cement contents of 7.0 to 10.0 sacks/yd^3 (390 to 558 kg/m^3), comparing strengths on the basis of constant slump. Similarly, Freedman[44] concluded that the cement content must be at least

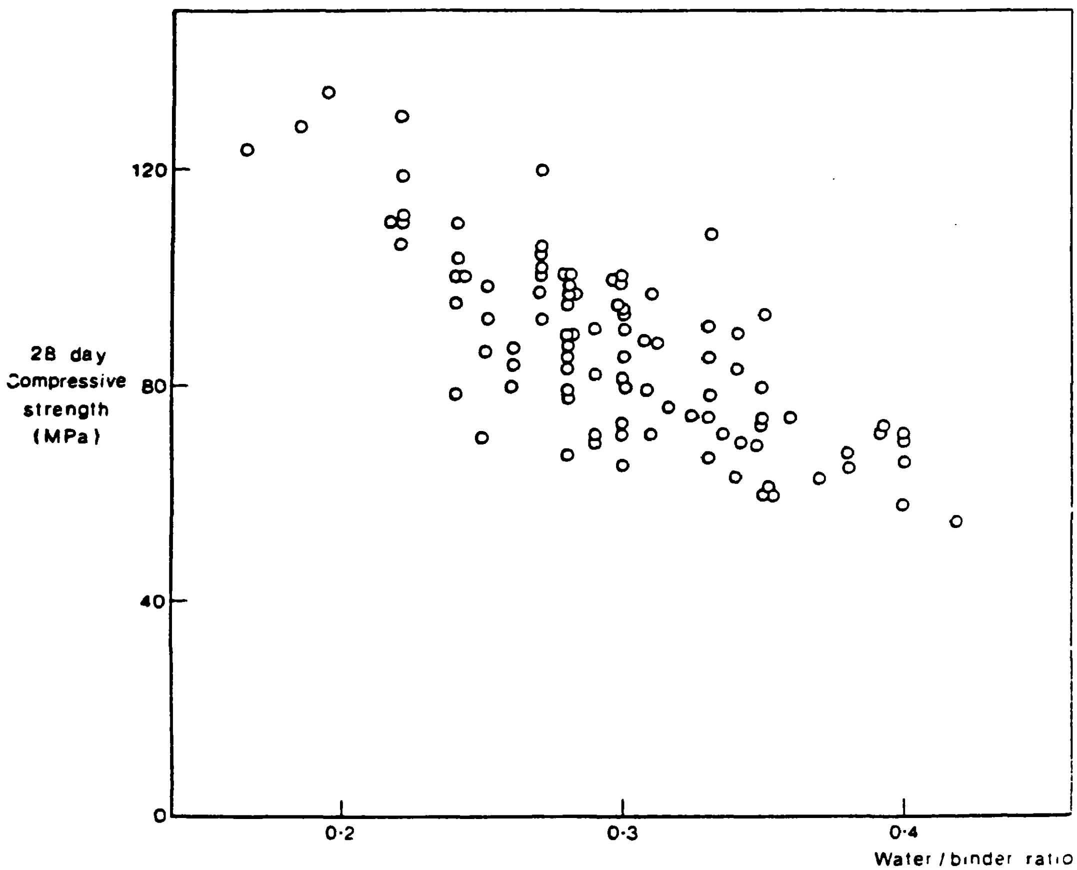


Figure 3.95: Effect of water-binder ratio on the 28-day compressive strength(32). Data include a wide variety of binders, chemical admixtures, aggregates, etc..

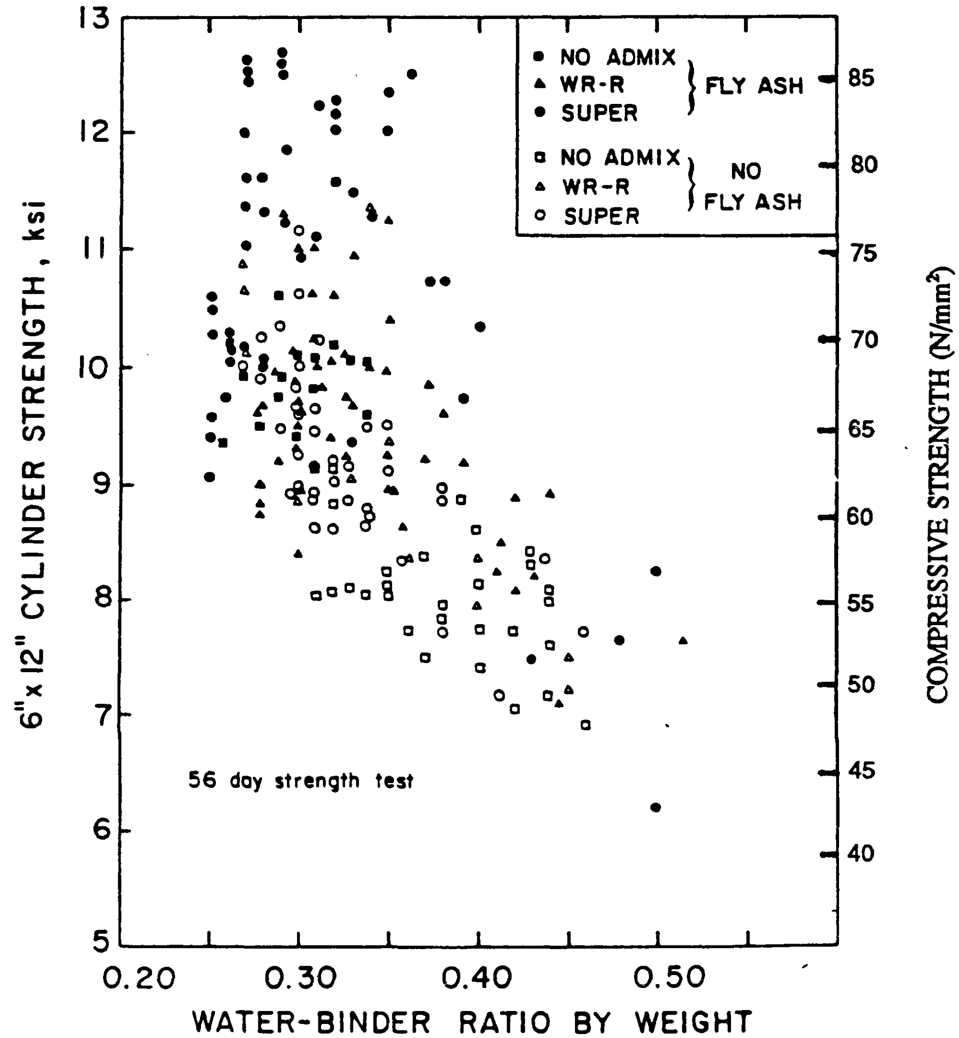
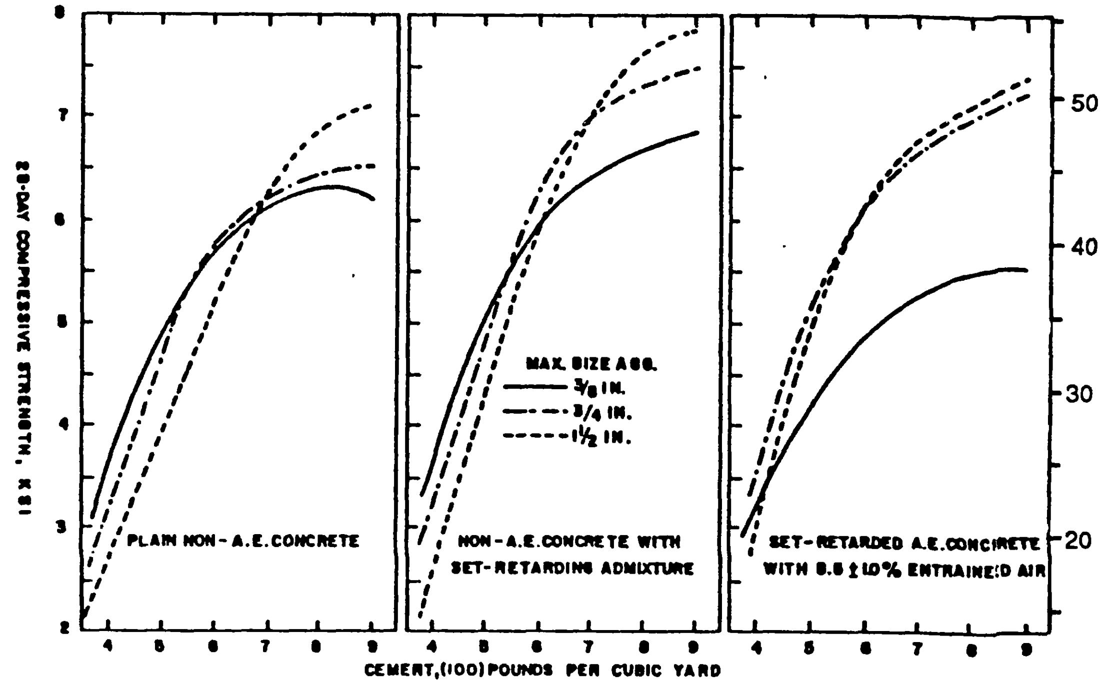


Figure 3.96: Water-binder ratio versus concrete compressive strength[45].



lb/yd³	kg/m³
400	237.2
500	296.5
600	355.8
700	415.1
800	474.4
900	533.7

Figure 3.97: Effect of cement content on the compressive strength of concrete(44).

6.5 sacks/yd^3 (363 kg/m^3) for producing high strength concrete having a 100 mm slump, but that in order to achieve 10,000 psi (69.0 N/mm^2) concrete strengths at 90-days a cement content of 10 sacks/yd^3 (557 kg/m^3) is needed.

Peterman and Carrasquillo[27] stated that cement contents in excess of 8.5 sacks/yd^3 (474 kg/m^3) and as high as 11.0 or 12.0 sacks/yd^3 (613 or 669 kg/m^3) should be used for trial concrete batches containing no PFA and no chemical admixtures. The low mixing water requirement associated with high cement factors was greatly responsible for achieving high strength in mixes containing no chemical or mineral admixtures, (Figure 3.98). However, when evaluating the effects of cement content and superplasticiser dosage on concrete strengths, cement contents in the range from 6.0 to 10.0 sacks/yd^3 (390 to 559 kg/m^3) should be considered. The optimum cement content for their superplasticised concrete mixes was 8.5 sacks/yd^3 (475 kg/m^3), (Figure 3.99). They also noted that compaction of specimens was most effective in the 8.5 sacks/yd^3 (475 kg/m^3) mixes. Fresh concrete mixes containing 7.0 sacks/yd^3 (390 kg/m^3) tended to be harsh while the 10-sack mixes (559 kg/m^3) were generally "sticky".

Yamamoto and Kobayashi[234] reported that 9.0 sacks/yd^3 (502 kg/m^3) was the most economical cement content and the minimum for producing high strength concrete without segregation. Another report[149] concluded that the optimum cement content depends on cement type: 10 sacks/yd^3 (558 kg/m^3) for Type I (ordinary Portland) cement and 9.25 sacks/yd^3 (516 kg/m^3) for Type II (moderate heat Portland) cement.

A principle consideration in establishing the desired cement content will be the identification of combinations of materials which will produce maximum strengths. Ideally, evaluations of each potential source of cement, pozzolana, liquid admixture and aggregate in varying concentrations would indicate the optimum combination of materials. Testing costs and time requirements usually have limited the completeness of the testing programmes[1].

There are also factors other than strength, as has been mentioned on page 159, which may limit the maximum quantity of cement which may be desirable in a high strength concrete

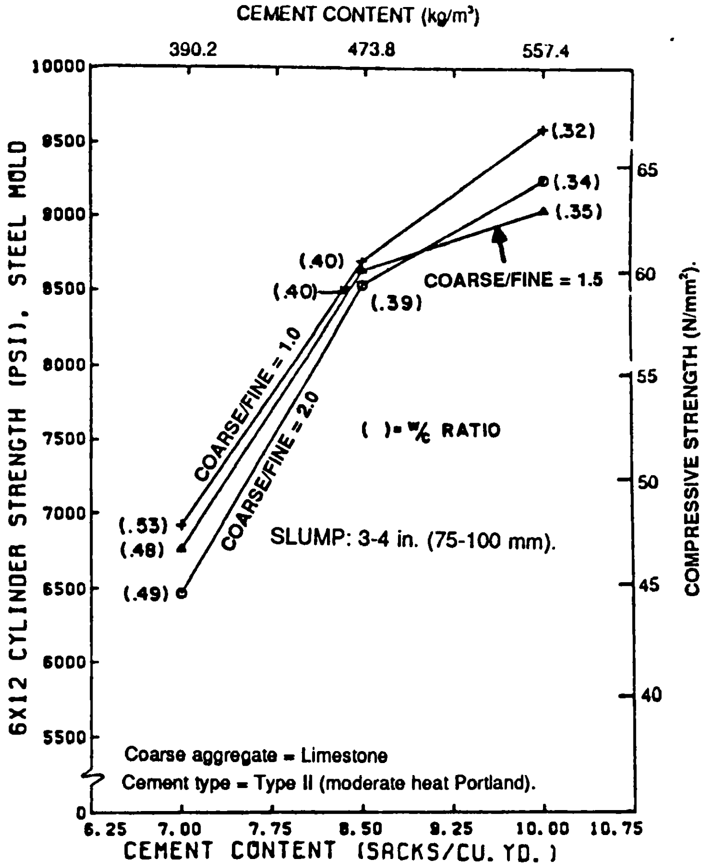


Figure 3.98: Effect of cement content on the 56-day compressive strength of non-superplasticised concrete(27).

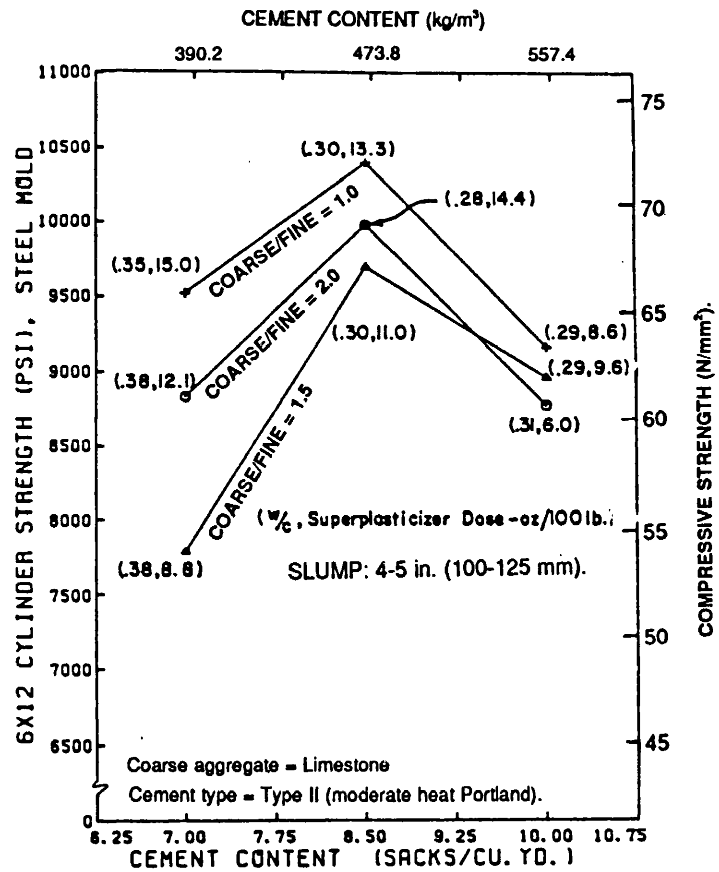


Figure 3.99: Effect of cement content on the 56-day compressive strength of superplasticised concrete(27).

mixture.

(III) Aggregate proportions.

In the proportioning of concrete of all strength levels, the aggregates are a very important consideration since they occupy the largest volume of any of the ingredients in the concrete. It is generally agreed that the fine aggregates or sand have considerable impact on the properties of fresh concrete since they have a much higher surface area than the coarse aggregates. Since the surface of all the aggregate particles must be coated with a cementitious paste, not only the fineness modulus of the sand, (see page 108), but also the proportion of fine to coarse can have a direct quantitative effect on paste requirements. According to Albinger and Moreno[11]:

> "Optimum strength and workability are attained with a ratio of coarse to fine aggregate above that usually recommended for normal strength concretes."

ACI Committee 363[1] also advocated that:

> "Low fine aggregate contents with high coarse aggregate contents have resulted in a reduction in paste requirements and normally have been more economical. Such proportions have also made it possible to produce higher strengths for a given amount of cementitious materials. However, if the proportion of sand is too low, serious problems in workability become apparent."

Peterman and Carrasquillo[27] did not find any clear trend between compressive strength of concrete as a function of the coarse-fine aggregate ratio in mixes containing no admixtures, (Figure 3.100). For mixes containing superplasticisers, a coarse-fine aggregate ratio of 2.0 produced only slightly higher compressive strengths, as shown in Figure 3.101. The effect of the coarse-fine aggregate ratio on the workability of the superplasticised mixes with various cement contents has been summarised as in Table 3.23.

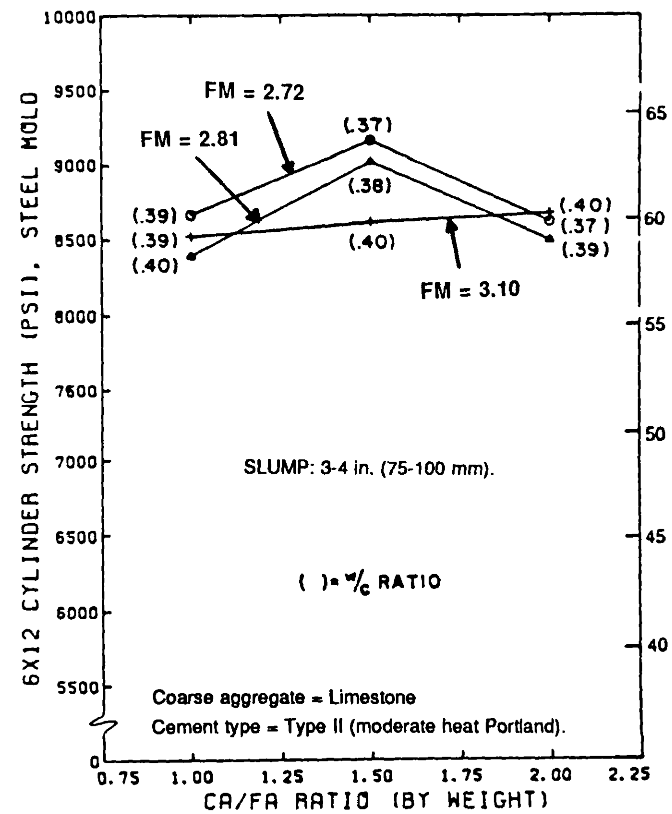


Figure 3.100: Effect of coarse-fine aggregate ratio on the 56-day compressive strength of non-superplasticised concrete[27].

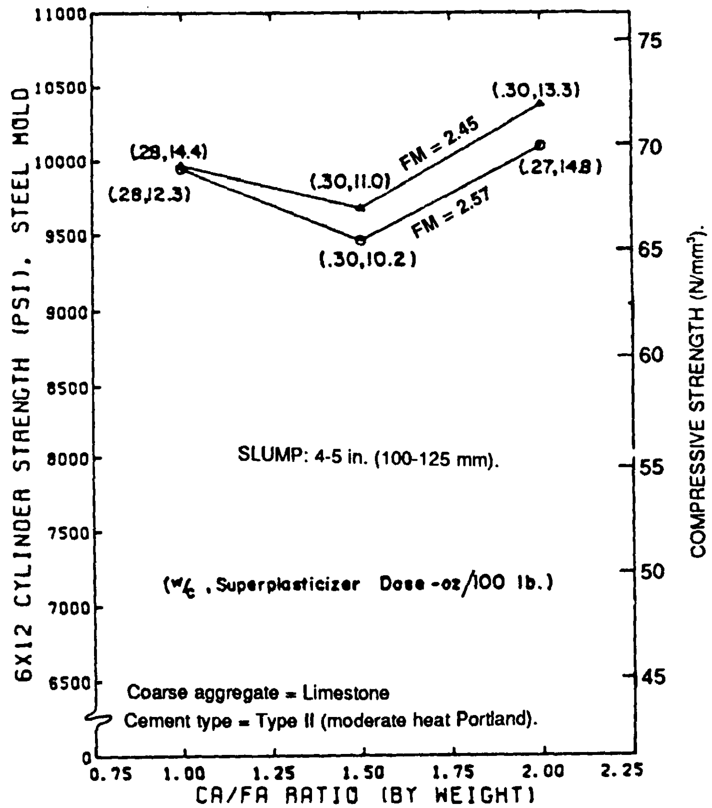


Figure 3.101: Effect of coarse-fine aggregate ratio on the 56-day compressive strength of superplasticised concrete[27].

Cement C
1/2-in. Limestone E
Sand B
Superplasticizer B
Slump = 4-5 in.
w/c ≈ 0.30

Cement Content (sacks/cu.yd.)		Coarse/Fine Aggregate Ratio (lb/lb)		
		1.0	1.5	2.0
	7.0	Workable	Slightly Rocky	Harsh --- Unworkable
	8.5	Slightly Sticky	Workable	Slightly Rocky
	10.0	Sticky --- Unworkable	Slightly Sticky	Workable

Table 3.23: Effect of coarse-fine aggregate ratio and cement content on the workability of superplasticised concrete mixes made with 1/2 in. (12.7 mm) limestone[27].

The optimum amount and size of coarse aggregate for a given sand will depend to a great extent on the characteristics of the sand. Most particularly it depends on the fineness modulus (FM) of the sand. This is apparent from Table 3.12, shown on page 167, which is taken from ACI 211.1[48]. Tests made by Bloem and Gaynor[142] on normal strength concrete showed that concrete-mixing water requirements per cubic metre of concrete increased by 2.7 kg for each one per cent increase in the void content of the sand. One reference[235] suggests that the proportion of coarse aggregate, again shown in Table 3.12, might be increased by up to 4 per cent if sands with low void contents are used. If the sand particles are very angular, then it is suggested that the amount of coarse aggregate should be decreased by up to 4 per cent from the values in the table. Such adjustments in the proportioning of coarse aggregate and sand have been intended to produce concretes of equivalent workability, although such changes will alter the water demand for a given slump. When more or less water is needed in a given volume of concrete, to preserve the same consistency of paste, it is also necessary to adjust the amount of cement or cementitious materials if a given water-binder ratio is to be maintained.

(IV) Test age.

The selection of mix proportions can be influenced by the age at which a given strength performance is required, which varies depending upon the construction requirements[1]. Most often the testing age has been thought to be the age at which the acceptance criteria are established, for example at 28-days, but this can vary depending upon the type of information required.

Prestressed concrete operations may require strengths in 12 to 24 hours. Special applications for early use of machinery foundations, pavement traffic lanes, or slip formed concrete have also required high strengths at early ages. Post-tensioned concrete is often stressed at ages of approximately 3-days and requires relatively high strengths. The optimum materials selected, and the mix proportions, may therefore vary for different test ages.

A very common test age for compressive strength of concrete has been 28-days. This has produced good results for concretes within the lower strength ranges. High strength concretes gain considerable strengths at later ages and, therefore, are evaluated at later ages such as 56 or 90-days. This has often been justified where high strength concrete has been placed in columns of high-rise buildings where full loadings may not occur until later ages. Testing at later ages has been desirable in order to take advantage of long-term strength gains so that efficient use of construction materials was achieved[1].

3.3.3 Conclusions.

Production of high strength concrete not only requires materials of the highest quality, but also their optimum proportioning. However, the process of arriving at the right combination of cement, aggregate, water, and admixtures is not easy because it involves the art of balancing various conflicting requirements. Most mix proportions are, therefore, developed from extensive laboratory testing, and these proportions are generally applicable to a narrow range of locally available materials used in the tests[46]. This has also made difficult the development of a standardised mix design procedure for high strength concrete, which is therefore lacking. Obviously, a simple and widely applicable method is desired by which the mix proportions for the first trial batch may be computed without the need for costly laboratory testing.

CHAPTER 4.
OBJECTIVES AND SCOPE OF INVESTIGATION.

4.1 INTRODUCTION.

From the review of literature presented in the previous two chapters, it was clear that despite the amount of research that has been performed on high strength concrete there are still many areas where research is needed in order to ensure the economical and proper application of the material. There are definite advantages, both technical and economical, in using high strength concrete in structures today. However, many proponents of high strength concrete also indicate that it has certain disadvantages. Possible drawbacks often result from a lack of research on how they can be alleviated.

Many reports have listed the research needs for high strength concrete(2,10,12,51,67,90,237). One of the most recent ones, only published in September 1991 by Carino and Clifton(90), is the summary of a workshop which has "developed a listing of critical research needs to overcome the technical barriers and provide a sound basis for new standards". These are shown in Table 4.1. The need for an improved proportioning methodology was identified by the author of this thesis in 1988 and, as a result, the initial aim of the research reported herein was to attain a systematic procedure for optimising the mix proportions of concrete with strength in excess of 60 N/mm^2.

Having optimised the cement and aggregate contents, partial cement replacement by PFA, GGBFS, and CSF, at low water-cement ratios, was investigated, in order to assess:

(i) their effect on the workability of concrete,

(ii) their contribution to reducing the heat evolution of concrete mixes,

and, (iii) their contribution to long term strengths.

In this chapter the main points from the literature review are summarised, followed by the consequent objectives of the current research.

Topic	Need
Materials & proportioning	Improved proportioning methodology Evaluation methods Statistical methods for mixture optimization Understanding effects of materials on properties of HPC
Processing & curing	Understand of rheology and workability Effects of mixing on properties Effects of curing conditions Effects of extended setting time Field studies for guidelines on mixing and placing
Mechanical properties & test methods	Standards for compressive strength testing Relationships among mechanical properties In-place strength tests Guidelines for core testing Understanding of elastic modulus Thermal effects of HPC
Durability & test methods	Development of durability design standard Methodology for service life prediction Improved understanding of degradation mechanisms Improved test methods to evaluate durability potential Field test for permeability Methodology to predict the freezing and thawing resistance
Structural performance & design	Monitoring of HPC structures Seismic performance of HPC structures High-strength lightweight concrete Behavior and design criteria under axial load plus bending Development length and details for reinforcement Behavior and design criteria under shear and torsion

Table 4.1: A listing of the critical research needs for high strength concrete, (after Carino and Clifton[90]).

4.2 OBJECTIVES.

4.2.1 Mix design for high strength concrete.

The production of high strength concrete that meets requirements for workability and strength development places more stringent requirements on material selection than for lower strength concretes. At the same time, the proportioning of the materials becomes a critical process. Usually, specially selected pozzolanic and chemical admixtures are employed, and the attainment of a low water-binder ratio is considered essential[1].

Previous attempts to develop mix design procedures for high strength concrete have generally been based on an empirical extension of the current ACI[48] and BRE[47] procedures for the design of normal strength concrete mixes, that are themselves based on compilations of a large number of experimental data. This empirical approach becomes complicated as the production of high strength concrete usually employs pozzolanas and water-reducers or superplasticisers, the latter not covered by the current BRE mix design method for normal strength concrete[47]. Also, because of the variability in physical properties and availability of concrete making materials in different regions, mix design procedures based on the empirical extension of the current procedures become even less precise. Since there are no generally accepted mix design procedures for high strength concrete, many trial mixes have often been required to generate the data that enables the researcher to identify optimum mix proportions, as can be seen, for example, from the research carried out prior to the construction of the First Republic Bank Plaza, the proposed Dallas Main Center Project[59], (see page 27), and the research carried out by Peterman and Carrasquillo[27], (see page 27). Obviously, a simple and widely applicable method is desired by which the mix proportions for the first trial batch may be computed without the need for costly laboratory testing.

The initial objective of this research programme was, therefore:

> To try and establish a systematic procedure for attaining high strength concrete, i.e. 60 N/mm^2 and above, with readily available materials using

conventional mixing, placing and curing procedures.

4.2.2 Workability of high strength concrete.

The attainment of a low water-binder ratio is considered essential in achieving high strength concrete. Data from non-superplasticised concretes indicate that PFA[150,151] and GGBFS[162] will in general permit the water content of a concrete mix to be reduced without loss of workability. Superplasticisers have been shown to be compatible with PFA and GGBFS in concrete, but the benefits claimed for the superplasticisers in neat OPC concrete were not as apparent in PFA and GGBFS mixtures, particularly with respect to water reductions and duration of increased plasticity. The low water reductions were attributed to the lower water requirement for PFA and GGBFS concrete as compared to plain concrete for equal consistencies; since there was less excess water initially available, the addition of water-reducers was less effective. The "highly plastic phase" obtained with the use of superplasticisers was found to diminish after 15 minutes and to cease after about 30 minutes with PFA concrete[155].

Although it has been widely accepted that the use of CSF in concrete reduces workability and requires a consequent addition of a superplasticiser, two teams from France[173,174] have shown that above a certain superplasticiser dosage CSF has a water reducing effect. They claim that the use of ultrafine particles, i.e. grain size smaller than that of cement, facilitates the production of low water-binder ratio concrete by the filler effect: the grains fill the voids between those of cement thus reducing the water requirement.

According to Djellouli et al[157] combinations of CSF with PFA or GGBFS appear to be promising in the production of high strength concrete from an economical point of view. Their combined effect on workability has not, however, been thoroughly investigated.

The use of a superplasticiser becomes necessary to efficiently use all cementitious materials and to maintain the lowest practical water-binder ratio. Increased dosages above those recommended by the admixture manufacturers have been used to further decrease the water-cement ratio and, therefore, achieve higher strengths. The efficiency of superplasticisers in reducing the water-binder ratio has not, however, been investigated at dosage levels above those recommended by the admixture manufacturers. This is despite that spectrophotometrical determinations of the adsorption of superplasticisers on cementitious particles have shown possible amounts for the saturation of these particles, (see page 83).

The mix workability in terms of "stickiness" is adversely affected by the use of high cement contents and superplasticisers(1). At the same time, the placement of high strength concrete becomes a critical matter in view of the necessity to minimise crack-like voids against coarse aggregates and reinforcements(1). In order to be able to understand how differently fresh high strength concrete will behave from traditional concrete and to be able to optimise the mix design according to the requirements given by the methods of placement and compaction, there is a need for a fundamental knowledge of how to measure the workability(2). Many researchers(2,16,49,50) have advocated that the slump may give the wrong impression of the workability of high strength concrete. Knowledge of the workability of high strength concrete, should if possible, be expressed according to classical rheological models such as for example those of Bingham or Newton(2).

The objectives here for studying the workability of high strength concrete were:

1. to quantify the reduction of superplasticiser dosage required for constant slump by the use of PFA, GGBFS and CSF.

2. to determine whether PFA, GGBFS and CSF reduce the workability loss of superplasticised concrete.

3. to assess the efficiency of superplasticisers, at dosage levels above those recommended by the admixture manufacturers, in reducing the water-cement ratio while retaining a constant slump.

4. to express the workability of high strength concrete according to the

Bingham model.

4.2.3 Heat evolution due to hydration.

The high cement contents that have generally been used for the production of high strength concrete can lead to high heat of hydration temperature rises. These have caused significant concern, e.g., during the construction of the Columbia Center[57], Scotia Plaza[61], First Republic Bank Plaza[60], and the Texas Commerce Tower[58] and during trials by the British Cement Association, as has been mentioned in Section 2.3.4.

Although high early temperature rises can be moderated by the use of cooled or iced mix water or liquid nitrogen cooling of the concrete these precautions if available add considerably to the cost. The most economic way of moderating the heat of mass concrete for dams has been the use of low heat binders, e.g., PFA and GGBFS. However, despite the use of these in high strength concrete, the peak temperatures recorded are as high as 90^0C. This therefore raises some doubts as to the effectiveness of PFA and GGBFS in reducing the temperature rise of high strength concrete mixes.

Therefore, the objective was:

> To assess whether the effectiveness of partial cement replacement by PFA, GGBFS or CSF in reducing the temperature rise of high strength concrete is as significant as that monitored for normal strength concrete.

4.2.4 Strength development characteristics.

Wittmann[136] stated that the strengths of aggregates are decisive for determining the ultimate load-bearing capacity of high strength concrete. However, the compressive strength of concrete, it being a heterogeneous material, does not only depend on the strength of its constituent materials, i.e. mortar and coarse aggregate, but also on the bond strength of the mortar to coarse aggregate. The latter can be improved, as has been mentioned on page 94, by an increase in the roughness of the coarse aggregate and a reduction in the water-binder ratio. The compressive strength of non-superplasticised concrete also increases gradually as the maximum size of coarse aggregate decreases(140,141,142). As has been shown in Section 3.2.4, Part II, this effect may be eliminated by the use of a superplasticiser above a certain dosage, in combination with a good quality crushed aggregate and the use of a mineral admixture.

The use of a coarse sand, with an FM of about 3.0, in superplasticised high strength concrete may be advantageous in reducing the "stickiness" and the water demand of the concrete(11,104,108), as has been mentioned in Section 3.2.5. The increased water demand resulting from the use of finer sands can, however, be counteracted in high strength concrete by increasing the superplasticiser dosage. The effect of increasing fineness modulus of sand, without changing the water-cement ratio, on the compressive strength of concrete has not been sufficiently investigated. The data are limited and inconclusive.

It has always been thought necessary to use mineral admixtures in the production of high strength concrete. For example, according to Day(143):

> "The chosen mix was a "maximum strength" design in that cement contents higher than the 520 kg/m^3 used did not give a higher strength (although different admixtures now would). PFA was not available at the time."

The other mineral admixture that its use has been widely advocated is CSF. Indeed Bennett(35) feels it is needed for concrete over 60 N/mm^2, although he accepts that: "that boundary could be pushed a little more now". On the contrary, other investigations indicate that the effect of CSF is either small(190) or negative(132,169). As has been mentioned on page 145, there are indications that the efficiency of CSF in impoving the strength may be influenced by the binder, superplasticiser(190), water-binder ratio, and the level of cement replacement(169).

GGBFS, according to ACI Committee 363(1), shows particular promise for high strength concrete production. However, there have been very few investigations on its use in high strength concrete. It has been shown that concrete with a compressive strength of as high as 100 N/mm^2 can be produced with cement replacement by GGBFS of 50 per cent[164]. However, studies are required that will show the contribution of GGBFS to strength, i.e. the strength of GGBFS should be compared to the strength development of OPC concrete.

The objectives, therefore, were:

1. to determine the dependence of the concrete compressive strength on the crushing value, surface texture and maximum size of the coarse aggregate.

2. to determine the effect of increasing fineness modulus of sand, without changing the water-cement ratio, on the compressive strength.

3. to assess the contribution of mineral admixtures (PFA, GGBFS and CSF) to the strength of concrete with low water-binder ratios.

4.3 SCOPE OF INVESTIGATION.

It was thought worthwhile to summarize the scope of the investigation (test methods, range of variables etc.), at this point in the thesis. Details are given in subsequent chapters on each section of the work.

4.3.1 Mix design for high strength concrete.

I thought that the best way of approaching the problem of proportioning the materials for the production of high strength concrete was, first, to test concrete with mix proportions similar to those used by other workers. The variables were the water-binder ratios, binder contents, type of aggregate (gravel or granite), and maximum aggregate size (20 and 10 mm).

The efficiency of two superplasticisers (a sulphonated naphthalene formaldehyde condensate based (Conplast 430) and a sulphonated melamine formaldehyde condensate based (Conplast M1)) were investigated and the most efficient of the two was used for most of the subsequent experiments.

The approach adopted for optimising the proportions of materials for the production of high strength concrete was "The Maximum Density Theory", that has been described in Section 3.3.1. This is the requirement that the aggregate occupies as large a relative volume as possible. The relative proportions of the aggregates are chosen to produce the minimum void content. This, therefore, results in a minimum required volume of cement paste, and hence minimum cement content. The experimental programme for quantifying these factors involved, first, the measurement of the aggregate void content. Combinations of aggregates investigated were 20 and 10 mm gravel, limestone and granite with a fine sand (FM of 2.1). A small quantity of coarse sand (FM of 2.73) became available to me and a combination of this with 10 mm granite was also investigated.

The relation between the percentage overfill of voids by pastes required to "fluidify" the concrete with the combination of aggregates that produced the minimum void content were investigated. It was apparent that the "Maximum Density Theory" does not consider the effect that the aggregate surface area has on the requirement of excess paste for lubrication, and it therefore needed modification. To investigate the combined effect of void content and surface area, mixes with lower sand proportions than that required for minimum void content were tested for slump. The optimum sand proportion is the one that produces the highest slump, for a particular cement content. The optimum sand proportion was investigated for different water-cement ratios, maximum aggregate size, fineness modulus of sand, and the use of CSF to partially replace cement. The variables investigated are listed in Table 4.2.

STAGE.	SCOPE OF INVESTIGATION.	VARIABLES INVESTIGATED.
1.	Applicability of other workers' mix designs to our materials.	Water-binder ratios, cement contents, PFA, GGBFS, CSF, gravel or granite aggregate, 20 and 10 mm maximum aggregate size.
2.	The applicability of the "Maximum Density Theory" as an approach to optimisation of the material proportions for the production of high strength concrete.	(i) Measurement of the aggregate void content. Combinations of aggregates investigated were 20 and 10 mm gravel, limestone and granite, with a fine sand (FM of 2.1). Combinations of a coarse sand with 10 mm granite were also investigated. (ii) The efficiency of two superplasticisers (a sulphonated naphthalene formaldehyde condensate based - Conplast 430, and a sulphonated melamine formaldehyde condensate based - Conplast M1) were investigated to find the most efficient for use in subsequent experiments. (iii) Relation between the percentage overfill of voids by pastes required to "fluidify" the concrete. Water-cement ratios of 0.30, 0.28, 0.26 with 2 % solids of Conplast 430 by weight of cement were investigated.
3.	Modification of the "Maximum Density Theory" so that it considers both the void content and the surface area of the aggregate.	Lower sand proportions than that required for minimum void content were tested for slump in order to determine the optimum sand proportion. Different water-cement ratios, maximum size of aggregates, fineness modulus of sand, and the use of CSF to partially replace cement were investigated.

Table 4.2: Stages involved in the development of the "Modified Maximum Density Theory", i.e. a systematic procedure for optimising the mix proportions of high strength concrete.

4.3.2 Workability of high strength concrete.

(i) Quantification of the reduction of the superplasticiser dosage for constant slump by the use of PFA, GGBFS and CSF.

Having optimised the aggregate proportions for ordinary Portland cement concrete the superplasticiser dosage required for 150 mm slump concrete was investigated for various water-binder ratios and with partial cement replacement by PFA, GGBFS and CSF. A combination of GGBFS with CSF to partially replace cement was also investigated. The test variables are given in Table 4.3. A naphthalene based superplasticiser (Conplast 430) was used in all experiments.

(ii) Workability loss.

In order to determine whether PFA, GGBFS and CSF reduce the rate of workability loss of superplasticised concrete the mixes shown in Table 4.3 were tested for slump at ten minutes intervals after the initial 5 minutes mixing period.

(iii) Efficiency of superplasticiser.

To achieve the third objective in Section 4.2.2, the maximum amount of a naphthalene based superplasticiser adsorbed on the surface of cement and CSF was investigated spectrophotometrically. This was related to the results of Section (i), i.e. to the superplasticiser dosage required to reduce the water-binder ratio while retaining the slump at 150 mm.

No.	OBJECTIVE.	WATER-BINDER RATIO	MINERAL ADMIXTURE TYPE.	LEVEL OF CEMENT REPLACEMENT.	EXPERIMENTAL DETAILS.
1.	Quantification of the water reduction achieved by the use of PFA, GGBFS, and CSF.	0.38 - 0.20	PFA	0,10, 20, 30, 40	The superplasticiser dosage required to reduce the water-binder ratio while retaining the slump at 150 mm was investigated.
			GGBFS	0,10, 20, 30, 45, 60	
			CSF	0,5, 10, 15	
			GGBFS/CSF	50/10	
2.	Workability loss.	0.26	OPC		Concrete was tested for slump at ten minutes intervals, beginning afer the initial 5 minutes mixing period. The original slump was around 200 mm.
			PFA	40	
			GGBFS	30, 60	
			CSF	10	
3.	Efficiency of superplasticiser (Conplast 430)				The maximum amount of superplasticiser adsorbed on the surface of OPC and CSF has been investigated spectrophotometrically, and has been related to the results of Section (i).
4.	Workability of high strength concrete according to the Bingham model.	0.38, 0.35, 0.32, 0.30, 0.28, 0.26			Tattersall's two point test was used to measure the yield stress (g) and plastic viscosity (h) values for concretes of 150 mm slump.
		0.26	PFA	20, 30, 40	
		0.38, 0.32, 0.23	PFA	40	
		0.26	GGBFS	15, 30, 45, 60	
		0.38, 0.26	CSF	5, 10, 15	
		0.35, 0.32, 0.29	CSF	10	
		0.26, 0.23, 0.20	GGBFS/CSF	50/10	
		0.38, 0.35, 0.32, 0.29, 0.26	Coarse sand (FM of 2.73)		
		0.32, 0.29, 0.26	30 per cent sand.		

Table 4.3: Range of tests for the assessment of the workability of high strength concrete.

(iv) Workability of high strength concrete according to the Bingham model.

The workability of high strength concrete was expressed according to the Bingham model with tests carried out using Tattersall's two point test. The concrete mixes studied were designed according to the "Modified Maximum Density Theory", with a constant overfill by paste of 4 per cent, and at a slump of 150 mm. The variables in the concrete compositions investigated are shown in Table 4.3.

4.3.3 Heat evolution due to hydration.

Adiabatic temperature rises were measured for mixes with water-binder ratio of 0.26. Variables investigated were levels of cement replacement by PFA, GGBFS or CSF, with casting temperatures of 10, 20 and 30^{0}C. Mix parameters are given in Table 4.4.

4.3.4 Strength development characteristics.

(i) The effect of coarse aggregate properties and mix design procedure on the compressive strength of concrete.

Although the production of high strength concrete places more stringent requirements on material selection than for lower strength concretes, we should always keep in mind the economic advantages of using locally available materials. Gravel aggregate was the most readily available material, and samples of limestone and granite were obtained from ARC. The effect of these three types of aggregates, with different crushing values, and surface textures, on the compressive strength of concrete was investigated. The variables investigated are shown in Table 4.5. At this stage of the work the "Modified Maximum Density Theory" had not yet been developed, and the mixes were therefore designed according to the "Maximum Density Theory".

No.	OBJECTIVE	LEVEL OF CEMENT REPLACEMENT. (%)	WATER-BINDER RATIO.	CASTING TEMPERATURE. (°C)
1.	Effect of water-cement ratio and casting temperature.		0.26	10, 20, 30
			0.38	20
2.	Effect of PFA.	10, 20, 40	0.26	10, 20, 30
3.	Effect of GGBFS.	10, 30, 60	0.26	10, 20, 30
4.	Effect of CSF.	10	0.26	10, 20, 30
5.	Effect of a combination of PFA/CSF.	36/10	0.26	10, 20, 30
6.	Effect of a combination of GGBFS/CSF.	54/10	0.26	10, 20, 30

Table 4.4: Variables for adiabatic temperature tests.

No.	OBJECTIVE	Type and size of aggregate.	Water-binder ratio.	Mineral admixture type.	Level of cement replacement. (%)	Mix design method.	Testing ages. (days)
1.	Preliminary tests to investigate the effect of coarse aggregate properties.	20mm gravel	0.38 - 0.26			Maximum Density Theory	28
		10mm gravel	0.38 - 0.26				28
		20 mm gravel	0.26	0.26	5, 10		3, 7, 28, 56
		20mm limestone	0.38 - 0.28				28
		10mm limestone	0.38 - 0.28				28
		20mm granite	0.38 - 0.28				28
		10mm granite	0.38 - 0.28				28
		10mm granite	0.38 - 0.23			Modified Maximum Density Theory	7, 28, 56, 91, 180
2.	The effect of PFA.	10mm granite.	0.38, 0.32, 0.26	PFA	10, 20, 40		7, 28, 56, 91, 180
3.	The effect of GGBFS.	10mm granite.	0.38, 0.32, 0.26	GGBFS	10, 30, 60		
4.	The effect of CSF.	10mm granite.	0.38, 0.35, 0.32, 0.29, 0.26, 0.23	CSF	5, 10, 15		
5.	The effect of PFA/CSF.	10mm granite.	0.38 0.26	PFA/CSF PFA/CSF	38/5 36/10		
6.	The effect of GGBFS/CSF.	10mm granite.	0.38 0.26	GGBFS/CSF GGBFS/CSF	57/5 57/5, 54/10		
7.	The effect of maximum size of aggregate.	20mm granite	0.38, 0.32, 0.26				7, 28, 56, 91, 180
		20mm granite	0.38, 0.32, 0.26	CSF	10		
8.	The effect of fineness modulus. Mixes with sand of FM = 2.73.	10mm granite	0.38, 0.32, 0.26				
		10mm granite	0.38, 0.32, 0.26	CSF	10		

Table 4.5: Mix variables and strength ages for compressive strength tests.

Observation of the concrete made with gravel aggregate after testing showed that the failure of the concrete was initiated at the mortar-aggregate interface, i.e. in the transition zone. Several researchers[196,201-205] have shown that the microstructure of the transition zone can be improved by the use of CSF, and this offered me the challenge of using CSF in order to change the failure mode of the specimens and make the concrete rupture through the coarse aggregate. Two levels of cement replacement by CSF were investigated as shown in Table 4.5.

It was unfortunate that soon after I completed this first series of experiments I was informed that the ARC depot at Greenwich, supplying the limestone, had closed down. Although their West Drayton depot could supply limestone it would have been from a different source. This would have meant repeating the first series of tests with resulting delays. Since the concrete with granite aggregate showed equally good compressive strengths as that with the limestone, as will be shown in Chapter 9, the rest of the research programme was carried out using granite aggregate.

The "Maximum Density Theory" was later modified and was used to design mixes with 10 mm granite.

(ii) Contribution of mineral admixtures to the strength of concrete with low water-binder ratios.

Three water-binder ratios, i.e. 0.38, 0.32 and 0.26, were considered essential in determining not only the contribution of mineral admixtures to the strength of concrete but also to show if their contribution is affected by the water-binder ratio. Additional water-binder ratios of 0.29 and 0.23 were used for CSF concrete.

Workable concretes with water-binder ratios of as low as 0.20 became possible with the use of CSF, CSF-GGBFS and CSF-PFA combinations. The strength development of these mixes was also investigated.

All mix variables and testing ages that were investigated are given in Table 4.5.

(iii) The effect of maximum aggregate size.

The effect of maximum size of aggregate on the compressive strength of concrete was also investigated with mixes designed according to the "Modified Maximum Density Theory". The increased strengths achieved by the use of CSF in the preliminary tests led me to believe that this mineral admixture could alleviate the strength reduction, if any, resulting from the use of a larger size of coarse aggregate. Two series of mixes, one with OPC and the other with CSF, were therefore investigated.

(iv) The effect of increasing fineness modulus of sand.

Although a coarse sand (FM of about 3.0) has been widely recommended for use in the production of high strength concrete[1,44,102,104,108], the only type of sand that my supplier could provide me was a fine sand (FM of 2.1). Supplies of a coarser sand proved difficult to find. However, later in the research programme, a small quantity of a coarse sand with fineness modulus of 2.73 became available. It was therefore decided to investigate the effect of increasing fineness modulus of sand on the compressive strength of concrete. Three water-cement ratios were therefore investigated, i.e. water-cement of 0.38, 0.32 and 0.26. Mixes with 10 per cent CSF were also investigated for the same reason as with its use in mixes with different maximum size of aggregates.

CHAPTER 5.

MATERIALS AND EXPERIMENTAL PROCEDURES.

5.1 INTRODUCTION.

In this chapter the materials and the procedures for mixing, casting and curing the concrete specimens are described. The test procedures for density and compressive strength measurements are also described, while others, such as the two point workability test and the adiabatic temperature measurements, are more conveniently included in the relevant subsequent chapters.

5.2 MATERIALS.

5.2.1 Ordinary Portland cement (OPC).

Three batches of typical Ordinary Portland Cement (OPC), obtained from Rugby Cement plc., were used during the testing programme. This cement was suggested by the manufacturer as representing a "typical average" of that available in the United Kingdom. The chemical and phase analyses are given in Table 5.1, and the cement conforms with the requirement of the British Standard BS-12[3].

The cement was kept in large air tight drums (up to 500 kg) for long term storage. A small drum, capacity 50 kg, was used for short term storage, thus avoiding opening a large drum every time a mix was carried out.

5.2.2 Mineral admixtures.

Condensed silica fume (CSF) was obtained as a 50/50 mixture with water, from Elkem Chemicals who specified the specific gravity of this slurry to be between 1.38 and 1.40. The chemical analysis of the powder used in the slurry is given in Table 5.1. The slurry

	Ordinary Portland Cement[1] (% by weight)			Pulverised Fuel Ash[2] (% by weight)	Ground Granulated Blastfurnace Slag[3] (% by weight)	Condensed Silica Fume[4] (% by weight)
	BATCH 1 DEC. 89	BATCH 2 NOV. 91	BATCH 3 MARCH 91			
CaO	64.8	65.74	64.9	1.49	37.92	---
SiO_2	21.0	20.35	20.27	50.5	35.95	96.6
Al_2O_3	4.5	5.29	5.19	25.0	12.27	1.7
Fe_2O_3	2.2	2.65	2.68	9.95	0.56	0.09
SO_3	2.6	2.67	2.83	---	0.23	---
MgO	1.6	1.13	1.07	1.34	8.35	0.11
K_2O	0.65	0.42	0.5	3.63	0.47	---
Na_2O	0.23			1.47	0.36	---
IR	0.3	0.6	0.4	---	0.44	---
LOI	0.9	1.64	0.89	---	1.56	1.55
Surface area (m^2/kg)	398	314	308	---	432	---
PHASE COMPOSITION BY BOGUE EQUATION						
C_3S	57	61.9	59.2			
C_2S	18	11.7	13.5			
C_3A	8.4	9.5	9.2			
C_4AF	6.6	8.0	8.1			

(1) Rugby Cement plc.
(2) Building Research Establishment.
(3) Frodingham Cement Co. Ltd..
(4) Elkem Chemicals, High Wycombe, England.

Table 5.1: Chemical analyses of OPC, PFA, GGBFS and CSF.

was delivered in 25 litre plastic containers.

Ground granulated blast furnace slag (GGBFS) was supplied in powder form by Frodingham Cement Co. Ltd., under the marketing name of "Cemsave". The chemical analysis is given in Table 5.1. It was delivered in 25 kg bags and was stored in small sealed drums, capacity 30 kg.

Pulverised fuel ash (PFA) was supplied in powder form by the Building Research Establishment (BRE). The chemical analysis is given in Table 5.1. It was delivered in 25 kg sealed drums.

5.2.3 Water.

Tap water was used throughout for mixing and curing.

5.2.4 Chemical admixtures.

Two commercially available superplasticisers, one of each main type, and a retarder were used. All three were marketed by Fosroc Construction Chemicals and complied with BS-5075(114) and ASTM C-494(238). They were supplied as liquids and their specific gravity and active solids content are given in the Table 5.2.

The water content of these chemical admixtures was taken into account when calculating the total water content of the concrete.

5.2.5 Fine and coarse aggregate.

For most of the programme Thames Valley sand with the grading shown in Table 5.3 was used. The fineness modulus (FM) of 2.1 was lower than ideal, but alternative supplies

Trade Name	Superplasticiser Type	Specific Gravity at 20^{o}C	Active Solids (% by weight)
Conplast M1	Sulphonated Melamine Formaldehyde Condensate	1.10	20
Conplast 430	Sulphonated Naphthalene Formaldehyde Condensate	1.20	40
Conplast R	Hydroxycarboxylic Acid	1.19	25

Table 5.2: Types and physical properties of the chemical admixtures.

Sieve size (mm)	Percentage by mass passing BS sieve.					
	Grading Limits (from BS 882: 1983).				Fine Sand from Thames Valley	Coarse Sand Land Based
	Overall limits	Coarse	Medium	Fine		
10	100	---	---	---	---	---
5	89 - 100	---	---	---	100	100
2.36	60 - 100	60 - 100	65 - 100	80 - 100	91.4	87.5
1.18	30 - 100	30 - 90	45 - 100	70 - 100	82.7	74.8
0.600	15 - 100	15 - 54	25 - 80	55 - 100	69.3	54.8
0.300	5 - 70	5 - 40	5 - 48	5 - 70	39.0	9.8
0.150	0 - 15	---	---	---	7.6	0.4
0.075	---	---	---	---	0.7	0.1
			Fineness Modulus (FM):		2.10	2.73

Table 5.3: Sieve analysis for the two types of sand.

proved difficult to obtain. However, later in the research programme I managed to obtain a sample of a coarser sand, with a fineness modulus of 2.73 - the grading is also shown in Table 5.3.

The 20 and 10 mm gravel were also from Thames Valley.

The limestone aggregate is from the Limestone Ex Whatley Quarry Frome, in Somerset, and the granite from Prophyry Ex Penmaenmawr Quarry in North Wales. They were both supplied by ARC Ltd..

The coarse aggregate gradings are shown in Table 5.4. Results of tests carried out in accordance with BS 812: 1975[137] are shown in Tables 5.5 and 5.6.

Both the fine and the coarse aggregates were oven dried before the tests, and therefore allowance was made for absorption in calculating the batch weights of mixes.

5.3 MIXING PROCEDURES.

Three horizontal pan mixers, of capacities 0.01, 0.02 and 0.05 m^3, were used, according to the size of the batch, for all concrete mixing.

The 20 mm aggregate, if used, was placed first in the mixing drum, followed by the 10 mm aggregate, the cement, the PFA or GGBFS if used, the sand and finally the superplasticiser previously mixed with the water. Mixing was then initiated. If CSF was used it was added slowly to the rotating drum, its addition starting as soon as the concrete had a wet appearance, usually not more than half a minute after initiation of mixing.

The following points about the addition procedure of the superplasticiser are worth noting:

(i) Rixom and Dodson[134] stated that if the admixture is added just at the end of the mixing time of aggregates, cement and total gauging water, greater adsorption of

Sieve size (mm)	Percentage by mass passing BS sieves for nominal sizes.							
	Limits for single-sized aggregate (from BS 882)		Gravel		Limestone		Granite	
	20 mm	10 mm	20 mm	10 mm	20 mm	10 mm	20 mm	10 mm
50								
37.5	100		100		100		100	
20	85 - 100		94	100	68	100	94	100
10	0 - 25	85 - 100	9	81	1	93	1	98
5	0 - 5	0 - 25	2	12	0	18	0	6
2.36		0 - 5						

Table 5.4: Sieve analyses for the 20 and 10 mm gravel, limestone and granite.

	Gravel		Limestone		Granite		Fine Sand	Coarse Sand
	20 mm	10 mm	20 mm	10 mm	20 mm	10 mm		
Relative Density (oven dry basis)	2.53	2.53	2.65	2.62	2.72	2.71	2.39	2.40
Relative Density (saturated & surface dry basis)	2.57	2.58	2.67	2.64	2.75	2.74	2.43	2.44
Apparent Relative Density	2.62	2.67	2.69	2.68	2.78	2.79	2.50	2.49
Water Absorption (%)	1.4	2.0	0.8	0.8	0.8	1.1	1.7	1.50

Table 5.5: Relative densities and water adsorption of aggregates, (determined in accordance with BS 812: 1985).

	Gravel	Limestone	Granite
10 % Fines Value	250 kN	180 kN	320 kN
Aggregate Crushing Value	17.2	20.7	11.3

Table 5.6: Results of tests carried out in accordance with BS 812: 1985 on a sample of 6.35 mm aggregate.

the admixture on to the initial hydrates is obtained resulting in a higher workability. The water-cement ratios they had investigated were higher than 0.51. This procedure causes difficulties with high strength concrete because of the low water contents. Mixing without superplasticiser resulted in lumps of concrete forming soon after mixing began. This will deter the uniform dispersion of the superplasticiser solids.

(ii) Split addition of the superplasticiser, e.g., half of the total dosage added with the mixing water and the other half added directly to the fresh concrete after mixing starts, may result in a higher workability, (see page 92). However, the manufacturer cautioned that this procedure will present some practical difficulties in obtaining prefixed constant workability of the mixes. The same criticism has been made by Collepardi[119] for Rixom and Dodson's procedure for adding the superplasticiser.

Hence, the mixing procedure chosen is that of mixing the total superplasticiser dosage with the mixing water before being added to the mix.

5.4 SLUMP TEST.

Since the workability of high strength concrete often declines rapidly with time after mixing, this being influenced by the mineral admixture used, the slump test, described in BS 1881: Part 102[231], was performed immediately after mixing stopped. In order therefore to allow time for absorption of the mix water by the aggregate, the mixing time was increased from 3 minutes, as specified in BS 1881: Part 125[239], to 5 minutes.

5.5 CASTING AND CURING.

All compressive strength measurements were obtained on 100 mm cubes. These were cast in lightly oiled moulds and were filled in three layers and compacted on a vibrating table.

After casting, the specimens were covered with a polythene sheet, and were stored at an ambient temperature (about 20^{0}C) for about 24 hours. They were then demoulded and kept in water at 20^{0}C prior to testing.

5.6 DENSITY MEASUREMENTS.

The densities of the specimens were measured by weighing them in air and then in water. The density (ρ) in kg/m^3 is given by:

$$\rho = (W_A)/(W_A - W_W) \times 1000$$

where: W_A is the mass in air, expressed as kg,
W_W is the mass in water, expressed as kg.

5.7 COMPRESSIVE STRENGTH MEASUREMENTS.

Sufficient cubes were cast for sets of three or five to be tested using a Contest GD10A machine, shown in Figure 5.1, which complied to BS 1881: Part 4[240].

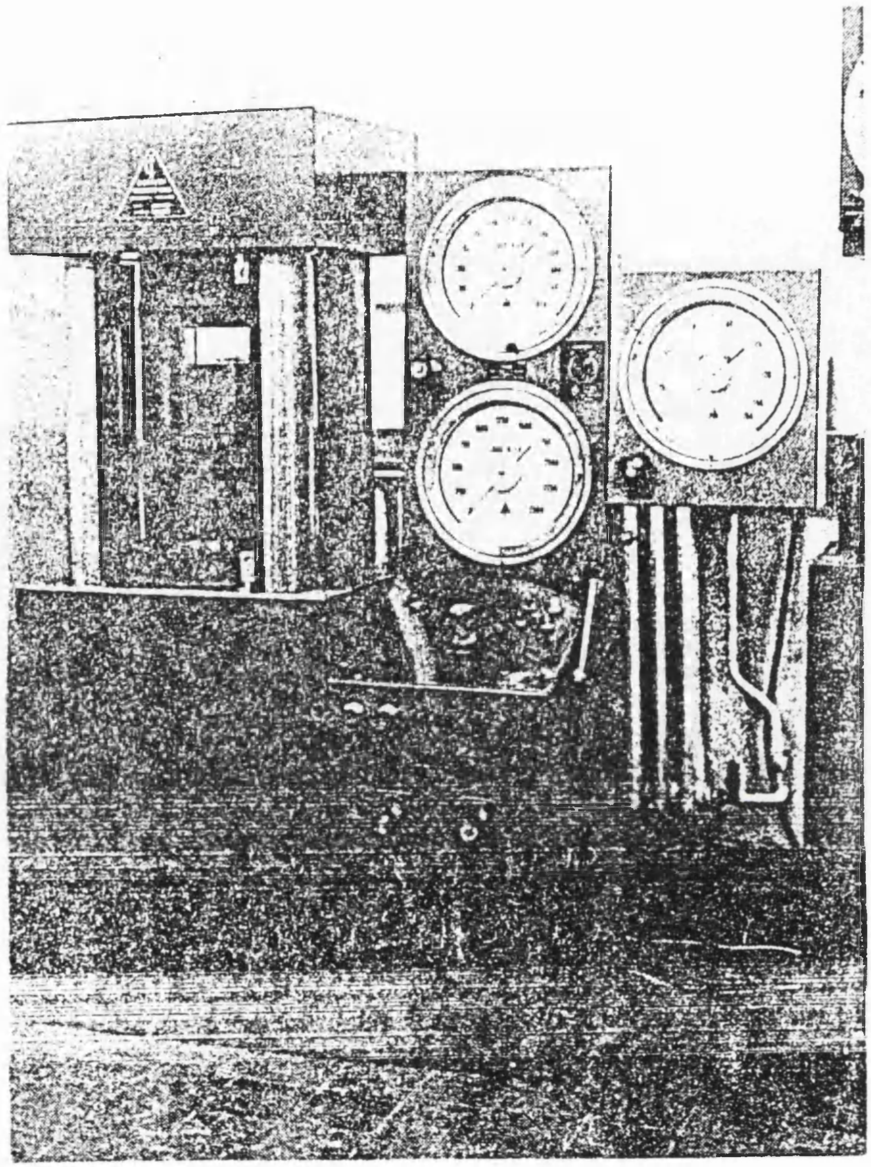


Figure 5.1: Contest GD10A compression testing machine.

CHAPTER 6.
MIX DESIGN OF HIGH STRENGTH CONCRETE.

6.1 INTRODUCTION.

The production of high strength concrete that meets requirements for workability and strength development places more stringent requirements on material selection than for lower strength concretes. At the same time, the proportioning of the materials becomes a critical process, and, therefore, many trial batches are often required to generate the data that enables the researcher to identify optimum mix proportions, as has already been mentioned in Chapter 3. This chapter describes the first part of the research programme which was aimed at producing a systematic procedure for optimising the mix proportions of high strength concrete mixtures. The stages in this were:

1. Preliminary tests of a number of concrete mixes with proportions similar to those used by other workers, to obtain an initial understanding of the strengths that could be obtained with our materials. Variables investigated include water-binder ratios, cement contents, partial cement replacement by PFA, GGBFS and CSF, and type and maximum size of coarse aggregate.

2. Testing of mixes designed using the "Maximum Density Theory" approach, which has been described in Section 3.3. Combinations of coarse and fine aggregates to produce minimum void contents were determined, and the effect of the amount of "overfill" of these voids by cement paste on slump was measured. Pastes with water-cement ratios of 0.30, 0.28 and 0.26, and the effect of type and dosage of superplasticiser were tested.

3. Development of a "Modified Maximum Density Theory", by investigating the effect the sand proportion has on the slump versus overfill relationship. For sand proportions of 35 to 45 per cent, the effect of water-binder ratios of 0.38, 0.32 and 0.26, 20 and 10 mm granite, two sands with fineness moduli of 2.73 and 2.1, and

partial cement replacement by 10 per cent CSF were investigated.

6.2 PRELIMINARY TESTS.

The preliminary tests investigated the strengths readily achievable with the materials available to us, with mix proportions similar to those used by other workers. The mixing, casting and curing procedures were as described in Chapter 5. Sets of three 100 mm cubes were tested for compressive strength at 28-days. A summary of the mix designs and the strengths obtained are shown in Tables 6.1 and 6.2.

It can be seen that concretes with high cementitious contents (600 kg/m^3), mixes A-1, A-3, A-4 and A-5, despite the low water-cement ratios, did not produce significantly higher strengths than the concretes with cement contents of 425 kg/m^3, mixes A-2 and A-6. This therefore confirmed that the maximum strength may not always be increased by the use of cement added to the mixture beyond an optimum cementitious content, as discussed in Section 3.3.2.

Mix Nos A-1 (with 10 mm gravel) and A-3 (with 20 mm gravel) showed that there may not be any strength reduction when using 20 mm maximum size of aggregate in combination with a superplasticiser. This is in agreement with Bedard and Aitcin[105] and Cook[59], (see page 104).

When the mixes with granite aggregate, (Mix Nos A-5, A-6 and B-7 to B-9), are compared to the gravel mixes, (Mix Nos A-1 to A-4, and B-1 to B-6), it is apparent that granite gives stronger concrete. This is in agreement with two other studies[76,104] that have shown that crushed stone produces higher strengths than rounded gravel, (see page 96).

When the compressive strengths obtained with ordinary Portland cement mixes are superimposed on the data gathered by Parrott[32], (Figure 6.1), it is seen that they are in the lower part of the compressive strength envelope, both for the gravel and the granite aggregates. Observation of the concrete after testing showed that the failure was initiated

Mix No.	OPC kg/m^3	Total water kg/m^3	Coarse aggregate kg/m^3	Fine aggregate[(1)] kg/m^3	SPA dosage[(2)]	W/C ratio	Slump (mm)	28-day strength (N/mm^2)
A-1	600	170	965 10-5 mm gravel	645	1.20	0.267	(*)	78
A-2	425	185	1265 10-5 mm gravel	650	0.90	0.381	50	72
A-3	600	182	965 20-5 mm gravel	645	0.70	0.278	135	76
A-4	600	175.5	970 10-5 mm gravel	645	2.00	0.259	over 150	61
A-5	600	163	965 10-5 mm granite	645	1.60	0.247	(*)	84
A-6	425	146	1265 10-5 mm gravel	650	1.40	0.320	10	81

(1) Sand with FM = 2.1.
(2) Superplasticiser (Conplast M1) dosage expressed as a percentage of solids by weight of cement.
(*) Zero to 5 mm slump.

Table 6.1: Preliminary mix designs and strengths obtained for OPC concrete.

Mix No.	Total binder kg/m³	OPC kg/m³	Mineral admixture kg/m³	Total water kg/m³	Coarse aggregate kg/m³	Fine aggregate(1) kg/m³	SPA dosage(2)	W/B ratio	Slump (mm)	28-day strength (N/mm^2)
B-1	578	525	52.5 CSF	164	1240 10-5 mm gravel	620	0.90	0.247	(*)	79
B-2	583	530	52.5 CSF	172	1240 10-5 mm gravel	620	1.40	0.259	50	94
B-3	427	395	32 CSF	178	1180 20-5 mm gravel	680	0.94	0.365	40	81
B-4	632	474	158 PFA	197	1160 10-5 mm gravel	580	1.30	0.279	(*)	73
B-5	640	480	160 PFA	205	1160 10-5 mm gravel	580	0.60	0.281	80	72
B-6	550	330	220 GGBFS	185	1220 10-5 mm gravel	515	0.73	0.300	50	74
B-7	635	530	52.5 CSF	175	1240 10-5 mm granite	620	1.10	0.252	65	104
B-8	582.5	530	52.5 CSF	149	1240 10-5 mm granite	620	1.65	0.230	(*)	101
B-9	427	395	32 CSF	165	1180 10-5 mm granite	680	1.10	0.349	over 150	92

(1) Sand with FM = 2.1.
(2) Superplasticiser (Conplast M1) dosage expressed as a percentage of solids by weight of cement.
(*) Zero to 5 mm slump.

Table 6.2: Preliminary mix designs and strengths obtained for PFA, GGBFS, CSF concrete.

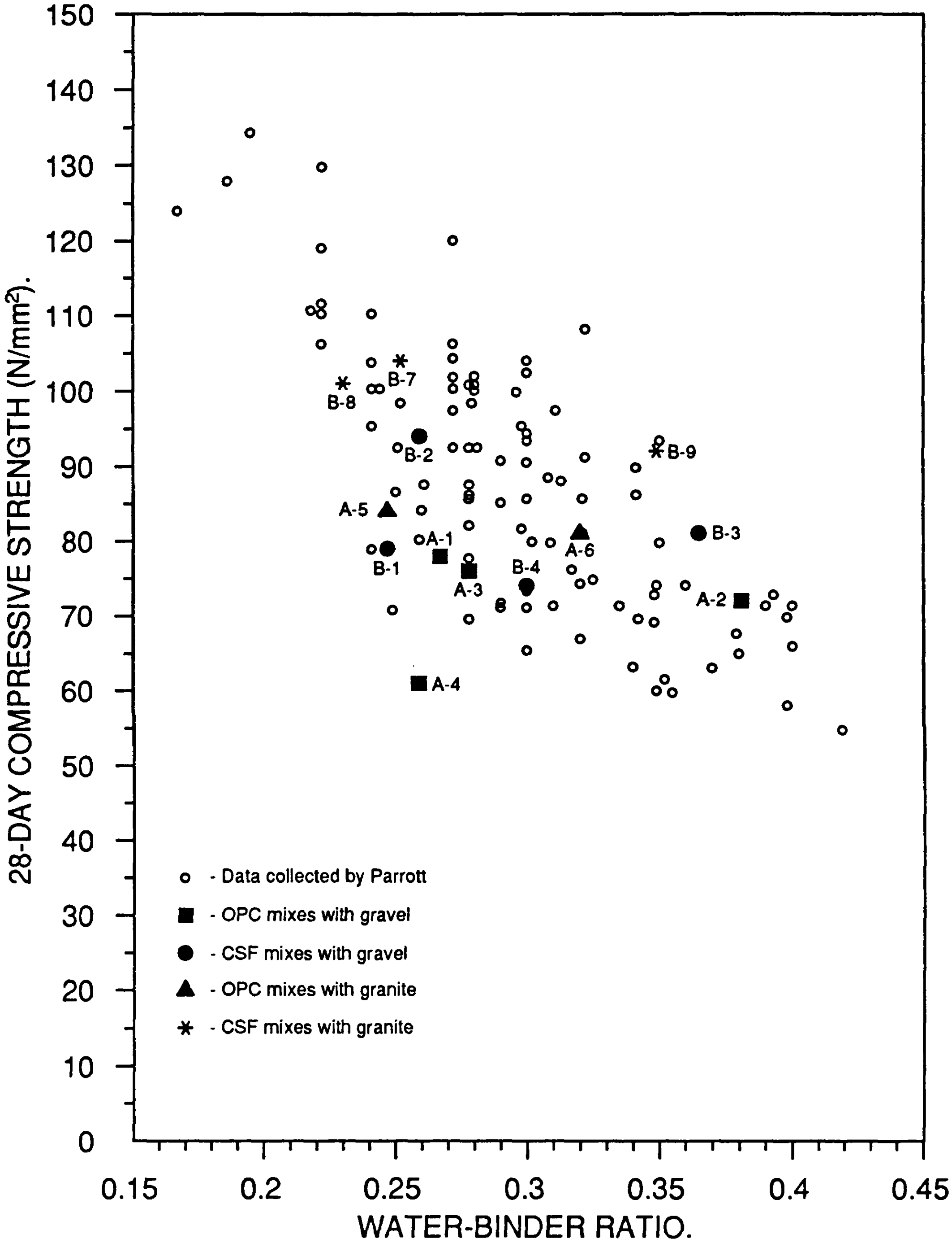


Figure 6.1: 28-day compressive strength versus water-binder ratio.

at the mortar-aggregate interface, i.e. in the transition zone.

The compressive strengths, however, of mixes incorporating condensed silica fume with high water-binder ratios of around 0.35 are high in the compressive strength envelope, as shown in Figure 6.1. The higher strengths could be due to an improvement of the mortar-aggregate bond resulting from the use of CSF, (see page 140), something that was not achieved by an increase in the cement content. For water-binder ratios close to 0.25, the granite mixes are in the middle while the gravel mixes are in the lower part of the strength envelope. This may indicate that the gravel mixes have reached the ceiling value of the aggregate strength. This was substantiated by observation of the concrete after testing; the failure plane passed through the aggregate.

These preliminary tests showed that cementitious contents higher than 425 kg/m^3 did not produce higher strengths. Indeed, several reports suggest trials with cement contents in the range 390 to 560 kg/m^3 in order to determine the optimum cementitious content beyond which the strength is not always increased, (see page 205). It was concluded, therefore, that a more detailed study was required to determine optimum cement contents, and optimum proportions of aggregates, before going on to determine the effects of water-binder ratio, type and size of coarse aggregate, sand grading, and partial cement replacement by PFA, GGBFS and CSF on the compressive strength development of concrete. Although the gravel was shown to be inappropriate for the production of high strength concrete, it was however the most readily available aggregate, while special arrangements with ARC Ltd. had to be made before I could obtain further quantities of limestone and granite. Gravel was therefore used to define a systematic procedure for optimising the proportions for high strength concrete mixes. The procedure could then easily be applied to limestone and granite aggregate.

6.3 THE "MAXIMUM DENSITY THEORY".

The approach for mix proportioning that I investigated was the "Maximum Density Theory", described in Chapter 3. This requires that the aggregate occupies as large a relative volume as possible, and therefore results in a minimum required volume of cement paste, and hence minimum cement content.

As has been mentioned in Chapter 3, this is desirable for economic reasons, but perhaps more critically, (a) to limit the maximum temperature rise during hydration, (b) to reduce the danger of alkali silica reaction, (c) to reduce the "stickiness" and loss of workability in the fresh concrete, and (d) to reduce creep and shrinkage in the hardened concrete.

The production of high strength concrete requires, firstly, a low water-binder ratio, secondly, sufficient cement paste to slightly overfill the voids, and thirdly, the use of a superplasticiser to ensure sufficient fluidity of the paste with low water-binder ratio. The first part of the experimental programme involved the determination of the proportions of coarse and fine aggregates required to produce minimum void content.

6.3.1 Void content of aggregates.

(I) Apparatus.

The void content measuring apparatus, shown in Figure 6.2, consists of a sample container which is connected by a rubber tube to a glass bulb at the top of a measuring tube. The lower end of this tube is connected to a reservoir via another rubber tube. The sample container is made of glass, graduated in 0.2 litre divisions up to 3.5 litres, fitted between the stand and an aluminium lid with air-tight rubber seal. Four screwed rods with wing nuts are used to make the container airtight.

The test procedure is:

1. The sample of material is placed in the container to the 3.5 litre mark and, with the air release tap open to atmosphere, the lid is secured.
2. The air release tap is closed, the levelling bulb is lifted from position 1, and placed in the hole provided in the stand, position 2. This creates a partial vacuum and the water flows from the measuring tube until an

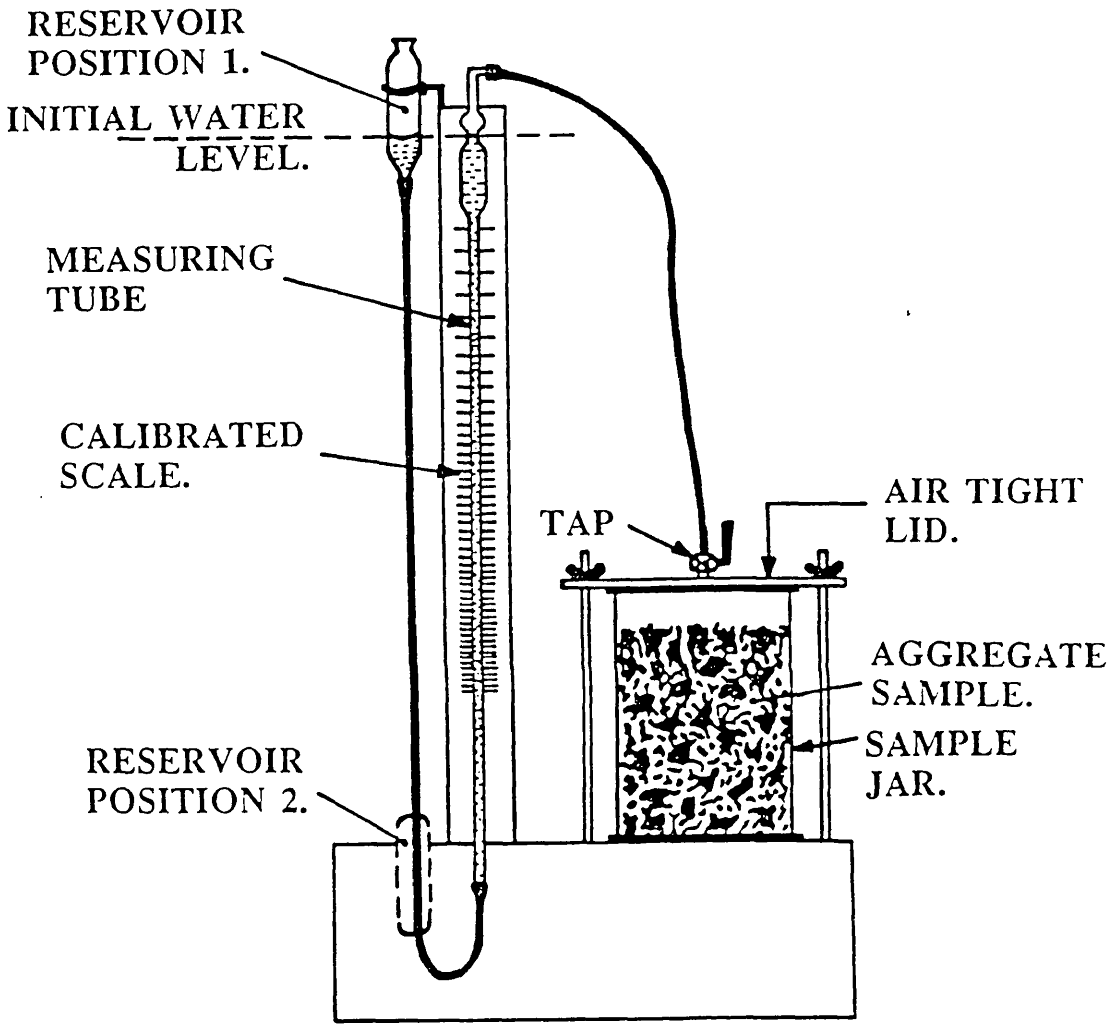


Figure 6.2: Void content apparatus.

equilibrium position is reached. This occurs when the pressure of the air in the system plus the pressure due to the height of the water column is equal to the atmospheric pressure. As the volume of air above the sample is constant, variation in the distance that the water column falls is directly related to the volume of air in the sample, and therefore the measuring tube is calibrated accordingly.

3. The measuring process is repeated several times to indicate a mean value and eliminate the chance of a faulty reading.

The apparatus also enables the specific gravity of the granular material to be determined, because the sample can be weighed and the volume occupied by the particles can be obtained by subtracting the void content from the total volume of the sample.

The method of filling depends on the required state of the material, i.e. compacted or uncompacted, and whether the sample contains more than one ingredient. For the uncompacted state, material is merely poured into the container. To obtain the compacted state, the sample is placed in the container in 25 mm layers, each layer being tamped 10 times by dropping a piston, shown in Figure 6.3, weighing 3.15 kg, through 25 mm. The voidage of the compacted state was considered to be more appropriate for the condition of the aggregate in high strength concrete, because of the use of mechanical vibrators that has always been considered essential for its proper compaction. Dewar[228,229], as mentioned on page 189, chose to measure the aggregate voidage in the loosely packed condition. According to him although the denser condition achieved by rodding the aggregate is inappropriate for the usual condition of aggregate in concrete, it may, however, be relevant for rolled mean or very low workability vibrated concretes.

(II) Test results.

The following combinations of aggregates were investigated:

1. 10 mm granite and sand.
2. 10 mm limestone and sand.

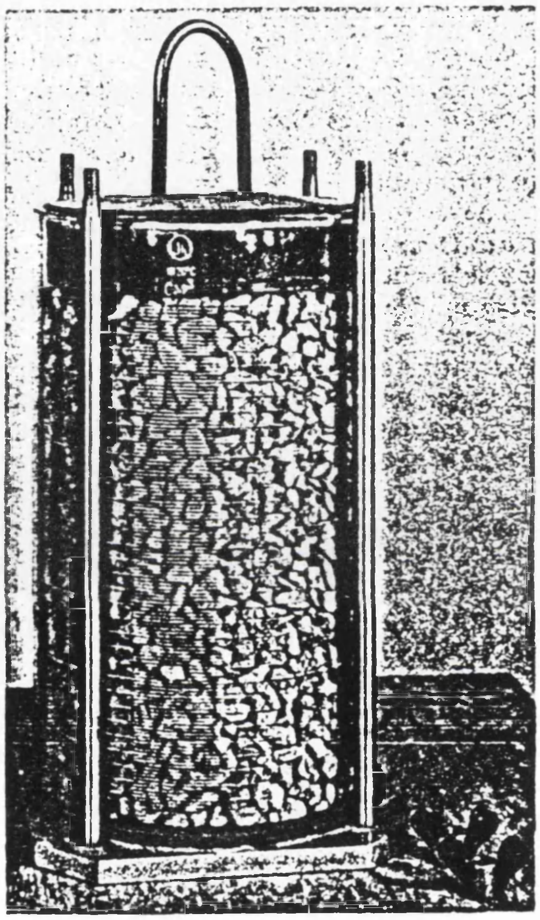


Figure 6.3: Piston used for compacting the aggregate.

3. 10 mm gravel and sand.
4. 20 and 10 mm granite
5. 20 and 10 mm limestone.
6. 20 and 10 mm gravel.
7. 20 and 10 mm granite (65 per cent of 20 mm and 35 per cent of 10 mm granite) and sand.
8. 20 and 10 mm limestone (50 per cent of 20 mm and 50 per cent of 10 mm limestone) and sand.
9. 20 and 10 mm gravel (68.5 per cent of 20 mm and 31.5 per cent of 10 mm gravel) and sand.
10. 10 mm granite and the coarse sand (fineness modulus = 2.73).

The detailed results of the above tests can be found in Appendix 2, and the void content curves are shown in Figures 6.4 to 6.7. The combination of 20 and 10 mm aggregates that were used to determine the minimum void content with sand were the proportions that gave the minimum void content determined in (4) to (6). The results are shown in Figure 6.5. Table 6.3 shows the combinations that gave the minimum void content, and their grading curves are shown in Figures 6.8 to 6.14. The use of a fine sand with FM of 2.1 is apparent from Figures 6.8 to 6.13. The grading curves of the Road Research Note No. 4[218], and similar curves for aggregate with a smaller maximum size, i.e. 3/8 in. (9.52 mm), and prepared by McIntosh and Erntroy[241], are also shown on these figures for the purpose of comparison. The grading curves for 10 mm aggregate are lying partly in zone B, and partly in zone C for particles smaller than 1.18 mm. The combinations with 20 mm granite and limestone aggregate even have higher proportions of the finer particles than that permitted by the Road Note No. 4 gradings[218]. The grading of 10 mm granite with the coarse sand (FM = 2.73) lies mainly in zone B, but has a lower proportion of particles smaller than 300 μm.

The combinations, shown in Table 6.3, that gave the minimum void content were then used as the basis for the next series of mixes.

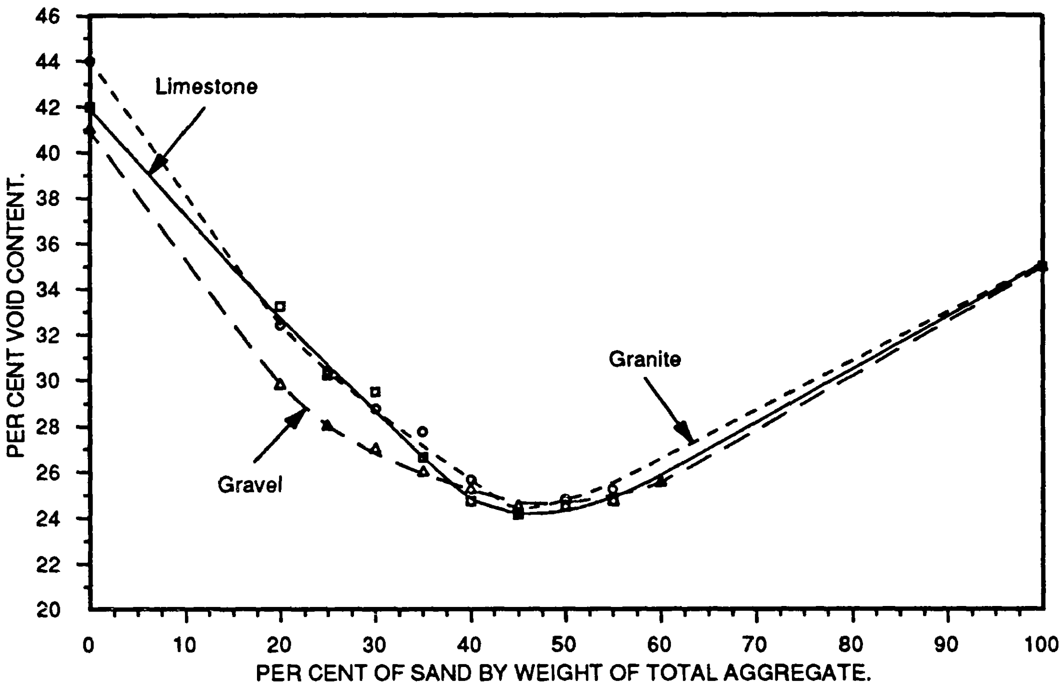


Figure 6.4: Void content curves for mixtures of 10 mm granite, limestone, and gravel with sand.

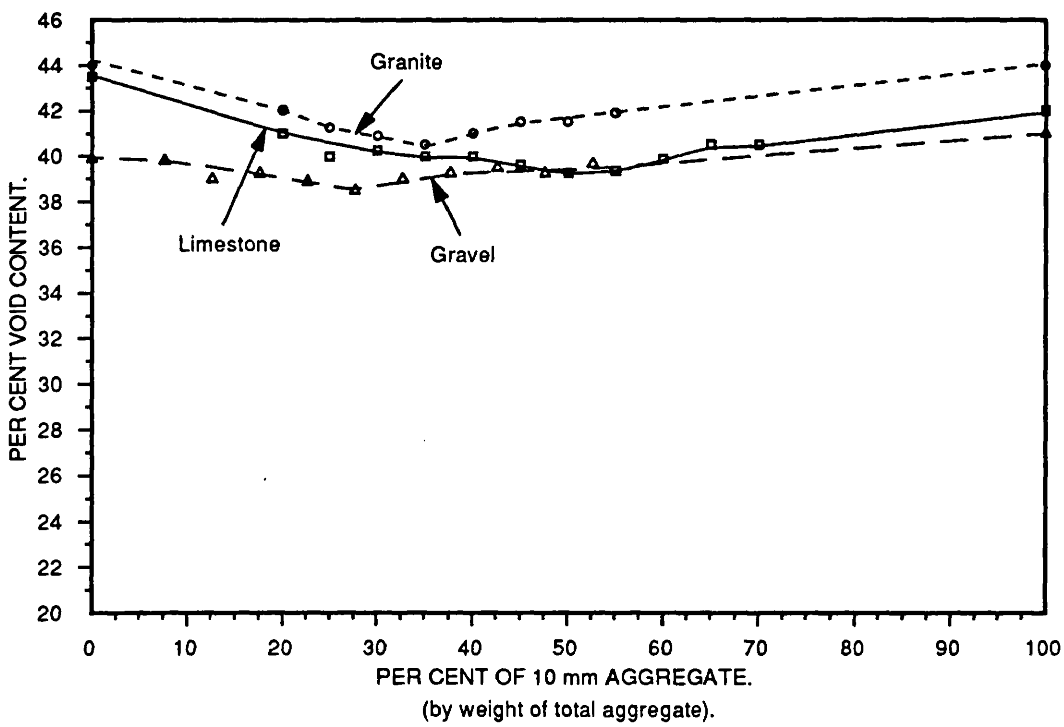


Figure 6.5: Void content curves for mixtures of 20 and 10 mm granite, limestone, and gravel aggregate.

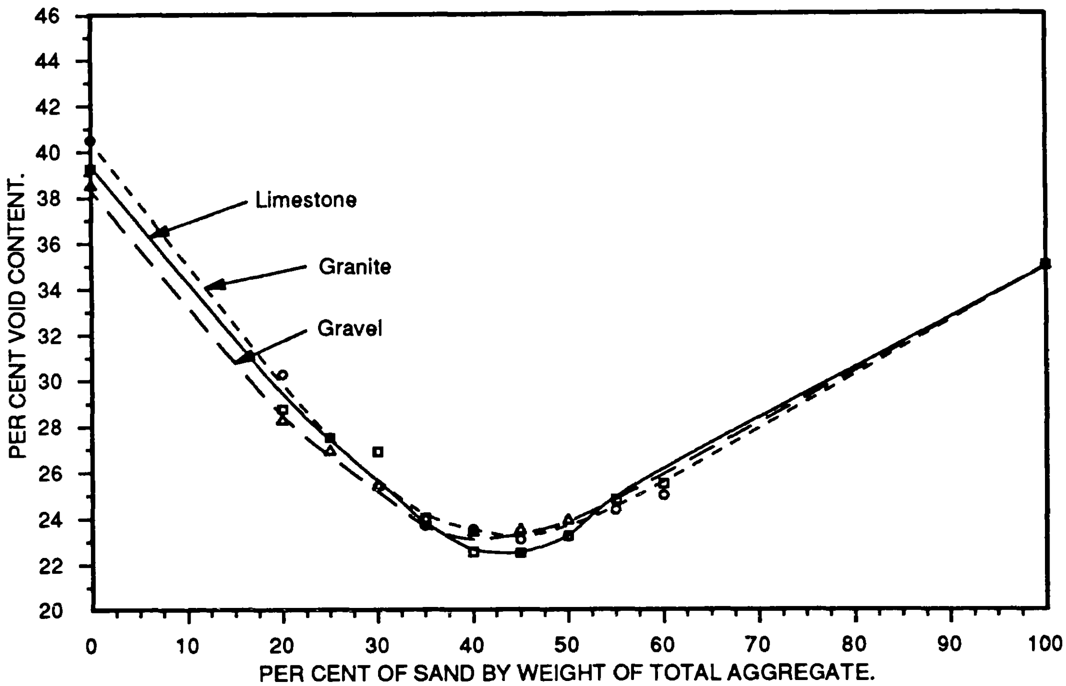


Figure 6.6: Void content curves for mixtures of 20-5 mm granite, limestone, and gravel with sand.

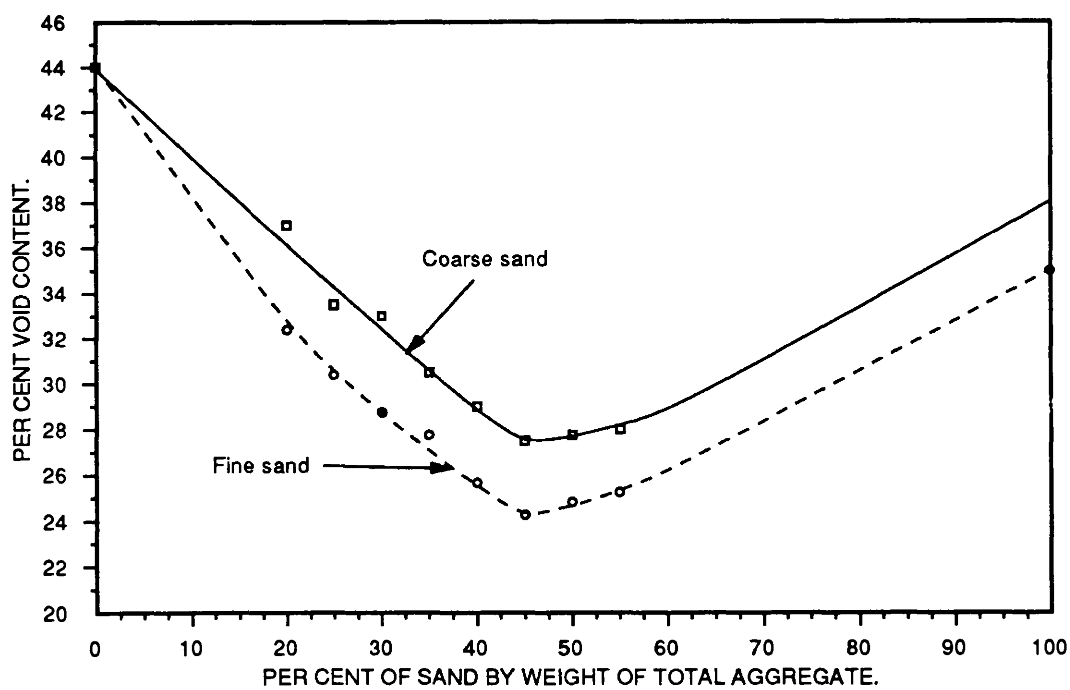


Figure 6.7: Void content curves for mixtures of 10 mm granite with sands of fineness moduli of 2.1 and 2.73.

	Aggregate Types	Proportions for minimum void content (% by weight)	Minimum void content (%)
1	10 mm granite/sand(1)	55/45	24.25
2	10 mm limestone/sand(1)	55/45	24.15
3	10 mm gravel/sand(1)	52.5/47.5	24.5
4	20 mm/10 mm granite	65/35	40.5
5	20 mm/10 mm limestone	50/50	39.25
6	20 mm/10 mm gravel	72.4/27.6	38.5
7	20-5 mm granite(2)/sand(1)	55/45	23.08
8	20-5 mm limestone(3)/sand(1)	55/45	22.5
9	20-5 mm gravel(4)/sand(1)	60/40	23.4
10	10 mm granite/coarse sand(5)	55/45	27.5

(1) Fine sand with FM = 2.1.
(2) Proportions of 20 and 10 mm granite for minimum void content as determined in 4.
(3) Proportions of 20 and 10 mm limestone for minimum void content as determined in 5.
(4) Proportions of 20 and 10 mm gravel for minimum void content as determined in 6.
(5) Coarse sand with FM = 2.73.

Table 6.3: Proportions of different aggregates required for minimum void content.

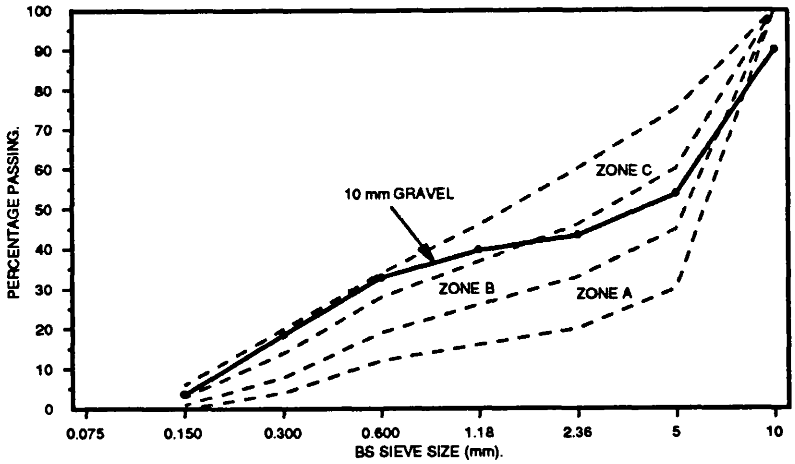


Figure 6.8: The grading curve of the mixture of 10 mm gravel and sand (FM = 2.1) that gave the minimum void content.

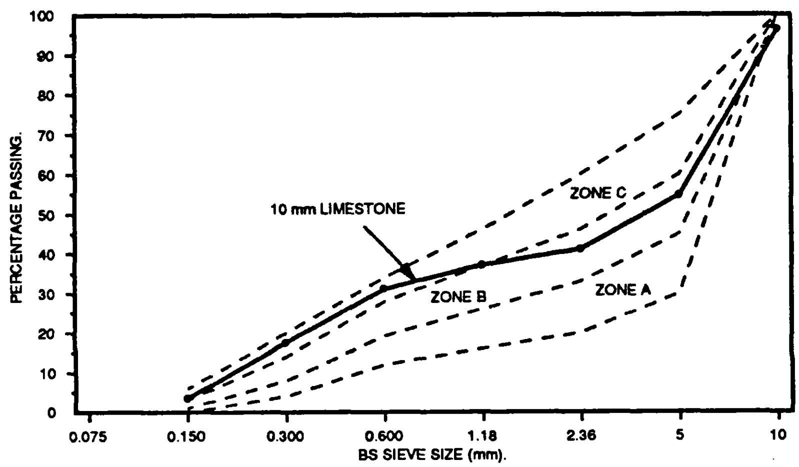


Figure 6.9: The grading curve of the mixture of 10 mm limestone and sand (FM = 2.1) that gave the minimum void content.

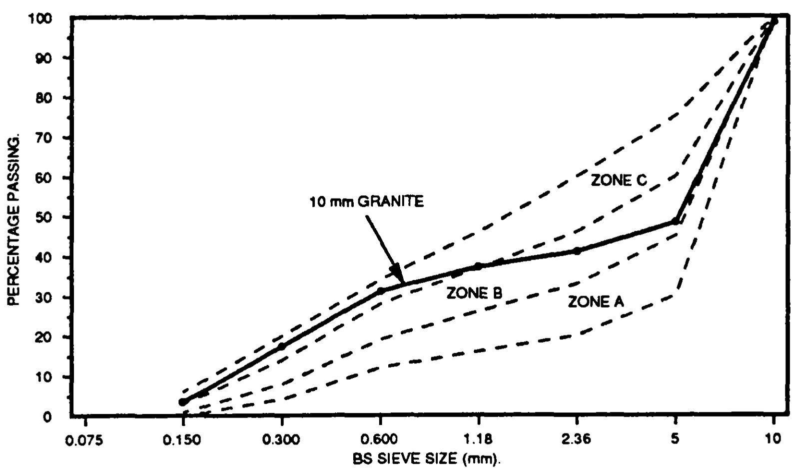


Figure 6.10: The grading curve of the mixture of 10 mm granite and sand (FM = 2.1) that gave the minimum void content.

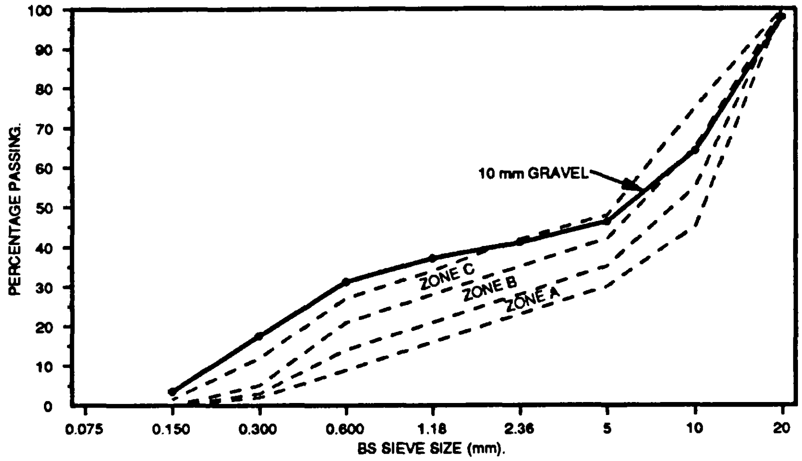


Figure 6.11: The grading curve of the mixture of 20 mm gravel and sand (FM = 2.1) that gave the minimum void content.

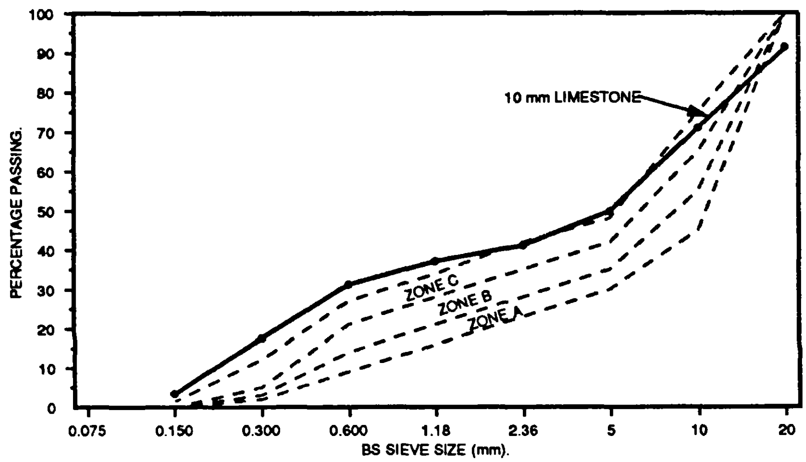


Figure 6.12: The grading curve of the mixture of 20 mm limestone and sand (FM = 2.1) that gave the minimum void content.

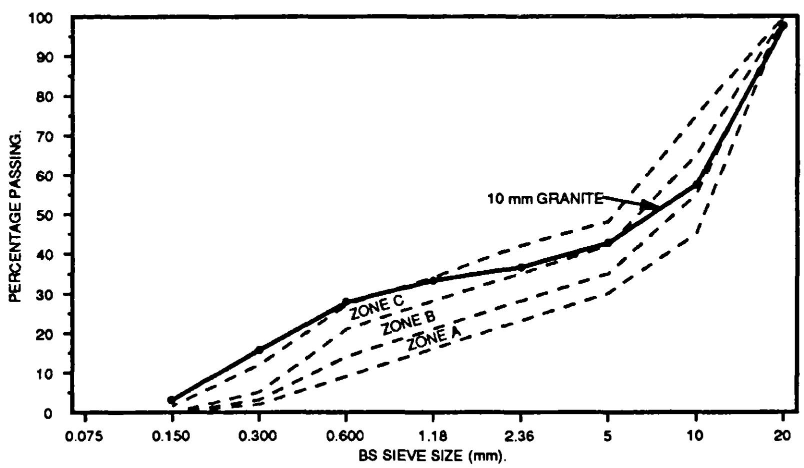


Figure 6.13: The grading curve of the mixture of 20 mm granite and sand (FM = 2.1) that gave the minimum void content.

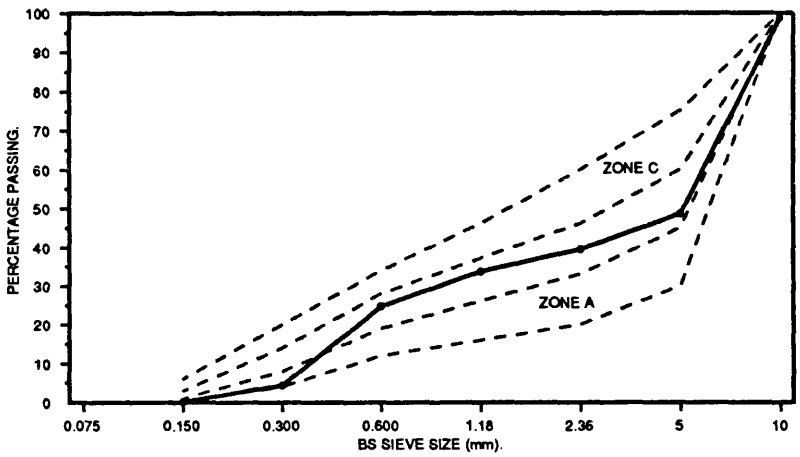


Figure 6.14: The grading curve of the mixture of 10 mm granite and coarse sand (FM = 2.73) that gave the minimum void content.

6.3.2 MIX DESIGNS AND WORKABILITY.

Tables 6.4 and 6.5 show the material proportions of the mixes. These were calculated from the void content of the aggregates, and the estimated density of the pastes. The mixes shown in Table 6.4 were used to investigate the effect of the superplasticiser type and dosage on the workability of concrete as measured by the slump test. The effects of the water-cement ratio and "overfill" by paste, expressed as a percentage of the total concrete volume, were investigated using the mixes shown in Table 6.5. The mixing procedure of the concrete was as described in Chapter 5.

(I) The effect of the superplasticiser type and dosage.

It was considered that a low water-cement ratio of 0.26 would be required to achieve compressive strengths of about 100 N/mm^2 at 28-days. Different dosages of two superplasticisers were therefore used in order to obtain a workable concrete at this low water-cement ratio. Figure 6.15 shows that the quantity of Conplast M1 had to be increased by one and a half times that of Conplast 430 in order to achieve a slump of 35 mm. It was therefore concluded that the sulphonated naphthalene formaldehyde condensate based superplasticiser (Conplast 430) was more efficient than the melamine based (Conplast M1). This is in agreement with the results of Penttala[132] and Murata et al[20], (see page 89). The Conplast 430 was therefore used for most of the subsequent research programme.

(II) The effect of water-cement ratio and overfill.

Figure 6.16 shows that an excess amount of paste above that required to fill the voids in the aggregate increases the slump. Low percentages of overfill, in combination with the aggregate proportion that gave minimum void content, produced a very cohesive but low slump concrete. This effect was greater at the 0.26 water-cement ratio. It is therefore apparent that there is a critical percentage of overfill that is required to "fluidify" the

Mix No.	1.	2.	3.	4.	5.
Superplasticiser dosage[1] (%) Conplast type:	2.5 430	5.0 430	7.5 430	5 M1	7.5 M1
Cement content (kg/m^3)	482	482	482	482	482
Coarse aggregate[2] (kg/m^3)	1015	1015	1015	1015	1015
Fine aggregate[3] (kg/m^3)	830	830	830	830	830
Free water (kg/m^3)	125	125	125	125	125
Total water (kg/m^3)	151	151	151	151	151
Slump (mm)	15	35	60	20	35

(1) Dosage is expressed as a percentage of superplasticiser solids by weight of cement.
(2) 10 mm gravel.
(3) Sand with fineness modulus of 2.1.

NOTE: The above mixes:
(i) have a free water-cement ratio of 0.26,
(ii) have been designed according to the "Maximum Density Theory", and
(iii) all have an overfill of 5 per cent.

Table 6.4: Mix proportions used to study the efficiency of two types of superplasticisers:
Conplast 430 is a naphthalene based superplasticiser, and,
Conplast M1 is a melamine based superplasticiser.

Mix No.	1.	2.	3.
Percentage overfill (%)	2	4	5
Cement content (kg/m^3)	417.6	440.4	452.4
Coarse aggregate (kg/m^3)	1044.1	1024.0	1014.3
Fine aggregate (kg/m^3)	852.9	836.5	828.6
Free water (kg/m^3)	125.5	131.7	135.2
Total water (kg/m^3)	152.0	157.7	161.0
Slump (mm)	30	70	185

(a) Mixes with water-cement ratio of 0.30.

Mix No.	1.	2.	3.	4.	5.
Percentage overfill (%)	2	4	5	6	8
Cement content (kg/m^3)	430.4	453.8	466.7	474.5	505.6
Coarse aggregate (kg/m^3)	1044.1	1024.0	1014.3	1004.7	986.1
Fine aggregate (kg/m^3)	852.9	836.5	828.6	820.8	805.6
Free water (kg/m^3)	120.6	126.9	130.5	133.0	141.7
Total water (kg/m^3)	147.1	152.9	156.2	158.5	166.7
Slump (mm)	15	45	60	160	195

(b) Mixes with water-cement ratio of 0.28.

Mix No.	1.	2.	3.
Percentage overfill (%)	5	7	9
Cement content (kg/m^3)	481.9	504.7	527.5
Coarse aggregate (kg/m^3)	1014.3	995.3	977.1
Fine aggregate (kg/m^3)	828.6	813.1	798.2
Free water (kg/m^3)	125.3	130.8	137.2
Total water (kg/m^3)	151.0	156.1	161.9
Slump (mm)	35	45	155

(c) Mixes with water-cement ratio of 0.26.

Table 6.5: Mixes tested to determine the variation of slump for increasing percentage of overfill of concrete by paste.

NOTE: The above mixes have been designed according to the "Maximum Density Theory", and all have a superplasticiser (Conplast 430) dosage of 2.0 per cent. The coarse aggregate used was 10 mm granite and the fine aggregate was sand with fineness modulus of 2.1.

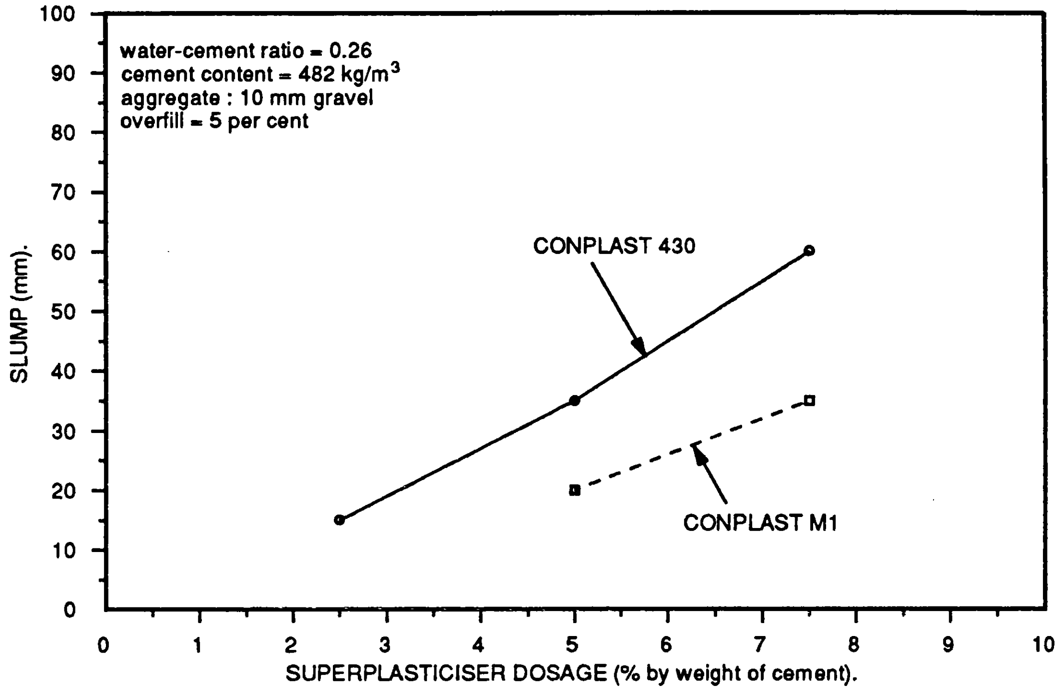


Figure 6.15: The effect of the superplasticiser type and dosage on the workability of concrete.

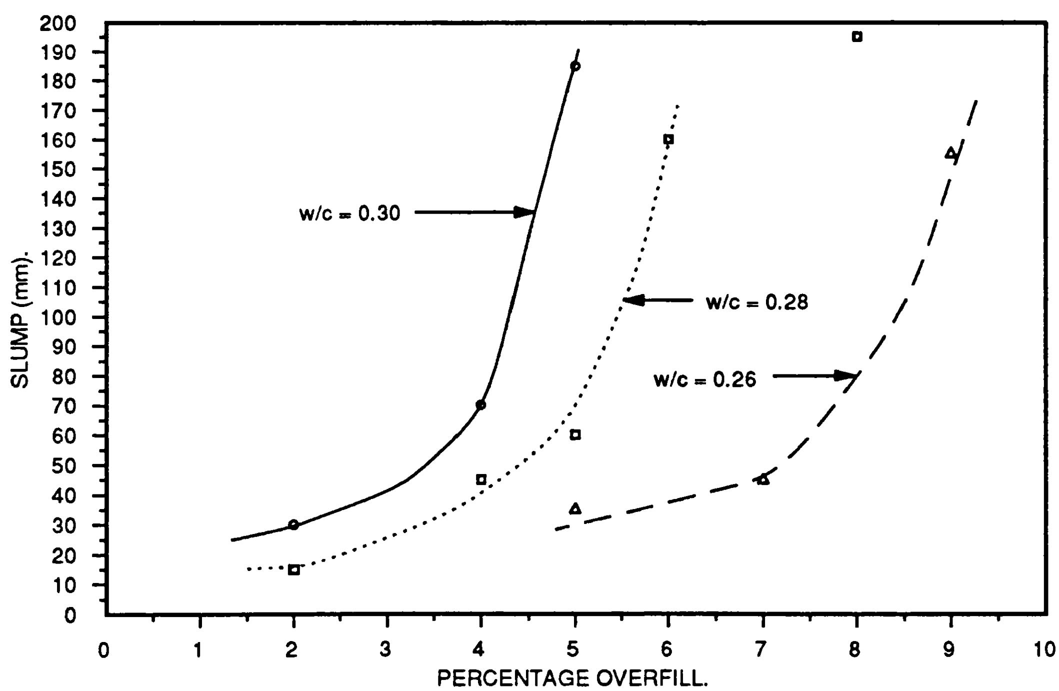


Figure 6.16: The effect of water-cement ratio and overfill on the workability of concrete.

concrete. This critical excess amount of paste is thought to be required in order to cover the surface of all the particles and therefore act as a lubricant. The excess volume of paste does not only depend on the surface area of the particles, (see page 161), but it also depends on its fluidity; the more fluid the paste, the less of it is required.

When the "Maximum Density Theory" was adopted as an approach to the mix design of high strength concrete, it was known that a percentage overfill would be required to produce a workable concrete. However, despite the fluidity of a paste with a very high superplasticiser dosage, i.e. 2 per cent solids by weight of cement, a high percentage overfill of 9 per cent was required to "fluidify" the concrete with a water-cement ratio of 0.26. This appears to arise from the fact that the "Maximum Density Theory" does not consider the effect that the aggregate surface area has on the requirement of excess paste for lubrication. A modification of this theory was therefore required, in order to take account of the combined effect of void content and surface area.

At this stage of the research programme, the experiments described in Section 4.2.4, Part (i): "The effect of type and size of coarse aggregate on the compressive strength of concrete", had already been completed. The results from these, which are more conveniently described in Chapter 9, led to the decision to use 10 mm granite aggregate for most of the research programme. The use of gravel was therefore terminated, and 10 mm granite was used for the next series of experiments.

6.4 THE "MODIFIED MAXIMUM DENSITY THEORY".

In order to investigate the effect of surface area on the amount of excess paste required to "fluidify" the concrete with a water-cement ratio of 0.38, mixes with lower sand proportions than required for minimum void content were tested for slump. Detailed data of the mixes are shown in Table 6.6. The total aggregate contents that were used for these tests have been re-calculated using the void content graph in Figure 6.4. This means that by reducing the sand proportion the granite content was increased, thus partly compensating for the increase in void content resulting from the reduction of the sand

Mix No.:	1	2	3	4	5	6	7	8
Percentage overfill (%)	3	4	5	4	5	6	5	6
Cement content (kg/m^3)	428.3	438.0	447.5	423.6	433.2	442.6	418.8	428.4
Coarse aggregate[(1)] (kg/m^3)	1179.6	1168.3	1157.1	1115.4	1104.8	1094.3	1095.2	1084.9
Fine aggregate[(2)] (kg/m^3)	635.9	629.8	623.8	668.3	661.9	655.7	728.6	721.7
Proportion (%)	35	35	35	37.5	37.5	37.5	40	40
Free water (kg/m^3)	162.8	166.4	170.1	161.0	164.6	168.2	159.1	162.8
Total water (kg/m^3)	186.6	190.0	193.4	184.6	188.0	191.4	183.6	187.0
Slump (mm)	90	105	175	80	110	175	60	105

Mix No.:	9	10	11	12	13	14	15
Percentage overfill (%)	7	7.5	7	8	9	8	10
Cement content (kg/m^3)	437.8	442.4	428.4	437.7	446.9	428.4	446.7
Coarse aggregate[(1)] (kg/m^3)	1074.8	1069.8	1037.4	1027.8	1018.3	986.1	968.2
Fine aggregate[(2)] (kg/m^3)	715.0	711.6	766.4	759.3	752.3	805.6	790.9
Proportion (%)	40	40	42.5	42.5	42.5	45	45
Free water (kg/m^3)	166.4	168.1	162.8	166.4	169.8	162.8	169.7
Total water (kg/m^3)	190.4	192.0	187.3	190.6	193.9	187.3	193.8
Slump (mm)	125	160	70	105	150	55	100

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1.

Table 6.6: Optimisation of mix proportions for mixes with a water-cement ratio of 0.38.

NOTE: The superplasticiser dosage (percentage of Conplast 430 solids by weight of cement) was 0.5 per cent for all mixes

content. Although the surface area of the aggregate has not been measured, an idea of how the total surface area varies with different combinations of coarse and fine aggregate can be obtained from the fineness modulus (FM); a lower sand proportion implies a higher average particle size for the combined aggregate, a greater FM, and therefore a lower total surface area.

Figure 6.17 shows that a smaller amount of excess paste is required to fluidify the concrete for lower sand proportions. The dotted line, in Figure 6.17, shows the slumps for equal cement contents at a water-cement ratio of 0.38. It can be seen that the highest slump, for a cement content of 440 kg/m^3, is obtained with a fine aggregate proportion of 37.5 per cent of the total aggregate content. This proportion, therefore, results in the minimum required cement content for a particular slump, e.g., a slump of 150 mm, for which the required cement contents with different fine aggregate proportions are shown in Figure 6.18. The slump was chosen to be 150 mm as at this value the gradients of the slump versus percentage overfill lines were at their highest, i.e. with only a 1 per cent increase in the overfill, the slump value was increased from about 100 to over 150 mm. Sand proportions higher than the optimum 37.5 per cent result in a higher requirement of excess paste because of the increased surface area. The increased void content of mixes with lower sand proportions than the optimum requires a larger volume of paste to fill the voids between the aggregates. Thus, despite lower values of percentage overfill required for 150 mm slump, the cement content per cubic metre of these mixes is higher than for the mix with optimum sand proportion. The effect of:

(i) different water-cement ratios,

(ii) binder type, i.e. the use of CSF to partially replace cement,

(iii) maximum aggregate size,

and, (iv) fineness modulus of sand,

on the optimum sand proportion were subsequently investigated and are now described.

(I) The effect of the water-cement ratio.

The same procedure as above was followed for the optimisation of the mix proportions

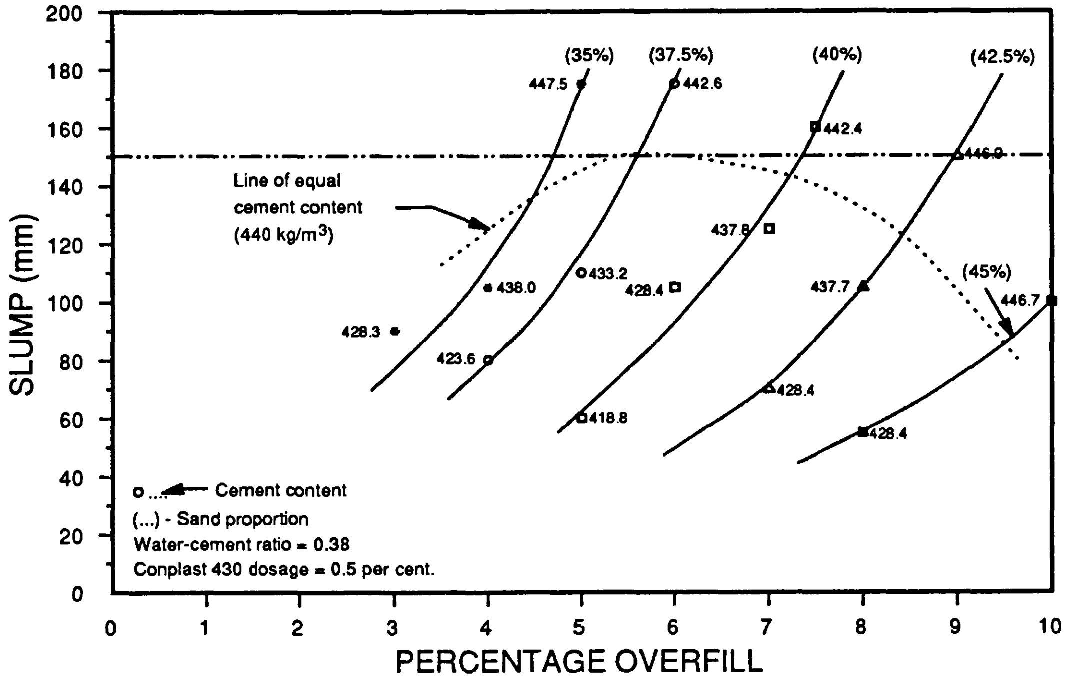


Figure 6.17: Slump versus overfill for various sand proportions at a water-cement ratio of 0.38.

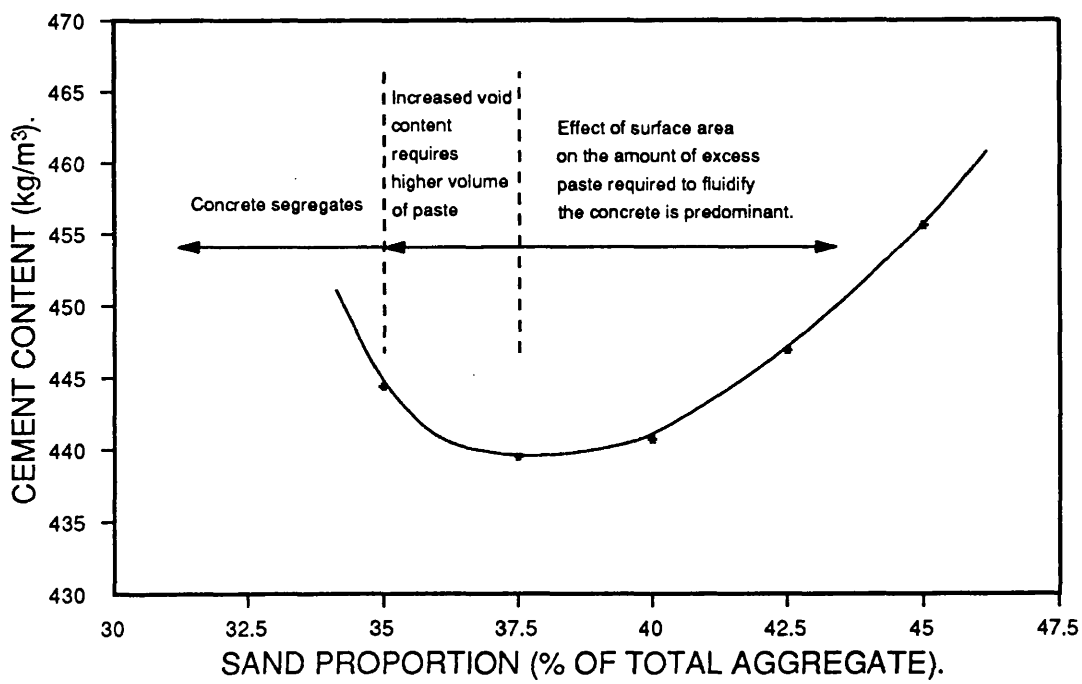


Figure 6.18: Cement contents required to produce a slump of 150 mm with various sand proportions.

for water-cement ratios of 0.32 and 0.26. Details of the mixes used are shown in Tables 6.7 and 6.8. As can be seen from Figures 6.19 to 6.22, the optimum sand proportion is not affected by the water-cement ratio. This is in agreement with Hughes' hypothesis(223), i.e. that the optimum aggregate combination occurs when the interstices of the coarse aggregate are just sufficiently large to readily accept the finer particles, especially the coarser fraction of the fine aggregate; changes in the water-cement ratio alone have no effect provided that the volume fractions of the coarse and fine aggregate remain constant, (see Figure 3.80). However, since his mix design was for non-superplasticised concrete he fixed the optimum coarse aggregate content and varied both the water, cement and fine aggregate contents to achieve the required workability. For high strength concrete, this can be maintained by increasing the superplasticiser dosage, even while decreasing the water-cement ratio.

Figure 6.23 shows that the grading of the "optimum proportions", determined by the "Modified Maximum Density Theory", is coarser than that required for minimum void content. It lies mainly in zone B and is similar to curve No. 2 for particles larger than 1.18 mm.

(II) The effect of the binder type.

Details of the mixes tested are shown in Table 6.9. A 10 per cent level of cement replacement by CSF was used, as it was considered that 5 per cent would not have been sufficient in showing clearly any effects on the optimum proportion of sand. The experiments described in Section 4.2.2, Part (i) had already been completed, and these led to the decision to use a water-binder ratio of 0.26. At this water-binder ratio, the superplasticiser dosage of 1.5 per cent required for 150 mm slump appears to sufficiently disperse the CSF particles so that the "filler effect", (see page 133), was apparent. The superplasticiser dosage was therefore slightly reduced to 1.3 per cent and the same procedure as for OPC mixes was followed for the optimisation of the mix proportions.

As can be seen from Figures 6.24 and 6.25, the optimum sand proportion is increased to

Mix No.:	1	2	3	4	5	6	7
Percentage overfill (%)	3	3.5	4	3.5	4	5	4
Cement content (kg/m^3)	468.2	473.5	478.8	457.6	462.9	473.4	447.1
Coarse aggregate(1) (kg/m^3)	1179.6	1173.9	1168.3	1120.8	1115.4	1104.8	1105.8
Fine aggregate(2) (kg/m^3) Proportion (%)	635.9 35	632.9 35	629.8 35	671.5 37.5	668.3 37.5	661.9 37.5	735.6 40
Free water (kg/m^3)	149.8	151.5	153.2	146.5	148.1	151.5	143.1
Total water (kg/m^3)	173.6	175.2	176.7	170.2	171.7	175.0	167.8
Slump (mm)	90	130	170	60	110	170	45

Mix No.:	8	9	10	11	12	13	14
Percentage overfill (%)	5	6	6	7	8	7	9
Cement content (kg/m^3)	457.8	468.2	457.9	468.3	478.4	458.0	478.4
Coarse aggregate(1) (kg/m^3)	1095.2	1084.9	1047.2	1037.4	1027.8	995.3	977.1
Fine aggregate(2) (kg/m^3) Proportion (%)	728.6 40	721.7 40	773.6 42.5	766.4 42.5	759.3 42.5	813.1 45	798.2 45
Free water (kg/m^3)	146.5	149.8	146.5	149.8	153.1	146.5	154.9
Total water (kg/m^3)	171.0	174.1	171.2	174.3	177.3	171.3	177.3
Slump (mm)	80	130	60	100	130	25	80

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1.

Table 6.7: Optimisation of mix proportions for mixes with a water-cement ratio of 0.32.

NOTE: The superplasticiser dosage (percentage of Conplast 430 solids by weight of cement) was 0.8 per cent for all mixes

Mix No.:	1	2	3	4	5	6	7	8	9
Percentage overfill (%)	2	3	2	3	4	3	4	5	4
Cement content (kg/m^3)	513.1	524.9	504.6	516.5	528.2	498.9	510.7	522.3	493.3
Coarse aggregate(1) (kg/m^3)	1235.3	1223.3	1191.2	1179.6	1168.3	1126.2	1115.4	1104.8	1105.8
Fine aggregate(2) (kg/m^3)	593.1	587.4	642.2	635.9	629.8	674.8	668.3	661.9	735.6
Proportion (%)	32.5	32.5	35	35	35	37.5	37.5	37.5	40
Free water (kg/m^3)	133.4	136.5	131.2	134.3	137.3	129.7	132.8	135.8	128.3
Total water (kg/m^3)	157.1	159.9	155.2	158.1	160.9	153.6	156.4	159.2	153.0
Slump (mm)	60	140	40	110	170	60	125	190	55

Mix No.:	10	11	12	13	14	15	16	17
Percentage overfill (%)	5	6	7	6	7	8	8	9
Cement content (kg/m^3)	505.0	516.6	527.9	505.2	516.6	527.9	516.6	527.8
Coarse aggregate(1) (kg/m^3)	1095.2	1084.9	1074.8	1047.2	1037.4	1027.8	986.1	977.1
Fine aggregate(2) (kg/m^3)	728.6	721.7	715.0	773.6	766.4	759.3	805.6	798.2
Proportion (%)	40	40	40	42.5	42.5	42.5	45	45
Free water (kg/m^3)	131.3	134.3	137.3	131.3	134.3	137.2	134.4	137.2
Total water (kg/m^3)	155.8	158.6	161.3	156.0	158.8	161.5	158.9	161.6
Slump (mm)	80	130	170	60	100	160	60	90

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1.

Table 6.8: Optimisation of mix proportions for mixes with a water-cement ratio of 0.26.

NOTE: The superplasticiser dosage (percentage of Conplast 430 solids by weight of cement) was 1.5 per cent for all mixes

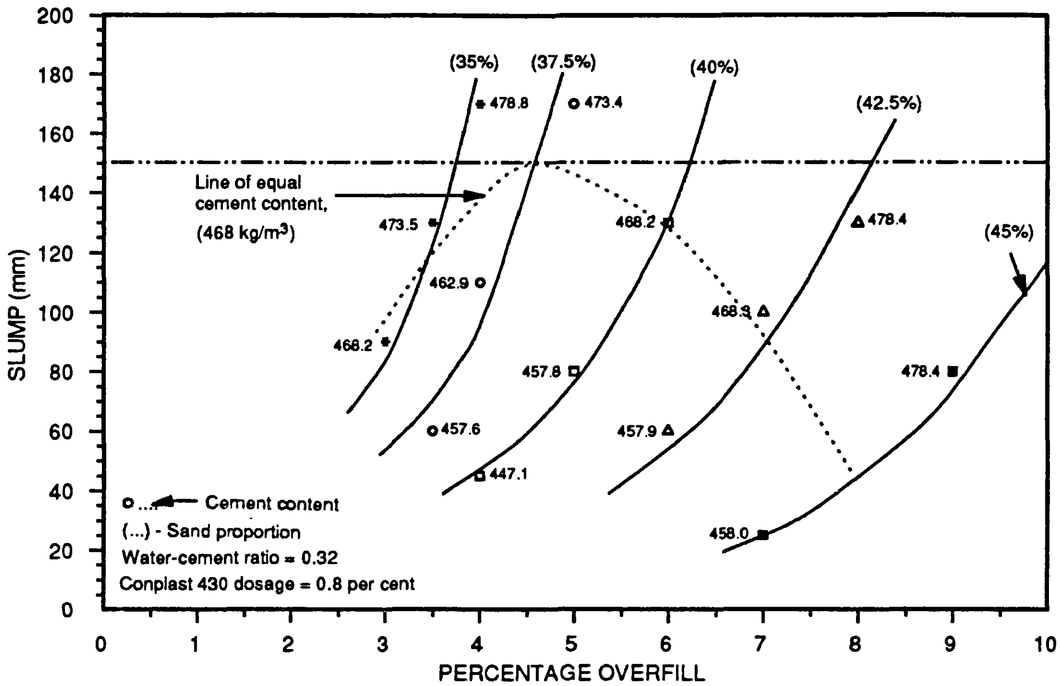


Figure 6.19: Slump versus overfill for various sand proportions at a water-cement ratio of 0.32.

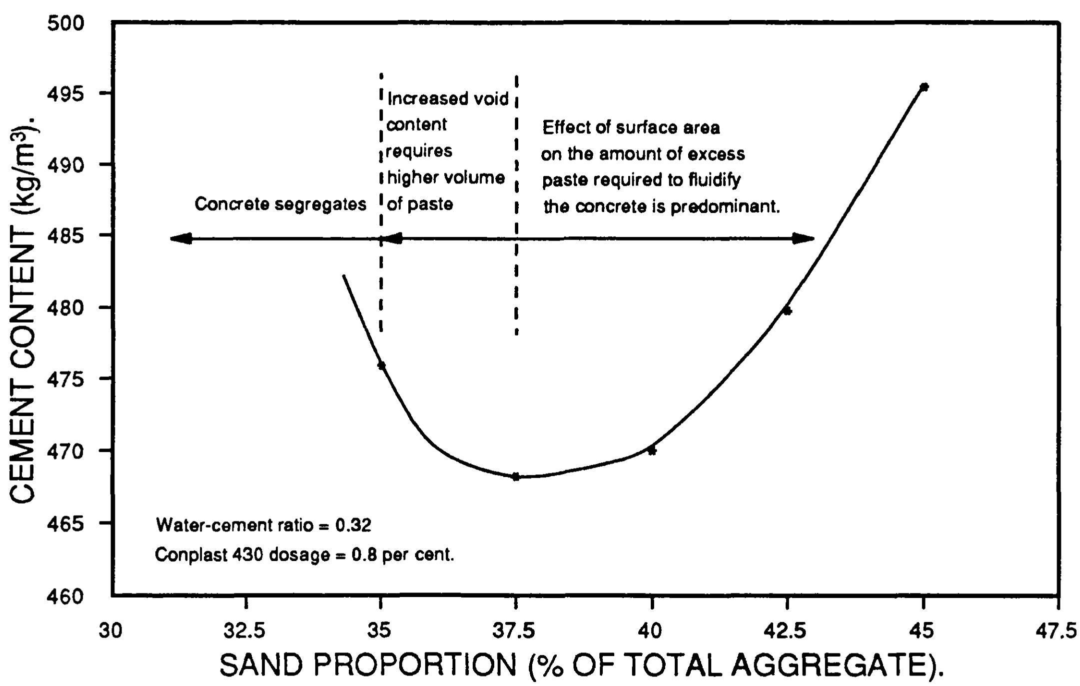


Figure 6.20: Cement contents required to produce a slump of 150 mm with various sand proportions.

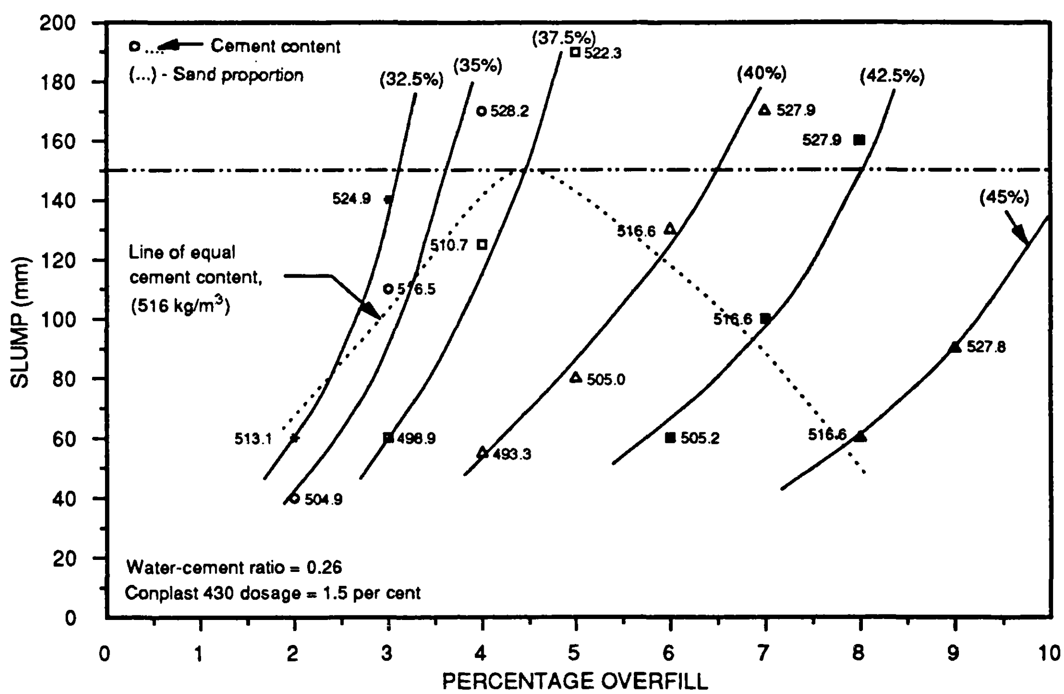


Figure 6.21: Slump versus overfill for various sand proportions at a water-cement ratio of 0.26.

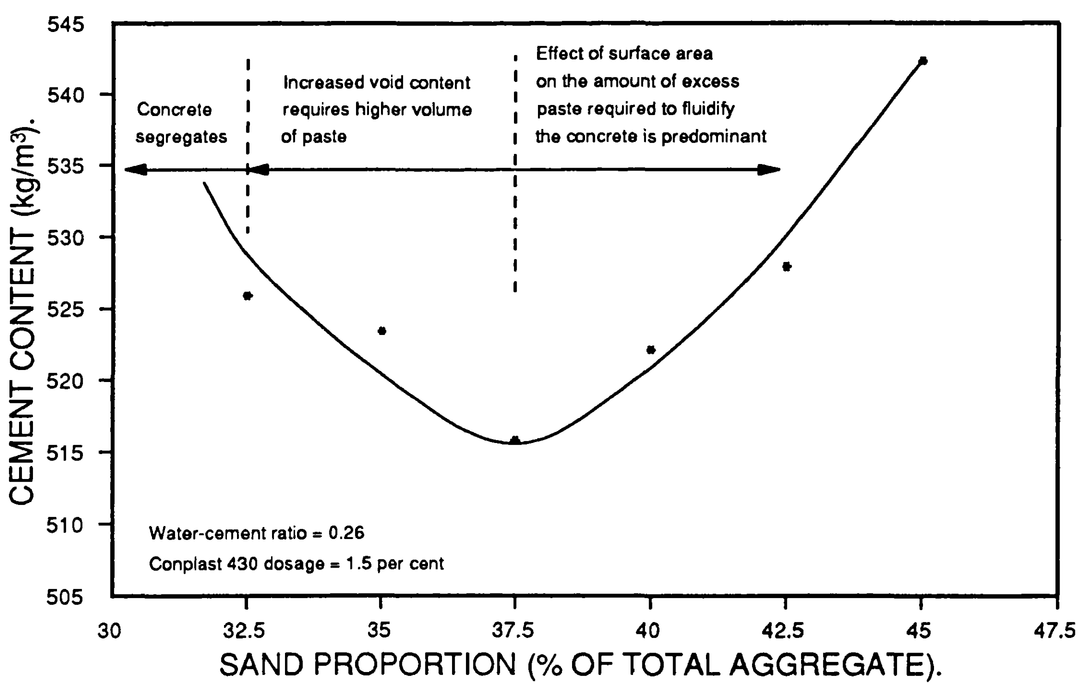


Figure 6.22: Cement contents required to produce a slump of 150 mm with various sand proportions.

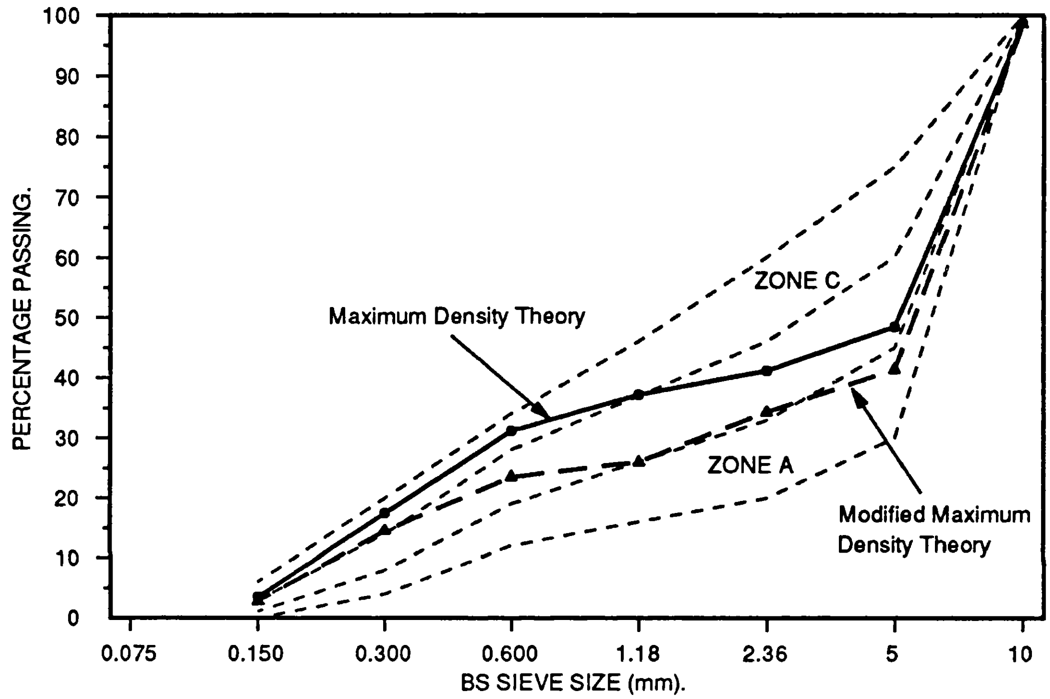


Figure 6.23: The grading curve of the "optimum proportions" of 10 mm granite and sand (FM = 2.1), determined by the "Modified Maximum Density Theory".

Mix No.:	1	2	3	4	5	6	7	8
Percentage overfill (%)	3	4	5	4	5	6	7	5
Cement content (kg/m^3)	464.9	475.4	485.6	459.7	470.1	480.4	490.5	454.6
CSF content (kg/m^3)	51.7	52.8	54.0	51.1	52.3	53.4	54.5	50.5
Total binder content (kg/m^3)	516.6	528.2	539.6	510.8	522.4	533.8	545.0	505.1
Coarse aggregate(1) (kg/m^3)	1179.6	1168.3	1157.1	1115.4	1104.8	1094.3	1084.1	1095.2
Fine aggregate(2) (kg/m^3) Proportion (%)	635.9 35	629.8 35	623.8 35	668.3 37.5	661.9 37.5	655.7 37.5	649.5 37.5	728.6 40
Free water (kg/m^3)	134.3	137.3	140.3	132.8	135.8	138.8	141.7	131.3
Total water (kg/m^3)	158.1	160.9	163.7	156.4	159.2	162.0	164.7	155.8
Slump (mm)	80	120	140	80	100	140	180	80

Mix No.:	9	10	11	12	13	14	15
Percentage overfill (%)	6	7	7	8	9	8	9
Cement content (kg/m^3)	464.9	475.1	465.0	475.1	485.0	465.0	475.0
CSF content (kg/m^3)	51.7	52.8	51.7	52.8	53.9	51.7	52.8
Total binder content (kg/m^3)	516.6	527.9	516.7	527.9	538.9	516.7	527.8
Coarse aggregate(1) (kg/m^3)	1084.9	1074.8	1037.4	1027.8	1018.3	986.1	977.1
Fine aggregate(2) (kg/m^3) Proportion (%)	721.7 40	715.0 40	766.4 42.5	759.3 42.5	752.3 42.5	805.6 45	798.1 45
Free water (kg/m^3)	134.3	137.3	134.3	137.2	140.1	134.4	137.2
Total water (kg/m^3)	158.6	161.3	158.8	161.5	164.1	158.9	161.6
Slump (mm)	105	140	100	130	170	80	110

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1.

Table 6.9: Optimisation of mix proportions for mixes with 10 per cent CSF and with a water-binder ratio of 0.26.

NOTE: The superplasticiser dosage (percentage of Conplast 430 solids by weight of cement) was 1.3 per cent for all mixes.

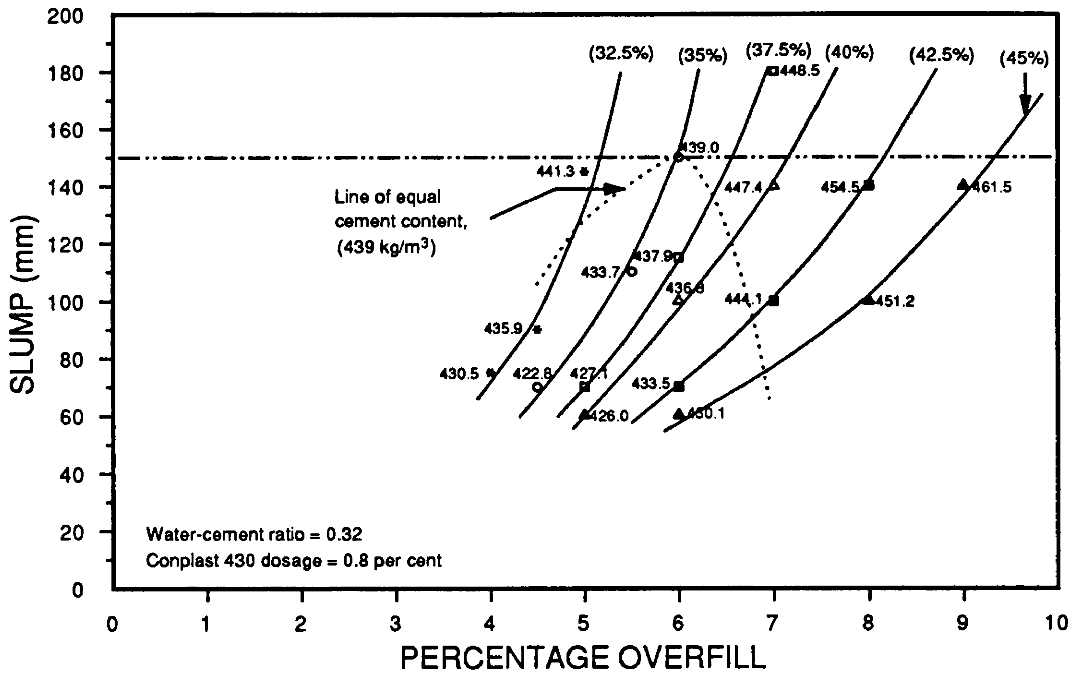


Figure 6.24: Slump versus overfill for mixes with 10 per cent CSF and various sand proportions. The water-binder ratio was 0.26.

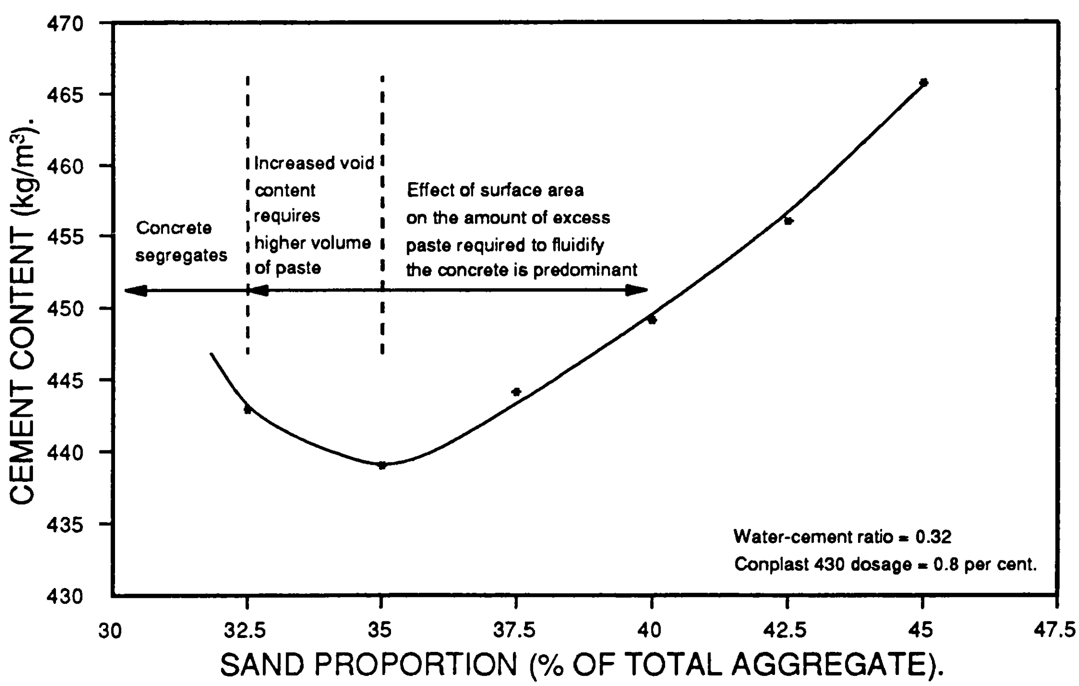


Figure 6.25: Binder contents required to produce a slump of 150 mm with various sand proportions.

40 per cent when CSF is used, as compared to 37.5 per cent for OPC mixes. The void content of the aggregate becomes predominant at a higher proportion of sand since the effect of the surface area of the combined aggregate on the excess amount of paste required for lubrication is reduced by increasing the fineness of the binder. The critical percentage overfill that is required to fluidify the concrete becomes less apparent, i.e. the increase in slump resulting from an increase of the overfill above the critical amount is not very large. This can be seen in Figure 6.24 as the curves for slump versus percentage overfill are not as steep as those for OPC mixes shown in Figure 6.21. This however may also have been partly influenced by the changes in the rheological properties of the paste resulting from increasing the fineness of the cementitious material.

(III) The effect of maximum aggregate size.

Details of the mixes tested are shown in Table 6.10. Since the water-cement ratio was shown previously not to affect the optimum mix proportions, the median of the three water-cement ratios previously used, i.e. 0.32, was used to optimise the mix proportions for 20 mm granite. As shown in Figures 6.26 and 6.27, the optimum sand proportion was found to be 35 per cent. The optimum sand proportion for 20 mm granite is therefore lower than for 10 mm granite despite the fact that both of these achieve a minimum void content with a sand proportion of 45 per cent. This difference can be explained using the void content curves shown in Figures 6.4 and 6.6. The void content for 20 mm granite is not greatly increased until the sand proportion is reduced below 35 per cent, while for 10 mm granite the void content increases significantly when the sand proportion is reduced from 40 to 35 per cent.

Figure 6.28 shows that the grading of the "optimum proportions" determined by the "Modified Maximum Density Theory" is coarser than that required for minimum void content. It is similar to curve No. 2 for particles larger than 5 mm but because of the use of a fine sand the grading lies both in zone B and C, and even has an excess of particles smaller than 300 μm.

Mix No.:	1	2	3	4	5	6	7	8	9
Percentage overfill (%)	4	4.5	5	4.5	5.5	6	5	6	7
Cement content (kg/m^3)	430.5	435.9	441.3	422.8	433.7	439.0	427.1	437.9	448.5
Coarse aggregate									
20 mm granite (kg/m^3)	788.5	784.7	781.0	800.0	792.4	788.7	763.8	756.6	749.5
10 mm granite (kg/m^3)	439.4	437.3	435.2	430.6	426.5	424.5	411.4	407.5	403.7
Fine aggregate[(1)] (kg/m^3)	604.8	601.9	599.0	663.2	656.9	653.8	704.8	698.1	691.6
Proportion (%)	32.5	32.5	32.5	35	35	35	37.5	37.5	37.5
Free water (kg/m^3)	137.8	139.5	141.2	135.3	138.8	140.5	136.7	140.1	143.6
Total water (kg/m^3)	159.2	163.0	162.5	157.8	161.0	162.6	159.3	162.5	165.8
Slump (mm)	75	90	145	70	110	150	70	115	180

Mix No.:	10	11	12	13	14	15	16	17	18
Percentage overfill (%)	5	6	7	6	7	8	6	8	9
Cement content (kg/m^3)	426.0	436.8	447.4	433.5	444.1	454.5	430.1	451.2	461.5
Coarse aggregate									
20 mm granite (kg/m^3)	731.4	724.5	717.8	694.3	687.9	681.5	664.2	651.9	645.9
10 mm granite (kg/m^3)	394.3	390.6	386.9	373.6	370.1	366.7	357.5	350.9	347.7
Fine aggregate[(1)] (kg/m^3)	750.5	743.4	736.4	789.6	782.2	775.0	835.8	820.4	812.8
Proportion (%)	40	40	40	42.5	42.5	42.5	45	45	45
Free water (kg/m^3)	136.3	139.8	143.2	138.7	142.2	145.5	137.6	144.4	147.7
Total water (kg/m^3)	159.2	162.5	165.7	161.8	165.0	168.1	161.1	167.4	170.6
Slump (mm)	60	100	140	70	100	140	60	100	140

(1) Sand with fineness modulus of 2.1.

Table 6.10: Optimisation of mix proportions for mixes with a maximum aggregate size of 20 mm and with water-cement ratio of 0.32.

NOTE: The superplasticiser dosage (percentage of Conplast 430 solids by weight of cement) was 0.8 per cent for all mixes.

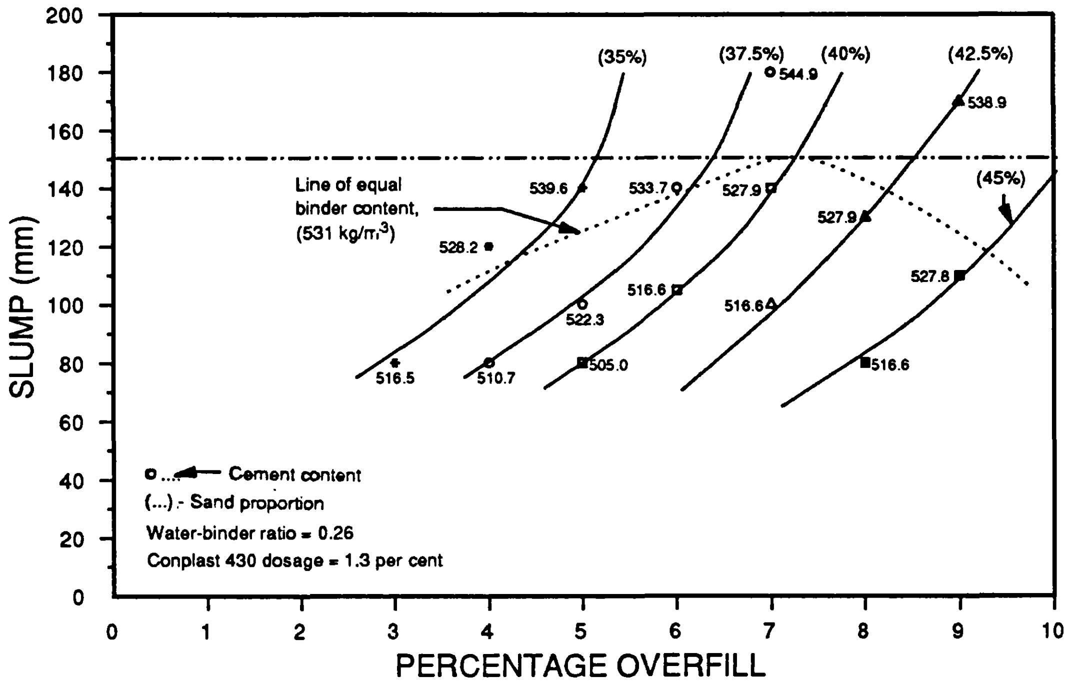


Figure 6.26: Slump versus overfill for mixes with 20 mm granite and various sand proportions. The water-cement ratio was 0.32.

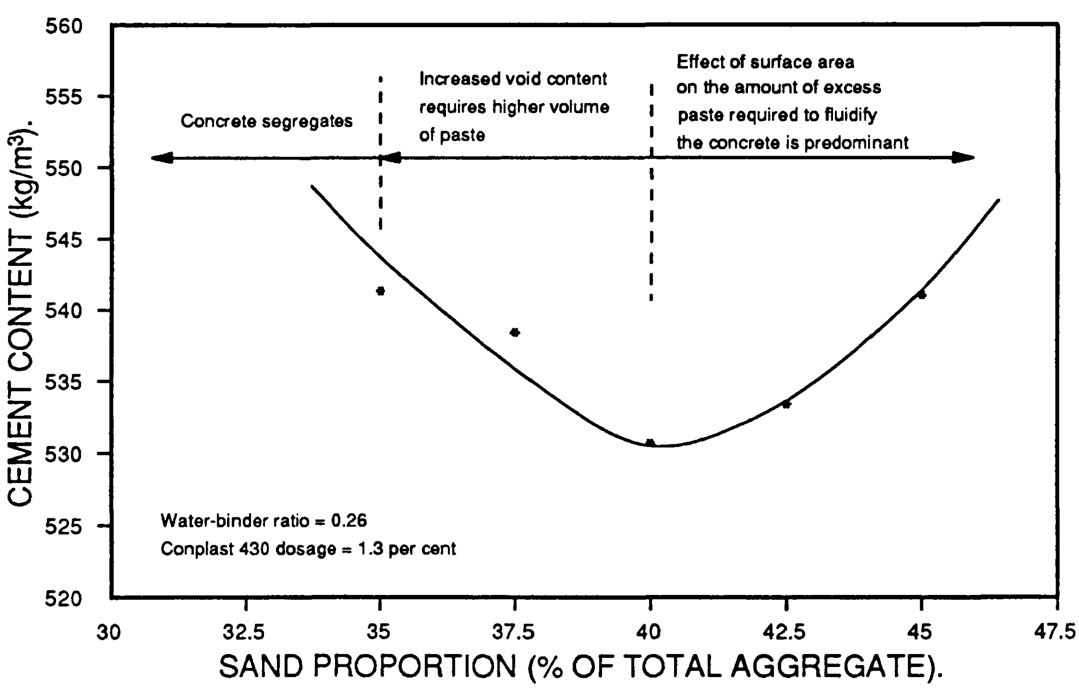


Figure 6.27: Cement contents required to produce a slump of 150 mm with various sand proportions.

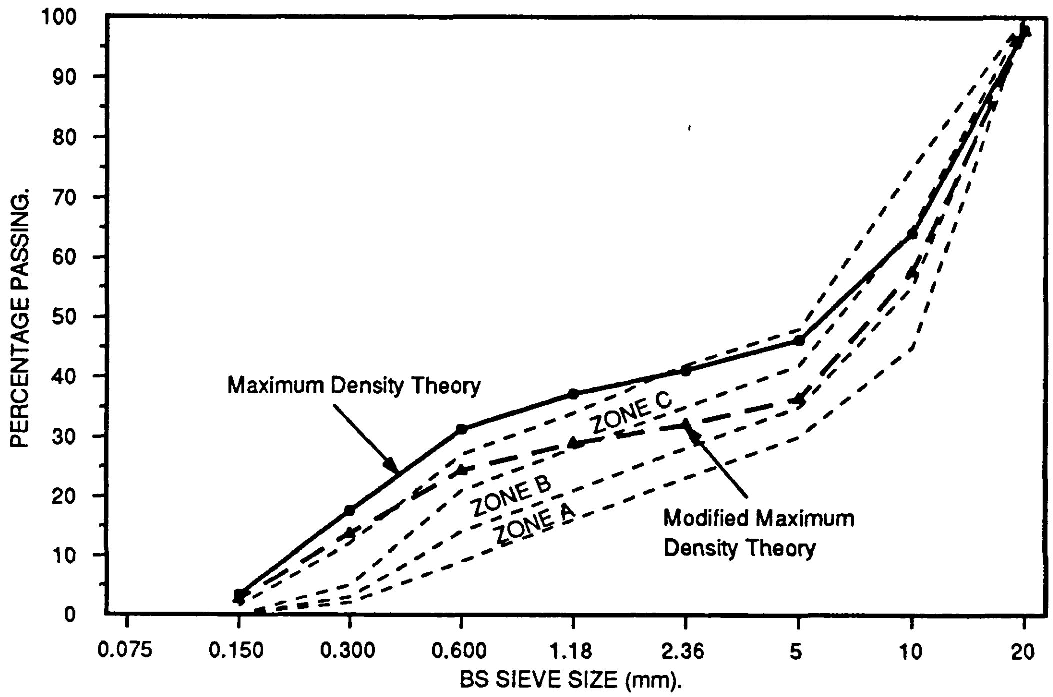


Figure 6.28: The grading curve of the "optimum proportions" of 20 mm granite and sand (FM = 2.1), determined by the "Modified Maximum Density Theory".

(IV) The effect of FM of sand.

The effect of changing the type of sand was investigated using the mixes shown in Table 6.11. The sand that has been used for the previous mixes had a fineness modulus of 2.1, but later during my research programme sand with fineness modulus of 2.73 became available, and it was used to optimise the mix proportions of concrete with 10 mm granite at a water-cement ratio of 0.32. As shown in Figures 6.29 and 6.30, the optimum sand proportion is very close to the sand proportion that produces the minimum void content, i.e. the optimum sand proportion is 42.5 per cent as compared to 45 per cent that produces the minimum void content. The void content of the aggregate becomes predominant at a higher proportion of sand since the effect of the surface area of the combined aggregate on the excess amount of paste required for lubrication is reduced by increasing the fineness modulus of the sand.

The aim of these experiments was to determine the optimum proportions to be used for the mixes described in Section 4.2.4, Part (iii): "The effect of the FM of sand on the compressive strength". An overfill similar to that of mixes with sand of FM = 2.1, i.e. 5 per cent, was considered to be more important than a similar superplasticiser dosage. Because of the different superplasticiser dosages for the mixes with coarse and fine sands, their cement contents cannot be compared. For example, the mix with fine sand (Mix No. 6 in Table 6.7) has a lower cement content but a higher superplasticiser dosage than the mix with coarse sand (Mix No. 11 in Table 6.11). Further experiments are therefore required to determine which FM of sand will have the lowest paste requirement for a certain slump. These should if possible include more than two sands with different FMs.

Since the optimum sand proportion changes only by 2.5 per cent, the combined aggregate grading still lies mainly in zone B, as shown in Figure 6.31. From this and from Figures 6.23 and 6.28 there is an indication that the best aggregate grading for the production of high strength concrete should lie in zone B. However, further work is needed before this can be substantiated.

Mix No.:	1	2	3	4	5	6	7
Percentage overfill (%)	1	2	2.5	2	3	3	3.5
Cement content (kg/m^3)	489.1	499.7	504.9	488.1	498.6	487.2	492.4
Coarse aggregate[(1)] (kg/m^3)	1026.7	1016.7	1011.7	1114.7	1103.9	1069.9	1064.7
Fine aggregate[(2)] (kg/m^3)	624.8	618.6	615.6	668.6	662.1	713.6	710.1
Proportion (%)	35	35	35	37.5	37.5	40	40
Free water (kg/m^3)	156.5	159.9	161.6	157.2	159.6	155.9	157.6
Total water (kg/m^3)	177.2	180.4	182.0	178.4	181.7	178.4	179.9
Slump (mm)	80	120	160	80	130	90	110

Mix No.:	8	9	10	11	12	13	14
Percentage overfill (%)	4	4	4.5	5	4	5	6
Cement content (kg/m^3)	497.6	486.3	491.5	496.6	475.0	485.4	495.6
Coarse aggregate[(1)] (kg/m^3)	1059.6	1026.0	1021.1	1016.2	985.6	976.2	967.0
Fine aggregate[(2)] (kg/m^3)	706.7	755.8	752.2	748.6	805.8	798.1	790.6
Proportion (%)	40	42.5	42.5	42.5	45	45	45
Free water (kg/m^3)	159.2	155.6	157.3	158.9	152.0	155.3	158.6
Total water (kg/m^3)	181.4	178.2	179.8	181.2	175.0	178.1	181.1
Slump (mm)	140	90	110	150	50	90	130

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.73.

Table 6.11: Optimisation of mix proportions for mixes with coarse sand and with a water-cement ratio of 0.32.

NOTE: The superplasticiser dosage (percentage of Conplast 430 solids by weight of cement) was 0.6 per cent for all mixes

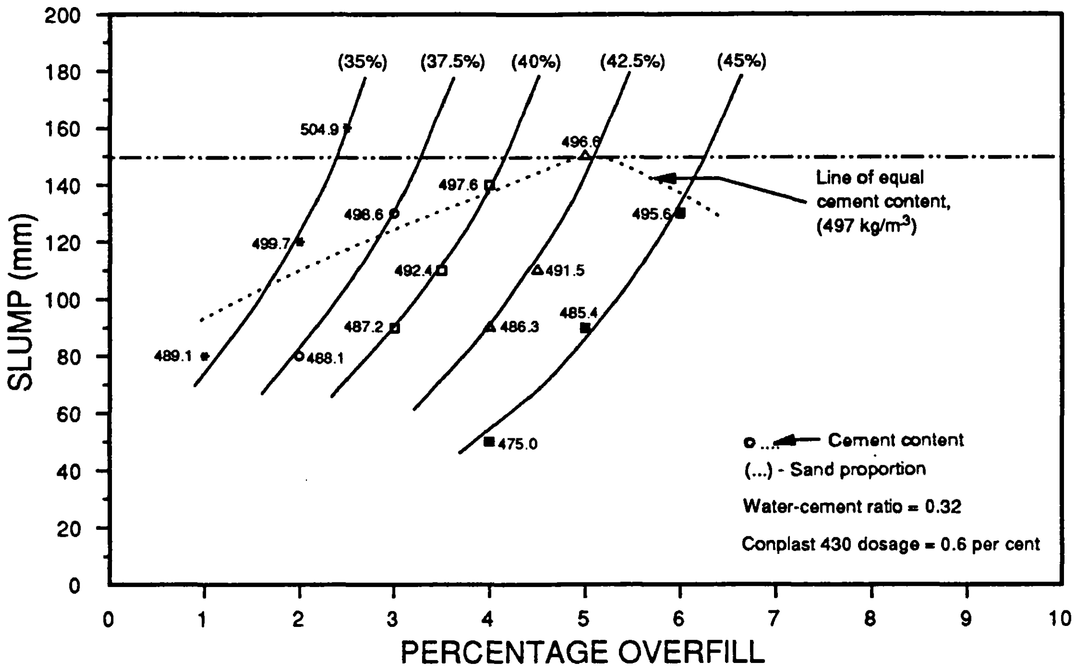


Figure 6.29: Slump versus overfill for mixes with coarse sand (FM = 2.73) at various proportions.

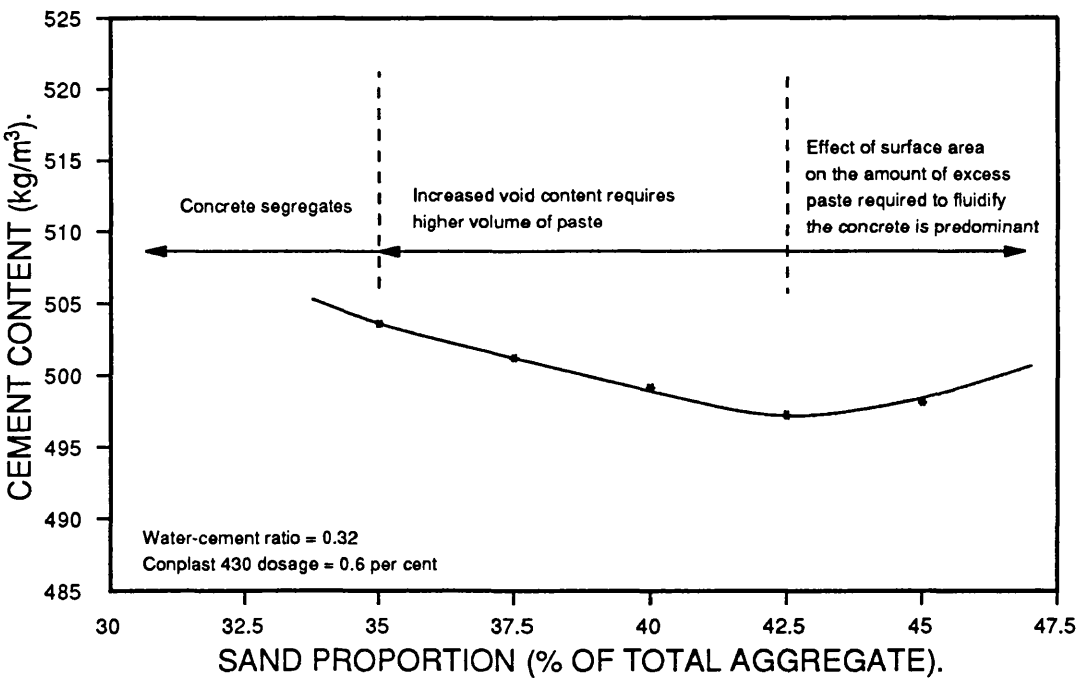


Figure 6.30: Cement contents required to produce a slump of 150 mm with various sand proportions.

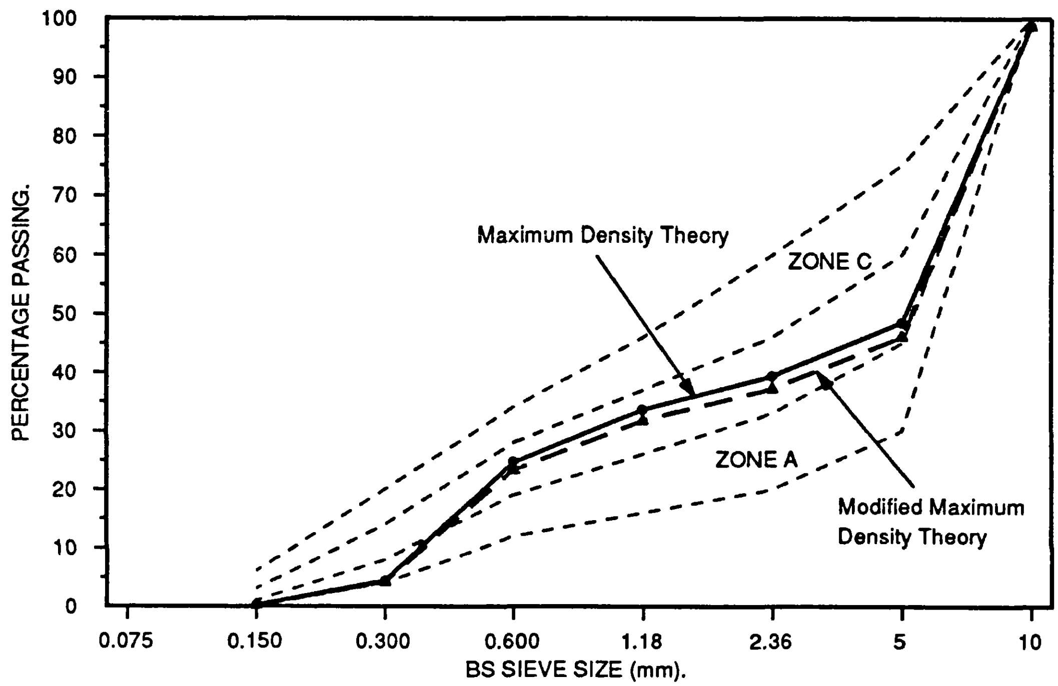


Figure 6.31: The grading curve of the "optimum proportions" of 10 mm granite and coarse sand (FM = 2.73), determined by the "Modified Maximum Density Theory".

6.5 CONCLUSIONS.

The "Modified Maximum Density Theory" can be used to optimise the relative proportions of the aggregates for a minimum required volume of cement paste. Mix proportions thus determined have been used for most of the subsequent research programme.

CHAPTER 7.
WORKABILITY OF
HIGH STRENGTH CONCRETE.

7.1 INTRODUCTION.

The required qualities of fresh concrete, which have usually been described with the word "workability", are first described, and the applicability of some of the accepted tests for measuring these is then reviewed.

Mineral admixtures have often been used in the production of high strength concrete, and therefore their effects on:

(i) the relation between water-binder ratio and superplasticiser dosage required for 150 mm slump,

and, (ii) the workability loss,

were determined. The parameters investigated have been described in Chapter 4.

The adsorption of the Conplast 430 (a naphthalene sulphonate formaldehyde) superplasticiser on the surface of cement (the main cementitious binder) and condensed silica fume (the mineral admixture with a very high specific area) have been determined spectrophotometrically.

As has been mentioned in Section 2.3.3, many researchers have advocated that the slump test may give the wrong impression of the workability of high strength concrete. Despite these warnings and recommendations that workability should be expressed according to classical rheological models such as that of Bingham, the slump test is still being used. An attempt was therefore made to use the Bingham model for high strength concrete, with tests carried out with Tattersall's two point test. The parameters investigated were water-binder ratios between 0.20 and 0.38, fineness modulus and content of sand, type of superplasticiser, and partial cement replacement by PFA, GGBFS and CSF.

7.2 "WORKABILITY" AND ACCEPTED "WORKABILITY" TESTS.

7.2.1 "Workability".

Although the properties of fresh concrete should be considered on the basis that they are important only in so far as they have an effect on the properties of the hardened concrete, it must be recognised that this effect may be a far-reaching one. The hardened concrete is required to satisfy more or less well defined standards with respect to shape, finish, strength, durability, shrinkage and creep, and to do so at as low a cost as possible. Because of these requirements, the fresh concrete must be of a suitable composition in terms of quality and quantity of cement, aggregates and admixtures and must also be capable of[(242)]:

1. being mixed satisfactorily and transported by one or more of a variety of methods including dumper truck, mixer truck, conveyor belt, tremie and pumping;

2. flowing into all corners of the mould or formwork to fill it completely, a process that might be made more difficult by the presence of awkward sections or congested reinforcement;

3. being compacted to expel as much included air as possible, with or without the use of machine methods including extrusion, vibration and pressure;

4. providing a good surface finish from the formwork, without honeycombing, being capable of being finished on a free surface by trowelling or other process.

The term that has traditionally been used in concrete technology to embrace all these necessary qualities is "workability" or "rheology" of fresh concrete, and one might include under the same general heading the requirement of "stability", which means that the mix must be capable of resisting segregation and bleeding. Indeed, Ritchie[(243)] subdivides rheology of fresh concrete into stability, compactibility, and mobility, as shown in Figure 7.1.

Unfortunately, there is no acceptable test which will measure directly the workability as defined earlier. Numerous attempts have been made, however, to correlate workability with some easily determinable physical measurement, but none of these is fully satisfactory although they may provide useful information within a limited range of workability.

7.2.2 Review of the accepted "workability" tests.

According to Tattersall[242], workability, in common with any other physical property, may be formally defined as being described by a set of one or more constants, W_i say, and those constants must fulfil the following requirements:

1. They should be based on fundamental physical quantities, and the numerical values obtained should not depend in any way on the details of the apparatus with which they are measured;
2. They must be such that if the values of all the constants are numerically the same for two or more materials, those materials will behave in exactly the same way as each other in any set of practical circumstances. (The converse is not necessarily true, in that two materials may behave similarly in given circumstances even if their constants are numerically different).

The accepted "workability" tests included in the British Standards, like the slump test, Vebe consistometer test, and the compacting factor test, depend on the tacit assumption that the number of constants in the set W_i is only one, W_1, so that workability can be expressed as a single figure, such as the slump value. It is obvious that the results from any of these tests do not satisfy either of the two conditions, in that:

1. they depend strongly on the apparatus,

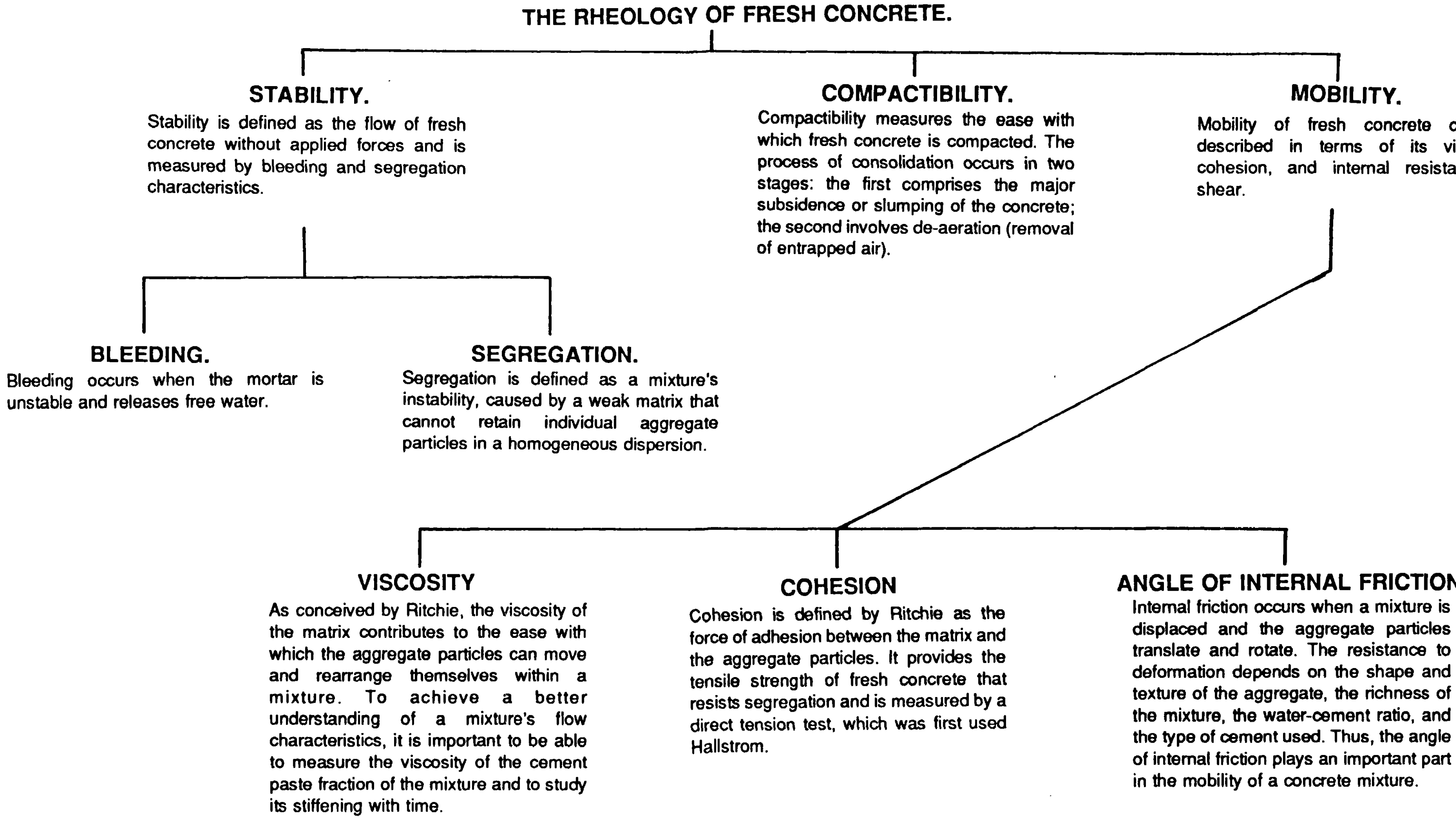


Figure 7.1: Rithie's subdivisions of the rheology of fresh concrete(243).

and, 2. each of these tests is capable of classifying as identical concretes those that can be seen to behave differently in other circumstances.

The above empirical tests that are included in British Standards and other specifications do not measure workability and results from them should not be used to imply that they do. The slump test measures by how much a concrete slumps after it has been placed in a standard manner in a standard mould and the mould has been removed; this is all that it measures and the result should be referred to only as the slump value.

The major criticisms that apply to all the empirical tests are as follows(242):

1. They are all single-point tests, i.e. in each test only one measurement is made and the result is quoted as a single figure. The practical outcome of this deficiency is that a given test may classify as identical two concretes that are subsequently found to behave differently on the job; Neville(117) has reported that "with different aggregates the same slump can be recorded for different workabilities, as indeed the slump bears no unique relation to the workability". To demonstrate this, Figure 7.2 shows specimens with equal slump values from two views. At the right, the specimens have been tapped with the tamping rod. The concrete in the upper view is a harsh mix, with a minimum of fines and water. The concrete in the lower view is a plastic cohesive mix; the surplus workability is needed for a different placement.

2. All the tests give results that depend on the dimensions and detailed arrangement of the apparatus. In many of them the result may also be influenced by minor variations in the technique of carrying out the test, i.e. they are operator-sensitive;

3. None of the tests are capable of dealing with concretes over the whole range of workabilities. For example, the slump test, which

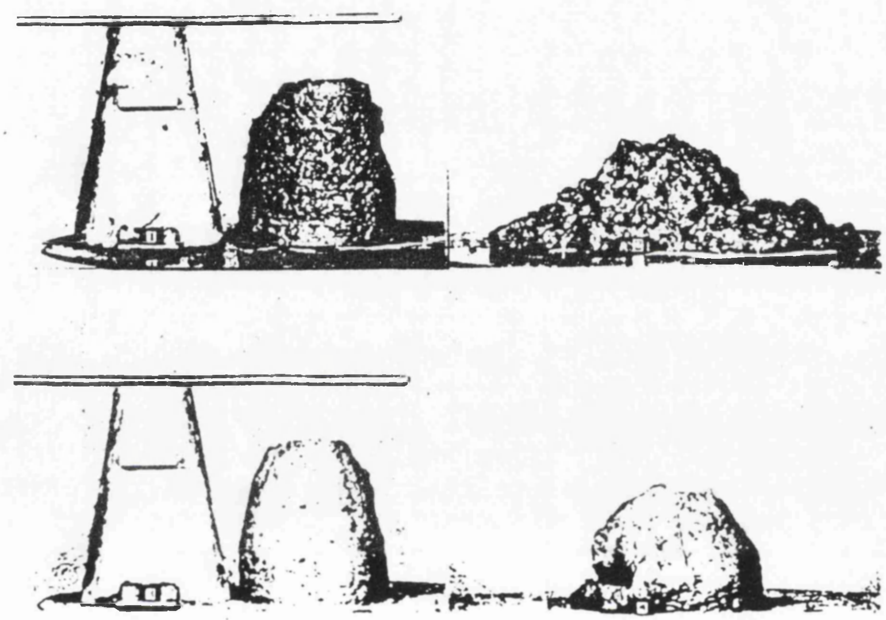


Figure 7.2: Specimens with equal slump values from two views. At the right, the specimens have been tapped with the tamping rod[117].

is the one most commonly used, is quite incapable of differentiating between two concretes of very low workability (zero slump), or two concretes of very high workability (collapse slump).

In addition to these criticisms, which generally apply to all the empirical tests, there are serious faults to each test:

(I) Slump test.

Results of the slump test on nominally identical mixes may be very variable not only quantitatively but qualitatively. For example, Glanville et al[244] found that, for one particular mix, the addition of a small amount of water would result in collapse or shear slumps, while only a little less water would give much more uniform slumps of less than 25 mm.

(II) The Vebe consistometer test.

The chief specific criticism of the Vebe consistometer test is that both end-points are very badly defined. The start is vague because it is related to the beginning of a vibration process that takes time to build up, and the finish is often difficult to assess because it is approached at a decreasing rate, or even asymptotically. Two investigators[245,246] have suggested the use of settlement-time recorders, in an effort to overcome the end point difficulties, but Hughes and Bahramian[247] found that the resulting curve does not facilitate more accurate assessment of the Vebe time. They do, however, suggest that the area under the curve can be used to give an indication of the cohesiveness of the concrete.

Bahrner[248] introduced a correction factor to the Vebe time, t, and expressed his results in Vebe degrees given by $(V'/V)t$ where V and V' are the volumes of the concrete before and after vibration. Hughes and Bahramian have pointed out that, since a concrete that loses its air voids in the making of the slump cone will have no correction applied to the VeBe time, whereas a less workable concrete that does not lose air until vibration is applied will have its Vebe time reduced, the effect of the correction factor is the opposite to what might be expected. They concluded that the use of Vebe degrees instead of Vebe time was an unnecessary complication.

(III) The compacting factor test.

The compacting factor test is designed to measure compactibility. Although the test has a wide range of applications, it has some limitations. Cohesive mixtures stick in the hoppers of the test apparatus and mixtures with low to very low workabilities produce wide variations in results. Cusens[(245)] has reported that, for concrete of very low workability, the actual amount of work required for full compaction depends on the richness of the mix while the compacting factor does not; leaner mixes need more work than richer ones. This means that the implied assumption that all mixes with the same compacting factor require the same amount of useful work is not always justified. Likewise the assumption that the wasted work (the work done against the surface friction) represents a constant proportion of the total work done reqardless of the properties of the mix is not correct.

These tests, quoting Tattersall[(242)]:

> "With all their faults, they have made progress in concrete mix design possible. There is not, however, any justification for their further proliferation and yet more of them appear each year, particularly when their birth is encouraged by some other development in concrete materials technology. However, the need is for a test that satisfies the conditions listed earlier, and that is also sufficiently robust and simple to operate that it can be used on site or in the plant by a semi-skilled operator. Perhaps the best chance of developing such a test lies in the application and extension of established rheological techniques."

Tattersall has developed a two point apparatus for measuring the workability of concrete, and the analysis of the results obtained using this assumes a Bingham model, as mentioned in Section 2.3.3. This apparatus has been used in this research programme and it is described in more detail in Section 7.6.

7.3 SUPERPLASTICISER DOSAGE VERSUS WATER-BINDER RATIO.

The slump test, despite its limitations described in the previous section, has been used extensively, because of its simplicity, in site work all over the world. The BRE report "Design of normal concrete mixes" also uses the slump test, as well as the Vebe test, as the meansby which the workability of the concrete is specified. This test was therefore used to study the relation between water-binder ratio and superplasticiser dosage required for a 150 mm slump concrete made using various binder combinations. Details of the mixes used are shown in Tables 7.1 to 7.5. The paste volume is 29.3 per cent for all OPC mixes. This means that the cement content varies from 471 to 567 kg/m^3 for water-cement ratios of 0.38 and 0.20 respectively. Substitution by mineral admixtures was on a one to one basis by weight. The paste volume was not adjusted to take account of the lower specific gravities of these admixtures. At this stage of the research programme, the experiments described in Section 4.2.4, Part (I): "The effect of coarse aggregate on the compressive strength of concrete", had already been completed. The results from these, which are described in Chapter 9, led to the decision, as has already been mentioned in Chapter 6, to use 10 mm granite aggregate for most of the reasearch programme. This aggregate was also used for this series of experiments.

7.3.1 Experimental procedure.

The mixing procedure and testing of concrete for slump were as described in Chapter 5.

7.3.2 Results and discussion.

The effect of pulverised fuel ash (PFA), at cement replacement levels of 10, 20, 30 and

MIX No. :	A1.	A2.	A3.	A4.	A5.	A6.
Cement content (kg/m^3)	421	439	460	482	510	525
Coarse aggregate(1) (kg/m^3)	1115	1115	1115	1115	1115	1115
Fine aggregate(2) (kg/m^3)	670	670	670	670	670	670
Free water (kg/m^3)	160	154	147	140	133	126
Total water (kg/m^3)	184	178	171	164	157	150
Free water-cement ratio	0.38	0.35	0.32	0.29	0.26	0.24
Superplasticiser dosage(3) (%)	0.5	0.6	0.8	1.1	1.5	2.3
Slump (mm)	160	160	145	140	150	155

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1.
(3) Dosage is expressed as a percentage of Conplast 430 solids by weight of cement.

Table 7.1: Details of OPC mixes tested to determine the relation between water-cement ratio and superplasticiser dosage required for concrete of 150 mm slump.

NOTE: Mixes have been designed according to the "Modified Maximum Density Theory" at a constant voids overfill of 4 per cent.

MIX No. :	B1.	B2.	B3.	B4.	B5.	B6.
Cement content (kg/m^3)	414	368	322	276	459	408
PFA content (kg/m^3)	46	92	138	184	51	102
Level of cement replacement (%)	10	20	30	40	10	20
Total binder content (kg/m^3)	460	460	460	460	510	510
Coarse aggregate[1] (kg/m^3)	1115	1115	1115	1115	1115	1115
Fine aggregate[2] (kg/m^3)	670	670	670	670	670	670
Free water (kg/m^3)	147	147	147	147	133	133
Total water (kg/m^3)	171	171	171	171	157	157
Free water-binder ratio	0.32	0.32	0.32	0.32	0.26	0.26
Superplasticiser dosage[3] (%)	0.6	0.5	0.4	0.3	1.15	1.0
Slump (mm)	150	145	150	140	160	150

MIX No. :	B7.	B8.	B9.	B10.	B11.	B12.
Cement content (kg/m^3)	357	306	483	430	510	340
PFA content (kg/m^3)	153	204	54	107	322	227
Level of cement replacement (%)	30	40	10	20	215	40
Total binder content (kg/m^3)	510	510	537	537	40	567
Coarse aggregate[1] (kg/m^3)	1115	1115	1115	1115	1115	1115
Fine aggregate[2] (kg/m^3)	670	670	670	670	670	670
Free water (kg/m^3)	133	133	124	124	124	113
Total water (kg/m^3)	157	157	148	148	148	137
Free water-binder ratio	0.26	0.26	0.23	0.23	0.23	0.20
Superplasticiser dosage[3] (%)	0.9	0.8	1.8	1.6	1.25	2.3
Slump (mm)	155	160	155	150	150	160

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1.
(3) Dosage is expressed as a percentage of Conplast 430 solids by weight of cement.

NOTE: Substitution by PFA was on a one to one basis by weight. The paste volume was not adjusted to take account of the lower specific gravity of the PFA.

Table 7.2: Details of PFA mixes tested to determine the relation between water-binder ratio and superplasticiser dosage required for concrete of 150 mm slump.

MIX No. :	C1.	C2.	C3.	C4.	C5.	C6.
Cement content (kg/m^3)	414	322	184	459	408	357
GGBFS content (kg/m^3)	46	138	276	51	102	153
Level of cement replacement (%)	10	30	60	10	20	30
Total binder content (kg/m^3)	460	460	460	510	510	510
Coarse aggregate(1) (kg/m^3)	1115	1115	1115	1115	1115	1115
Fine aggregate(2) (kg/m^3)	670	670	670	670	670	670
Free water (kg/m^3)	147	147	147	133	133	133
Total water (kg/m^3)	171	171	171	157	157	157
Free water-binder ratio	0.32	0.32	0.32	0.26	0.26	0.26
Superplasticiser dosage(3) (%)	0.7	0.5	0.3	1.3	1.1	1.0
Slump (mm)	150	160	155	165	150	150

MIX No. :	C7.	C8.	C9.	C10.	C11.	C12.
Cement content (kg/m^3)	280	204	483	430	215	227
GGBFS content (kg/m^3)	230	306	54	107	322	340
Level of cement replacement (%)	45	60	10	20	60	60
Total binder content (kg/m^3)	510	510	537	537	537	567
Coarse aggregate(1) (kg/m^3)	1115	1115	1115	1115	1115	1115
Fine aggregate(2) (kg/m^3)	670	670	670	670	670	670
Free water (kg/m^3)	133	133	124	124	124	113
Total water (kg/m^3)	157	157	148	148	148	137
Free water-binder ratio	0.26	0.26	0.23	0.23	0.23	0.20
Superplasticiser dosage(3) (%)	0.9	0.8	2.3	1.55	1.15	2.3
Slump (mm)	140	145	150	140	150	150

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1.
(3) Dosage is expressed as a percentage of Conplast 430 solids by weight of cement.

NOTE: Substitution by GGBFS was on a one to one basis by weight. The paste volume was not adjusted to take account of the lower specific gravity of the GGBFS.

Table 7.3: Details of GGBFS mixes tested to determine the relation between water-binder ratio and superplasticiser dosage required for concrete of 150 mm slump.

MIX No. :	D1.	D2.	D3.	D4.	D5.	D6.	D7.	D8.
Cement content (kg/m³)	400	379	358	417	414	391	434	484
CSF content (kg/m³)	21	42	63	22	46	69	48	26
Level of cement replacement (%)	5	10	15	5	10	15	10	5
Total binder content (kg/m³)	421	421	421	439	460	460	482	510
Coarse aggregate(1) (kg/m³)	1115	1115	1115	1115	1115	1115	1115	1115
Fine aggregate(2) (kg/m³)	670	670	670	670	670	670	670	670
Free water (kg/m³)	160	160	160	154	147	147	140	133
Total water (kg/m³)	184	184	184	178	171	171	164	157
Free water-binder ratio	0.38	0.38	0.38	0.35	0.32	0.32	0.29	0.26
Superplasticiser dosage(3) (%)	0.55	0.75	0.9	0.6	1.0	1.1	1.1	1.1
Slump (mm)	145	155	150	140	140	150	150	145

MIX No. :	D9.	D10.	D11.	D12.	D13.	D14.	D15.
Cement content (kg/m³)	459	433	519	491	464	528	510
CSF content (kg/m³)	51	77	27	55	82	28	57
Level of cement replacement (%)	10	15	5	10	15	5	10
Total binder content (kg/m³)	510	510	546	546	546	556	567
Coarse aggregate(1) (kg/m³)	1115	1115	1115	1115	1115	1115	1115
Fine aggregate(2) (kg/m³)	670	670	670	670	670	670	670
Free water (kg/m³)	133	133	120	120	120	117	113
Total water (kg/m³)	157	157	144	144	144	141	137
Free water-binder ratio	0.26	0.26	0.22	0.22	0.22	0.21	0.20
Superplasticiser dosage(3) (%)	1.25	1.35	1.6	1.7	2.0	2.3	2.3
Slump (mm)	140	160	150	160	150	145	140

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1.
(3) Dosage is expressed as a percentage of Conplast 430 solids by weight of cement.

NOTE: Substitution by CSF was on a one to one basis by weight. The paste volume was not adjusted to take account of the lower specific gravity of the CSF.

Table 7.4: Details of CSF mixes tested to determine the relation between water-binder ratio and superplasticiser dosage required for concrete of 150 mm slump.

MIX No. :	E1.	E2.	E3.	E4.
Cement content (kg/m^3)	168	175	204	226
GGBFS content (kg/m^3)	211	220	255	284
CSF content (kg/m^3)	42	44	51	57
Level of cement replacement (%)	GGBFS = 50 CSF = 10	GGBFS = 50 CSF = 10	GGBFS = 50 CSF = 10	GGBFS = 50 CSF = 10
Total binder content (kg/m^3)	421	439	510	567
Coarse aggregate(1) (kg/m^3)	1115	1115	1115	1115
Fine aggregate(2) (kg/m^3)	670	670	670	670
Free water (kg/m^3)	160	154	133	113
Total water (kg/m^3)	184	178	157	137
Free water-binder ratio	0.38	0.35	0.26	0.20
Superplasticiser dosage(3) (%)	0.55	0.6	0.85	1.3
Slump (mm)	140	140	155	160

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1.
(3) Dosage is expressed as a percentage of Conplast 430 solids by weight of cement.

NOTE: Substitution by CSF and GGBFS was on a one to one basis by weight. The paste volume was not adjusted to take account of the lower specific gravities of CSF and GGBFS.

Table 7.5: Details of CSF/GGBFS mixes used to determine the relation between water-binder ratio and superplasticiser dosage required for concrete of 150 mm slump.

40 per cent, on the relation between water-binder ratio and superplasticiser dosage required for a concrete with a slump of 150 mm is shown in Figure 7.3. Partial cement replacement by PFA causes a reduction in the amount of superplasticiser dosage required for a given degree of workability from that required for an equivalent concrete without PFA. An improvement of workability has also been found for non-superplasticised concrete, (see page 113). The small size and the essentially spherical form of low-calcium PFA particles have been credited as the reason for these effects[150]. Advantage can therefore be taken of the improved workability to either reduce the superplasticiser dosage or reduce the water-binder ratio used in a concrete and yet to maintain the same workability as non-PFA concrete.

The effect of ground granulated blast furnace slag (GGBFS), at cement replacement levels of 10, 20, 30, 45 and 60 per cent, on the relation between water-binder ratio and superplasticiser dosage required for a concrete with a slump of 150 mm is shown in Figure 7.4. Partial cement replacement by GGBFS has been found, as for PFA, to cause a reduction in the superplasticiser dosage required for a given slump. This improvement of workability has been reported by Meusel and Rose[162] for non-superplasticised GGBFS concrete, (see page 127). Comparison of Figures 7.3 and 7.4, shows that the superplasticiser dosage reduction resulting from 10, 20, 30 and 40 per cent cement replacement levels by PFA correspond approximately to 15, 30, 45 and 60 per cent cement replacement levels by GGBFS.

The effect of CSF, at cement replacement levels of 5, 10 and 15 per cent, on the relation between water-binder ratio and superplasticizer dosage required for a concrete with a slump of 150 mm is shown in Figure 7.5. As has been mentioned in Section 3.2.6, Part (III), it is widely accepted that the use of CSF in concrete reduces workability and requires a consequent addition of superplasticiser. Indeed Parrott[32] warned that the addition of water to counteract the reduction of workability would produce an unacceptable strength penalty in high strength concrete. However, the results shown in Figure 7.5 show that above a certain superplasticiser dosage CSF has a water reducing effect. This is in agreement with the results of two teams from France[173,174]. They claim that the use of ultrafine particles, i.e. grain size smaller than that of cement, facilitates the

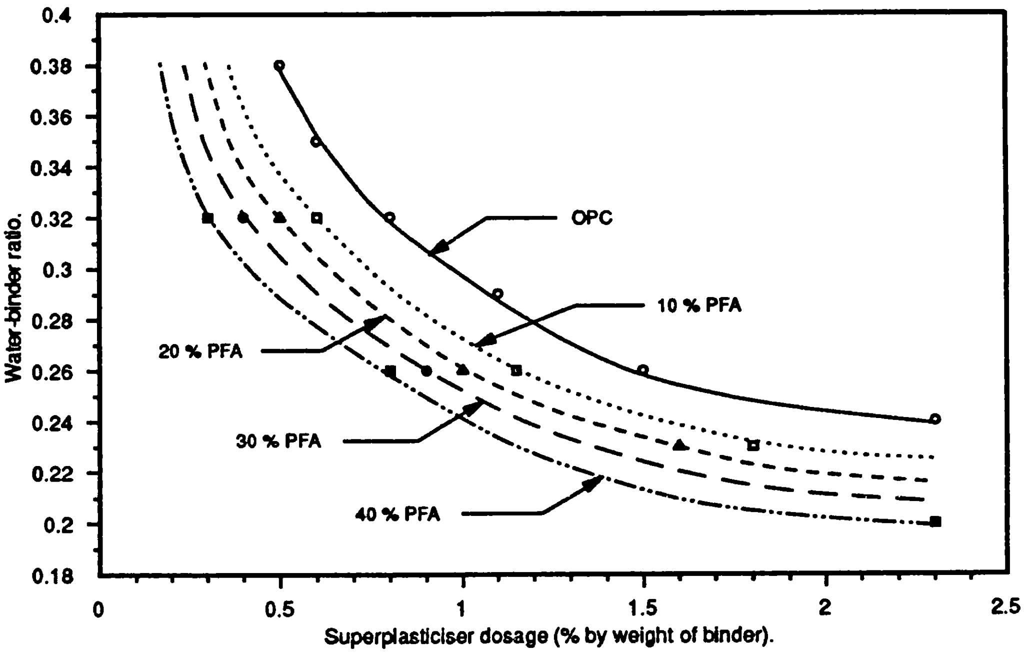


Figure 7.3: The effect of partial cement replacement by PFA on the superplasticiser dosage required for concrete mixes with a slump of 150 mm.

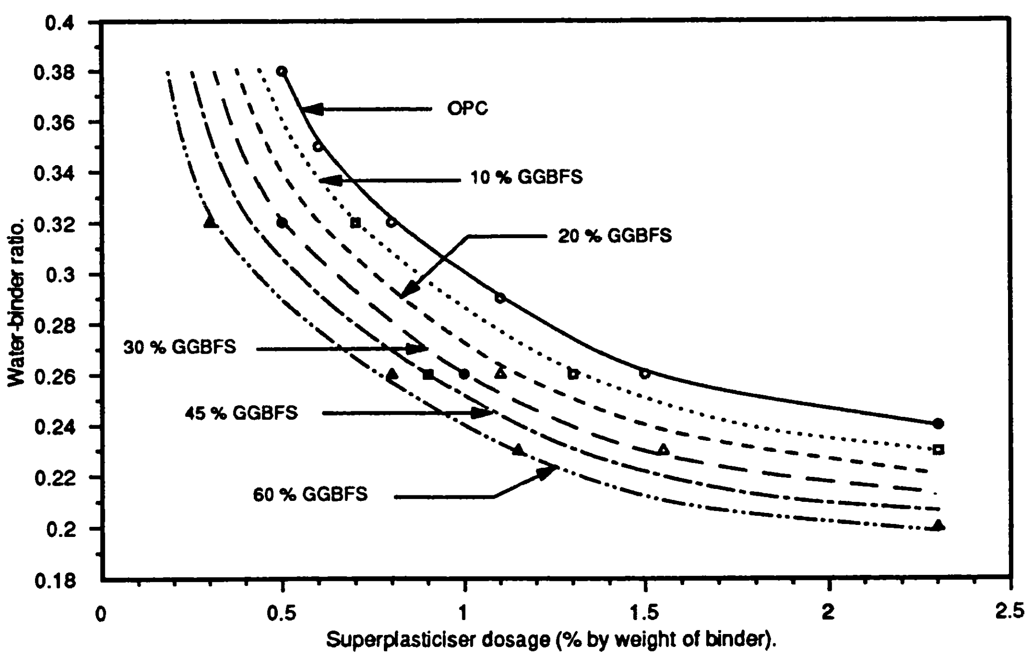


Figure 7.4: The effect of partial cement replacement by GGBFS on the superplasticiser dosage required for concrete mixes with a slump of 150 mm.

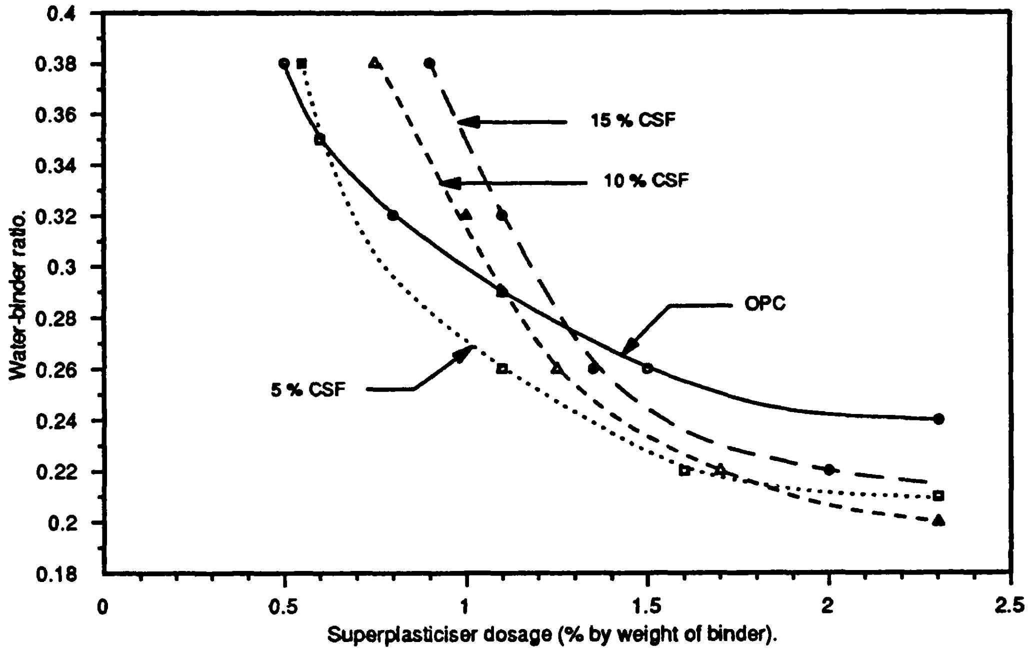


Figure 7.5: The effect of partial cement replacement by CSF on the superplasticiser dosage required for concrete mixes with a slump of 150 mm.

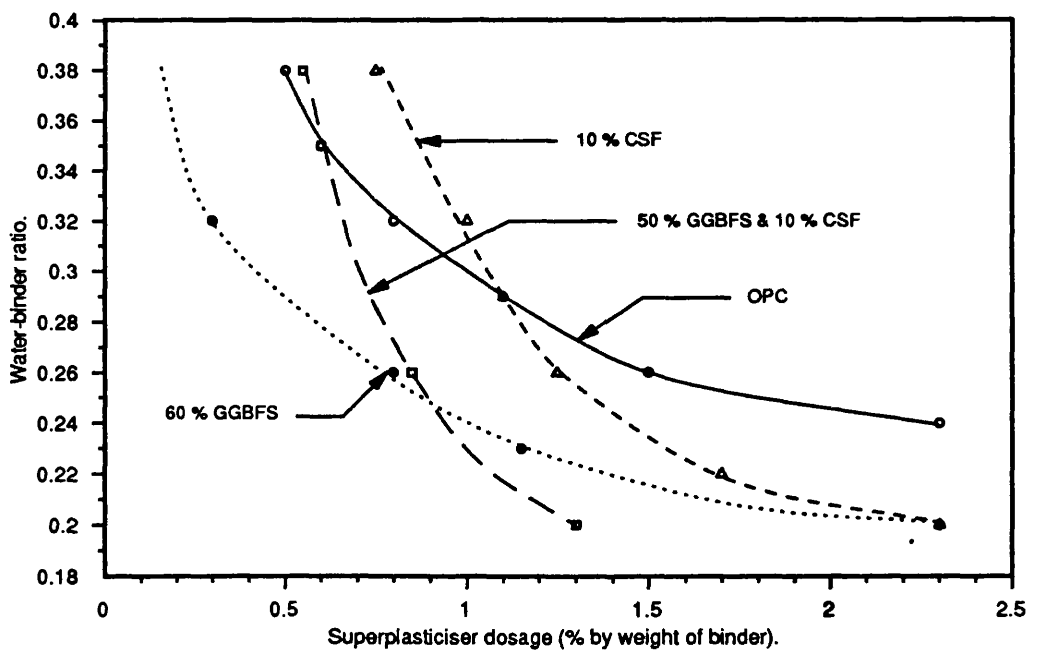


Figure 7.6: The effect of partial cement replacement by a combination of 50 per cent GGBFS and 10 per cent CSF on the superplasticiser dosage required for concrete mixes with a slump of 150 mm.

production of low water-binder ratio concrete by the filler effect: the grains fill the voids between those of cement thus reducing the water requirement. For this filler effect to occur it appears that the CSF paticles have to be in a dispersed and not agglomerated state. A certain superplasticiser dosage is therefore required to reduce sufficiently the cohesion of CSF particles so as to disperse them.

Results for partial cement replacement with a combination of 50 per cent GGBFS and 10 per cent CSF are shown in Figure 7.6. The decrease of workability at low superplasticiser dosages resulting from the use of CSF can be partially counteracted by the improvement of workability resulting from the use of GGBFS. At higher superplasticiser dosages, when CSF particles are well dispersed and act as fillers, CSF may have a water-reducing effect even for the GGBFS mixes, i.e. an 150 mm slump GGBFS-CSF concrete with a water-binder ratio of 0.20 requires only 1.3 per cent superplasticiser dosage as compared to 2.3 per cent for a 10 per cent CSF mix or a 60 per cent GGBFS mix.

The efficiency of the superplasticiser in reducing the water-cement ratio is reduced at higher dosage levels, e.g., with a reduction in the water-cement ratio of the OPC mix from 0.38 to 0.36 the superplasticiser dosage needs to be increased from 0.5 to 0.6 per cent, whereas a reduction of the water-cement ratio from 0.26 to 0.24 required an increase of the superplasticiser dosage from 1.5 to 2.3 per cent. The mixes described in Section 6.3.2, with a water-cement ratio of 0.26, required a superplasticiser dosage of 2.5 per cent for 15 mm slump. Doubling the superplasticiser dosage to 5.0 per cent only increased the slump to 35 mm. A further increase of superplasticiser dosage to 7.5 per cent only increased the slump to 60 mm. This indicates a possible point of saturation for the quantity of adsorbed Conplast 430 (sulphonated naphthalene formaldehyde condensate) on the cement particles. This was therefore investigated by spectrophotometrical determinations of adsorption of Conplast 430 on the cement particles, and the results are reported later in Section 7.5.

7.4 WORKABILITY LOSS.

Workability of high strength concrete has, as has already been mentioned in Section 2.3.3, been reported to decline rapidly with time. A limited number of tests were therefore performed to quantify this and assess whether it can be minimised by the use of mineral admixtures. Details of the mixes used are shown in Table 7.6.

7.4.1 Experimental procedure.

The mixing procedure for concrete was as described in Chapter 5. However, the concrete was returned to the mixer immediately after having been tested for slump, and remixing started. Almost continual mixing, only stopping at intervals of ten minutes in order to test the concrete for slump, was chosen as this simulates best the condition of the concrete during transportation.

7.4.2 Results and discussion.

Workability of high strength concrete was found to decline rapidly with time after mixing, as shown in Figure 7.7. The 200 mm slumps of concretes with a water-binder ratio of 0.26 were reduced to below 50 mm in only 50 minutes. Partial cement replacement by 10 per cent CSF and 40 per cent PFA have not been found to decrease the workability loss and GGBFS decreases the workability loss only at the highest level of cement replacement tested, i.e. 60 per cent. However, even at this level, the decrease of the slump from around 200 mm to 25 mm is only delayed by 10 minutes. It may therefore be concluded that these mineral admixtures do not significantly contribute to decreasing the workability loss, which appears to be mainly due to the limited duration of the admixture's fluidifying action, (see page 91). It must also be noted that the superplasticiser dosages used to obtain approximately the same slump as an OPC concrete were much lower with the concretes containing mineral admixtures. As shown in Table 7.6, the 60 per cent GGBFS and the 40 per cent PFA concrete required 1.1 per cent superplasticiser dosage for a 200 mm

MIX No. :	1.	2.	3.	4.	5.
Cement content (kg/m³)	510	459	204	357	306
PFA content (kg/m³)					204
GGBFS content (kg/m³)		51	306	153	
CSF content (kg/m³)					
Level of cement replacement (%)		CSF=10	GGBFS=60	GGBFS=30	PFA=40
Total binder content (kg/m³)	510	510	510	510	510
Coarse aggregate(1) (kg/m³)	1115	1115	1115	1115	1115
Fine aggregate(2) (kg/m³)	670	670	670	670	670
Free water (kg/m³)	133	133	133	133	133
Total water (kg/m³)	157	157	157	157	157
Free water-cement ratio	0.26	0.26	0.26	0.26	0.26
Superplasticiser dosage(3) (%)	1.6	1.1	1.1	1.3	1.1
Time after mixing	SLUMP (mm)	SLUMP (mm)	SLUMP (mm)	SLUMP (mm)	SLUMP (mm)
0 minutes	200	200	220	195	210
10 minutes	200	165	220	195	190
20 minutes	160	110	220	130	100
30 minutes	65	65	190	50	20
40 minutes	10	25	90		
50 minutes			20		

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1.
(3) Dosage is expressed as a percentage of Conplast 430 solids by weight of total binder.

Table 7.6: Details of the mixes used to determine the rate of workability loss of concrete mixes with different binders at a water-binder ratio of 0.26.

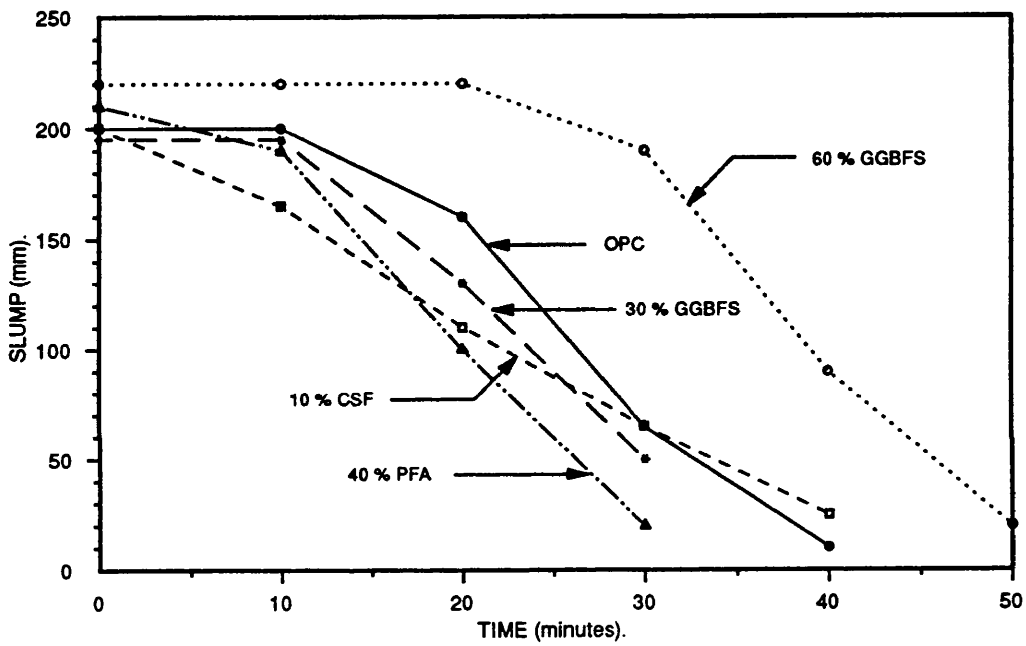


Figure 7.7: The workability loss of concrete mixes with a water-binder ratio of 0.26.

slump as compared to 1.6 per cent for the OPC concrete. It has been suggested that higher superplasticiser dosages lead to longer retardation times for the setting of concrete and they, therefore, may decrease the workability loss. Almeida and Goncalves[156] having carried out tests with 10 per cent cement replacement by CSF, PFA and a natural pozzolan, at a constant superplasticiser dosage, have concluded, as mentioned in Section 3.2.6, Part (I), that the rate of slump loss may be reduced by the presence of these admixtures. This was based on the fact that the OPC concrete was the one that most quickly reached zero slump. Their conclusion is however questioned as their OPC concrete had a higher percentage of the initial slump after 40 minutes than the PFA and natural pozzolan concretes.

Even if slump loss is not significantly decreased by mineral admixtures, advantage can be taken of the improvement of workability. A pozzolanic concrete of much higher workability can be obtained with the same superplasticiser dosage as an OPC concrete, thus allowing for the workability loss experienced during transportation. It is recommended that this is investigated in connection with actual projects. If the workability is still a problem, split addition of the superplasticiser dosage, as has been recommended by Murata et al[20], and Lacroix et Jaugey[31], (see page 91), should also be investigated.

7.5 ADSORPTION TESTS.

Adsorption tests were carried out in order to determine the maximum quantities of Conplast 430 (a naphthalene sulphonate formaldehyde) superplasticiser adsorbed on the surface of cement and condensed silica fume. Conplast 430 was supplied as liquid and the active solids content was, as mentioned in Chapter 5, 40 per cent. This value is not very accurately adhered to, and therefore I tried to get supplies of Conplast 430 in powder form. In the meanwhile, in order to become familiar with the apparatus adsorption studies of liquid Conplast 430 on cement and CSF were carried out. These were chosen because of the large difference in their specific areas, 308 m^2/kg and approximately 20,000 m^2/kg for cement and CSF, respectively. The supplier of Conplast 430 later informed me that the powder form is not imported in the U.K.. The experiments performed with the liquid form are therefore described below:

7.5.1 Experimental procedure.

A similar procedure to the one used by Buil et al[130], which has been shown to produce satisfactory results, has been used:

> Suspensions of OPC and CSF in aqueous solutions (20 g in 1 litre) containing Conplast 430 (sulphonated naphthalene formaldehyde condensate) at various concentrations, and lime (1 g/l) were stirred for three hours at approximately 20^0C. An aqueous solution containing calcium hydroxide has been shown by Massazza et al[126] to be required in order to disperse the monosulphate which decomposes in contact with pure water. The three hour period was chosen to allow full adsorption of the admixture on C_3S, but was less than the five hours after which further adsorption occurs due to hydration, (see Figure 3.11). The solid particles were then separated from the solution by centrifuging at a 3,500 rpm for 20 minutes. The amount of Conplast 430 adsorbed by the OPC and CSF particles was determined by measuring the concentration decrease of the admixture in solution after contact with the OPC and CSF particles. The admixture concentration was measured by UV spectrophotometry with a CAMSPEC Spectrophotometer, model 300, at 375 nm wavelength. Information on the calibration procedure of the instrument and of the various solutions used for the test can be found in Appendix 3.

7.5.2 Results and discussion.

The adsorption isotherms are shown in Figure 7.8. OPC adsorbs 1.6 per cent of superplasticiser by weight and this occurs when the ratio of superplasticiser to OPC in the initial solution is 2.2 per cent. CSF adsorps 4.15 per cent and this occurs when the ratio

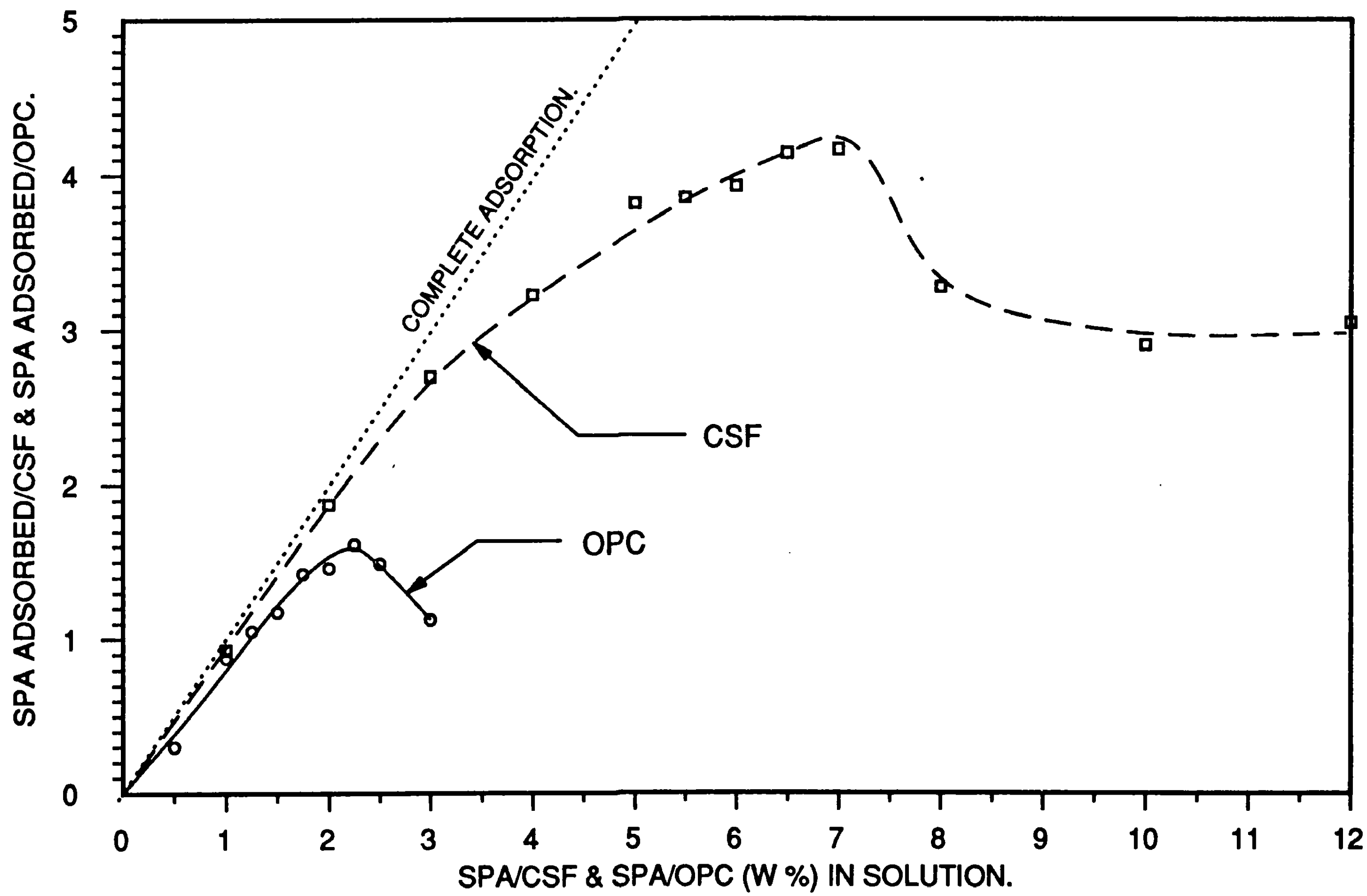


Figure 7.8: Adsorption of Conplast 430 (naphthalene formaldehyde condensate) superplasticiser on OPC and CSF particles.

of superplasticiser to CSF in solution is 7 per cent. The average CSF particle diameter being 100 times smaller than the average OPC particle, (see page 131), means that the CSF-OPC surface area ratio by weight is 143. (This ratio was calculated using specific gravities of 3.15 and 2.2 for cement and CSF, respectively). Since the CSF-OPC ratio of adsorbed superplasticiser by weight is 1.9, it appears that the saturation values determined represent multilayer adsorption with different number of layers for the OPC and CSF particles, (c.f. page 86).

It has been shown in the previous section that beyond 1.6 per cent superplastiser dosage by weight of cement the efficiency of the superplasticiser in reducing the water-cement ratio while retaining the same slump decreases significantly. Above 2.3 per cent superplasticiser dosage, no further reductions of the water-cement ratio can be achieved even by doubling the superplasticiser dosage. These values correspond to the superplasticiser dosage required for the cement particles to reach saturation and the superplasticiser-OPC ratio in solution required for the saturation point to be reached.

As CSF only replaces cement at small percentages the saturation point for the CSF-OPC mixture does not increase sufficiently to be detected by the slump test, as shown in Figure 7.6. By calculation, a 10 per cent CSF only changes the 1.6 per cent saturation value to 1.82.

The presence of a maximum in the adsorption isotherm beyond which the adsorption decreases has also been found by Massazza et al[126], and has tentatively been attributed to the formation of associations (micelles) among the isolated molecules (monomers), (see page 86).

7.6 TWO POINT WORKABILITY TESTS.

The workability tests described in the British Standards are all single point tests with serious deficiencies in each test, as has been mentioned in Section 7.2.2. The inapplicability of the slump test to high strength concrete mixes in particular has already been shown by, for example, Helland[80], (see Section 2.3.3), who also showed that Tattersall's two point apparatus may be more suitable. This apparatus was therefore used to study the workability of concrete mixes designed according to the "Modified Maximum Density Theory" with a constant voids overfill of 4 per cent. A common factor in these mixes was that they all had similar slump values. This was chosen so as to show the different behaviour of concretes that have been classified as identical by the slump test. As expected, preliminary tests showed that mixes with slump values less than 100 mm were overloading the machine; pressure readings were exceeding the allowable 650 psi[249]. A slump of 150 mm, as was used for the experiments in Sections 6.4 and 7.3, was found to be satisfactory and was therefore used.

In designing these mixes at a constant paste volume it meant that as the water-binder ratio was lowered an increase in the superplasticiser dosage was required to retain the slump at 150 mm. The effect of the following parameters on the yield (g) and plastic viscosity (h) values of 150 mm slump concrete were investigated:

1. Water-binder ratio,
2. fineness modulus (FM) and proportion of sand, (per cent of total aggregate),
3. type of superplasticiser,

and, 4. partial cement replacement by PFA, GGBFS and CSF. Replacement was on a one to one basis by weight.

7.6.1 Tattersall two-point test apparatus.

The development and theory of the Tattersall two point test apparatus, shown in Figure 7.9, devised to measure the workability of concrete has been well documented[242,249-257]. The concrete under test is contained within a cylindrical bowl and is sheared by a suitable impeller which is driven by an electric motor operating through an infinitely variable hydraulic transmission and a reduction gear. The pressure developed in the oil in the hydraulic transmission is measured by a Budenburg pressure gauge. The pressure produced by concrete shearing is obtained from the total pressure by subtracting the pressure produced by machine idling. This net value may be converted into impeller

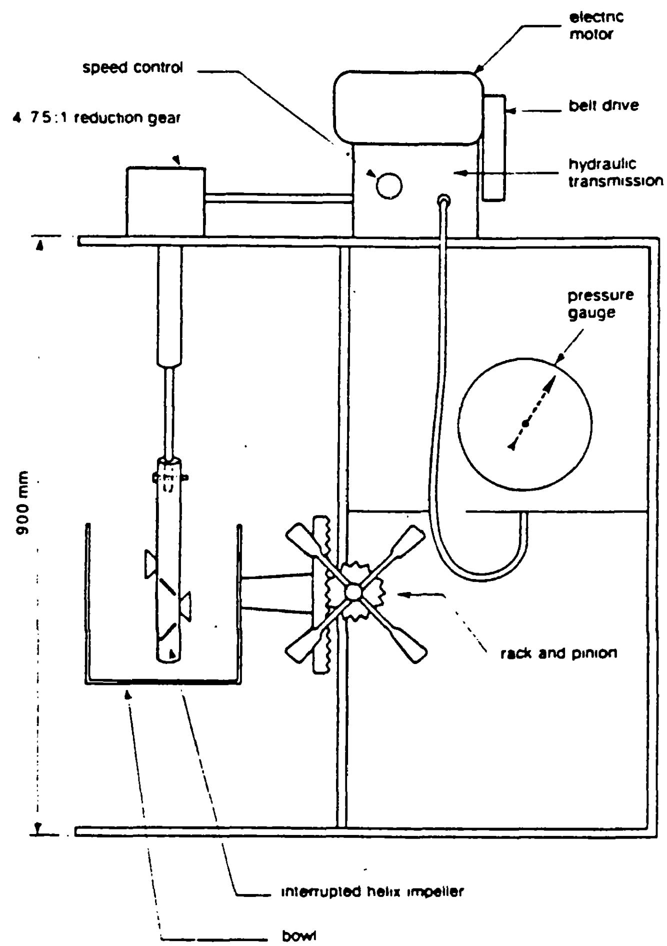


Figure 7.9: Tattersall two-point test apparatus[253].

torque after calibration, as shown in Appendix 4. The impeller torque T and speed N are found to be related by the linear equation:

$$T = g + hN$$

By measuring the torque produced on an impeller rotating in fresh concrete at various speed settings, the values of g and h can be determined, which therefore define the workability of a particular concrete mix. Different concretes at different workabilities produce different values of g and h. The resemblance of the equation to the rheological equation of Bingham flow is often noted (Bingham: $\tau = \tau_0 + \mu\gamma$ where τ = shear stress; τ_0 = yield value; μ = plastic viscosity, γ = shear rate). Theoretical justification that fresh concrete approximates to the Bingham model and that the two constants (g) and (h) are directly proportional to the Bingham constants τ_0 and μ has been provided by Tattersall(253).

7.6.2 Experimental procedure.

The mixing sequence was as described in Chapter 5. The concrete mixing time was however extended to a minimum of 15 minutes as is recommended in the operating instructions for the two point test apparatus(249). However, it was found impossible to consistently produce 150 mm slump concrete at exactly 15 minutes of mixing. This appears to have been due to the different mixing efficiencies of the mixers used; the small trial mixes were performed using the mixer with a capacity of 0.01 m^3, while the actual experiments used the mixer with a capacity of 0.02 m^3. To overcome this problem and to avoid the use of trial mixes, the concrete was designed to have a higher slump than 150 mm at 15 minutes, after which it was periodically tested for slump. Once the slump was close to 150 mm, which was found to be never longer than after 25 minutes of mixing, the concrete was tested in the two point apparatus. After this, which did not take longer than 10 minutes, the concrete was returned to the mixer, remixed for a further one minute, and again tested for slump. The average of the two slump values, i.e. before and after the two point testing of concrete, was calculated. The two slump values were found in most cases not to differ by more than 30 mm, despite the high workability loss of high strength concrete that has been reported in Section 7.4. The high shearing that the concrete undergoes in the two point apparatus may have reduced the rate of workability loss of the concrete.

7.6.3 Results and discussion.

The details of the mixes tested are given in Tables 7.7 and 7.8, and all the detailed experimental data can be found in Appendix 4.

(I) The effect of the water-cement ratio.

It was noticed during the slump test that as the water-cement ratio was reduced, the force required for the tamping rod to go through the concrete was increased. Also, when the cone of the slump test apparatus was lifted the concrete slumped at a much lower rate at the lower water-cement ratios. The workability obtained by the use of high dosages of plasticisers is, therefore, not directly comparable to workability as a result of high water content, despite the mixes having equal slump values.

Two point test results have shown, (Figure 7.10), that as the water-cement ratio was lowered the yield (g) and plastic viscosity (h) values increased despite the mixes all having a slump of 150 mm. The addition of a superplasticiser to a mix with a fixed water-cement ratio is known to mainly reduce the yield stress, i.e. its (g) value, in the fresh mix without significantly affecting the plastic viscosity, i.e. its (h) value. Decreasing the water-cement ratio increases both the (g) and the (h) values. The addition of a superplasticiser would therefore be expected to help decrease the rate at which the yield stress increases as the water-cement ratio is reduced, but not reduce the increase in the plastic viscosity. The increase in the yield value at constant slump was therefore somewhat unexpected, despite the increased force required for the tamping rod to go through the concrete. It seems to arise from the inherently higher solids content of the paste used as the water-

SERIES:	OPC MIXES WITH FINE SAND(1).							OPC MIXES WITH LOW SAND CONTENT(2).		
Mix No.:	1	2	3	4	5	6	7	8	9	10
Cement content (kg/m^3)	421	439	460	477	490	510	510	507	525	546
Coarse aggregate(3) (kg/m^3)	1115	1115	1115	1115	1115	1115	1115	1238	1238	1238
Fine aggregate(4) (kg/m^3)	670	670	670	670	670	670	670	526	526	526
Free water (kg/m^3)	160	154	147	143	137	133	133	152	147	142
Total water (kg/m^3)	184	178	171	167	161	157	157	174	169	164
Free water-binder ratio	0.38	0.35	0.32	0.30	0.28	0.26	0.26	0.30	0.28	0.26
Superplasticiser dosage(5) (%)	0.80	0.90	1.15	1.30	1.80	2.10	2.00	1.50	1.80	2.00

(1) Sand proportion of total aggregate = 37.5 per cent.
(2) Sand proportion of total aggregate = 30.0 per cent.
(3) 10 mm granite aggregate.
(4) Sand with fineness modulus of 2.1.
(5) Dosage is expressed as a percentage of Conplast 430 solids by weight of binder.

SERIES:	OPC MIXES WITH COARSE SAND.					OPC MIXES WITH CONPLAST M1.			
Mix No.:	11	12	13	14	15	16	17	18	19
Cement content (kg/m^3)	450	471	493	517	543	421	440	460	483
Coarse aggregate(1) (kg/m^3)	1013	1013	1013	1013	1013	1115	1115	1115	1115
Fine aggregate (kg/m^3) Fineness modulus:	749 2.73	749 2.73	749 2.73	749 2.73	749 2.73	670 2.1	670 2.1	670 2.1	670 2.1
Free water (kg/m^3)	171	165	158	150	141	160	154	154	140
Total water (kg/m^3)	194	187	180	172	163	184	177	178	164
Free water-binder ratio	0.38	0.35	0.32	0.29	0.26	0.38	0.35	0.32	0.29
Superplasticiser dosage(2) (%) Conplast type:	0.60 430	0.60 430	1.00 430	1.10 430	2.00 430	0.80 M1	0.90 M1	1.10 M1	1.40 M1

(1) 10 mm granite aggregate.
(2) Dosage is expressed as a percentage of superplasticiser solids by weight of binder.

Table 7.1: Details of the OPC mixes tested using Tattersall's two point test apparatus.

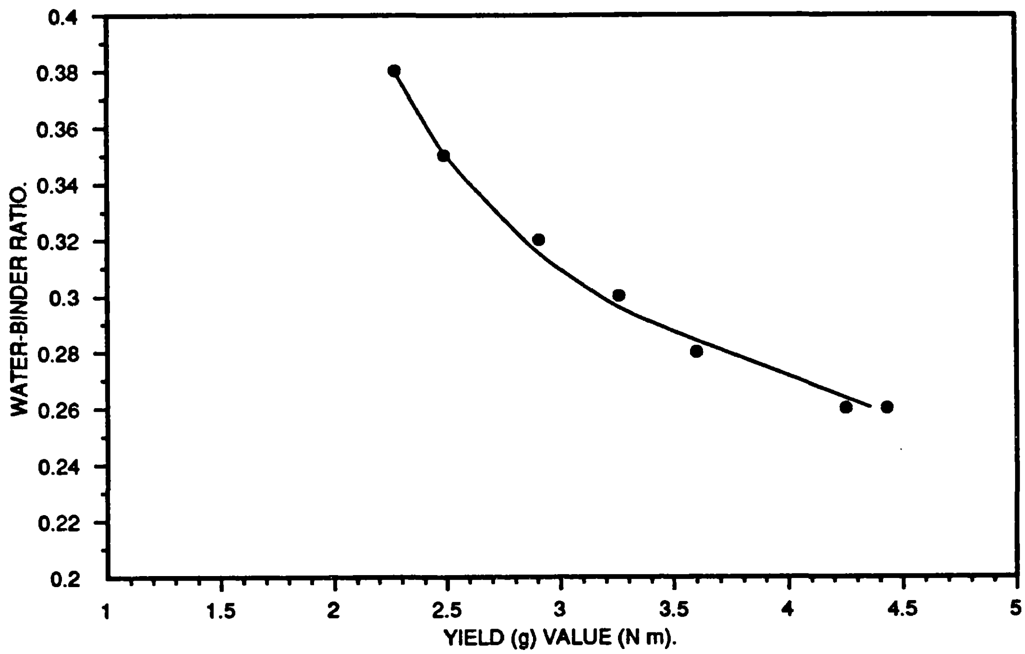
SERIES:	PFA MIXES.						GGBFS MIXES.			
Mix No.:	20	21	22	23	24	25	26	27	28	29
Cement content (kg/m³)	353	276	408	357	306	322	433	357	280	204
PFA content (kg/m³)	168	184	102	153	204	215				
GGBFS content (kg/m³)							77	153	230	306
Replacement level (%)	40	40	20	30	40	40	15	30	45	60
Total binder content (kg/m³)	421	460	510	510	510	537	510	510	510	510
Coarse aggregate(1) (kg/m³)	1115	1115	1115	1115	1115	1115	1115	1115	1115	1115
Fine aggregate(2) (kg/m³)	670	670	670	670	670	670	670	670	670	670
Free water (kg/m³)	160	147	133	133	133	124	133	133	133	133
Total water (kg/m³)	184	171	157	157	157	148	157	157	157	157
Free water-binder ratio	0.38	0.32	0.26	0.26	0.26	0.23	0.26	0.26	0.26	0.26
Superplasticiser dosage(3) (%)	0.25	0.70	1.45	1.25	1.15	1.80	1.90	1.50	1.20	1.10

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1.
(3) Dosage is expressed as a percentage of Conplast 430 solids by weight of binder.

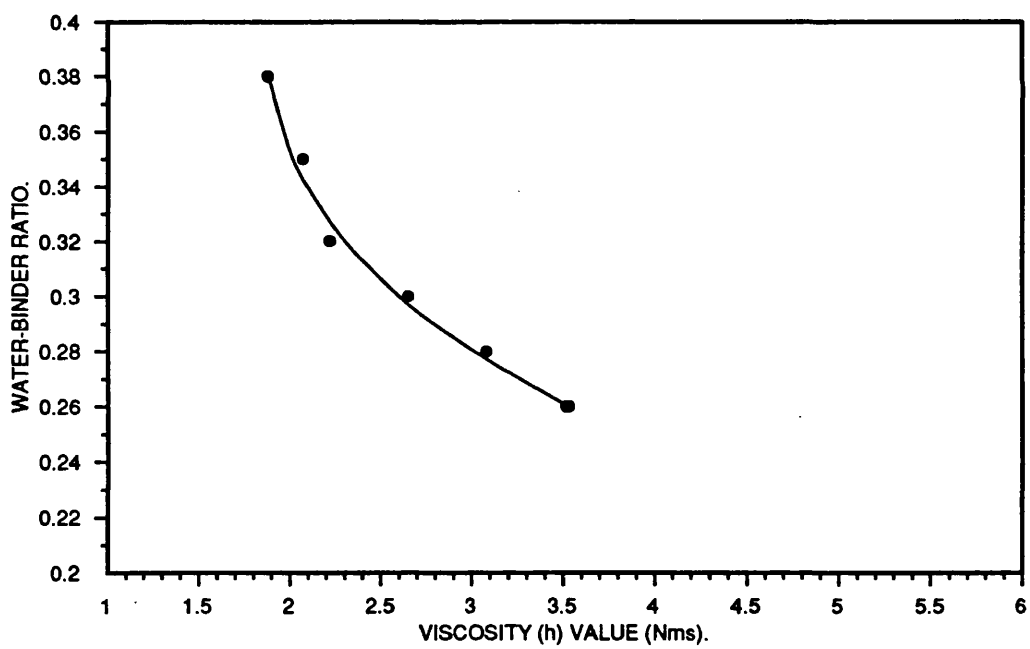
SERIES:	CSF MIXES.							CSF-GGBFS MIXES.		
Mix No.:	30	31	32	33	34	35	36	37	38	39
Cement content (kg/m³)	379	395	414	434	484	459	433	204	214	226
CSF content (kg/m³)	42	44	48	48	26	51	77	51	54	57
GGBFS content (kg/m³)								255	269	284
Replacement level CSF (%)	10	10	10	10	5	10	15	10	10	10
GGBFS (%)								50	50	50
Total binder content (kg/m³)	421	439	460	482	510	510	510	510	537	567
Coarse aggregate(1) (kg/m³)	1115	1115	1115	1115	1115	1115	1115	1115	1115	1115
Fine aggregate(2) (kg/m³)	670	670	670	670	670	670	670	670	670	670
Free water (kg/m³)	160	154	147	140	133	133	133	133	124	113
Total water (kg/m³)	184	178	171	164	157	157	157	157	148	137
Free water-binder ratio	0.38	0.35	0.32	0.29	0.26	0.26	0.26	0.26	0.23	0.20
Superplasticiser dosage(3) (%)	1.0	1.0	1.15	1.30	1.60	1.90	1.90	1.20	1.30	1.50

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1.
(3) Dosage is expressed as a percentage of Conplast 430 solids by weight of binder.

Table 7.2: Details of the PFA, GGBFS, and CSF mixes tested using Tattersall's two point test apparatus.



(a) Yield (g) value.



(b) Plastic viscosity (h) value.

Figure 7.10: Water-cement ratio versus yield (g) and plastic viscosity (h) values for OPC concrete mixes with a slump of approximately 150 mm.

NOTE: Sand proportion = 37.5 per cent.
FM of sand = 2.1

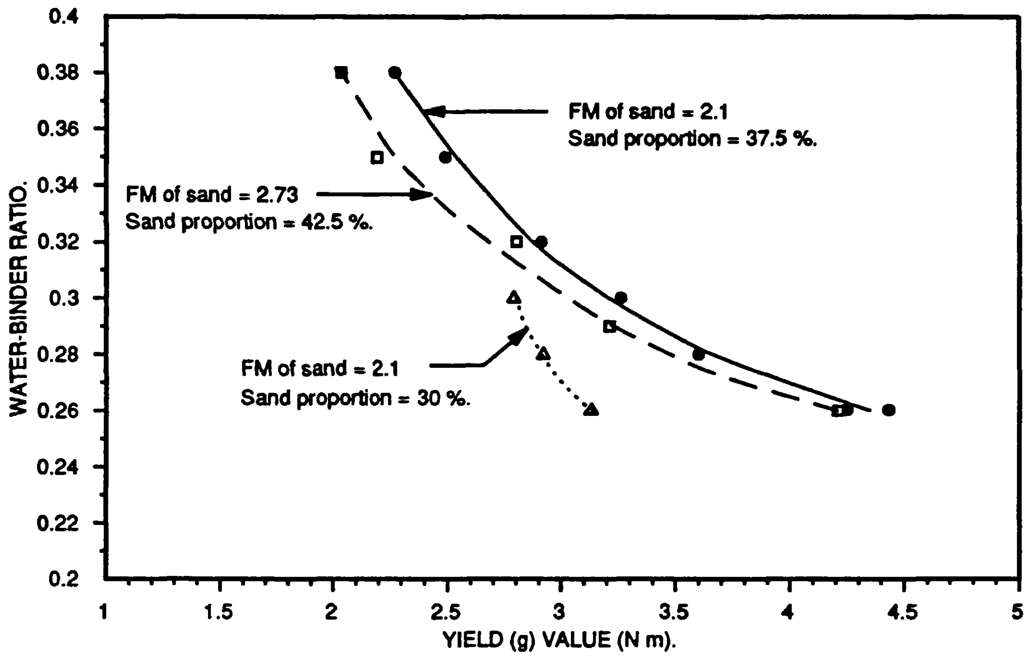
cement ratio was lowered.

(II) The effect of the fineness modulus and proportion of sand.

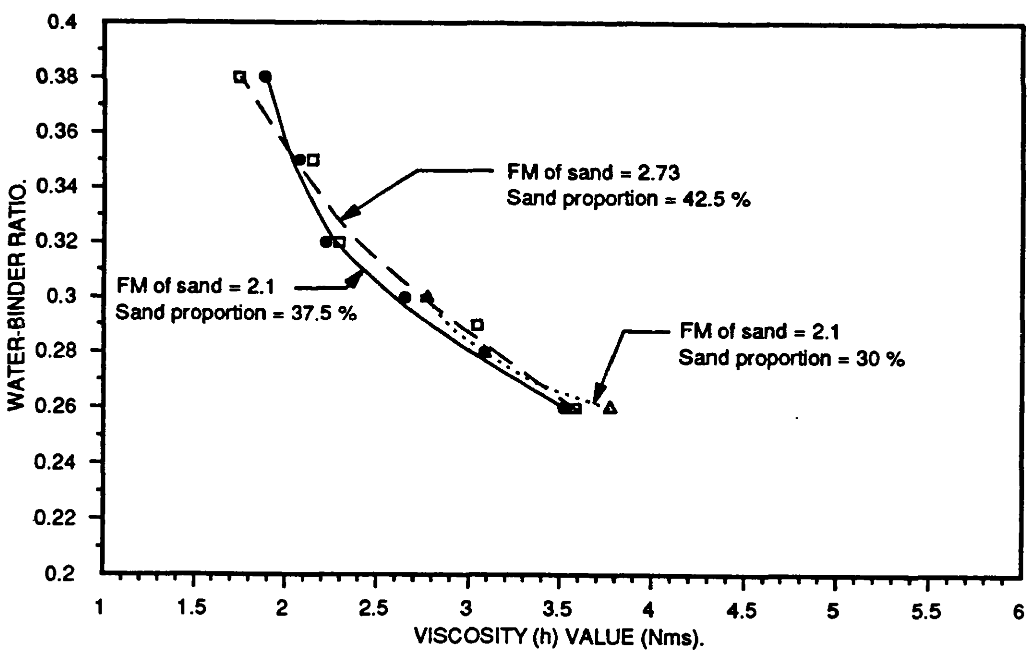
As has been mentioned in Section 3.2.5, the use of a coarse sand with an FM of about 3.0 has been recommended in reducing the "stickiness" of the high strength superplasticised concrete mixes. High coarse-fine aggregate ratios have also been recommended for this reason, (see Section 3.3.2.2, Part III). Mixes with a coarser sand, i.e. FM of 2.73 compared to 2.1 used previously, and a lower fine sand content, i.e. 30 per cent compared to 37.5 per cent, were also studied. The mixes with the coarser sand were also optimised according to the "Modified Maximum Density Theory". Since this theory considers both the void content and the surface area of all the aggregate particles, optimum mix proportions for the coarse sand mixes resulted in a higher coarse sand content; the coarse sand mixes had a 42.5 per cent sand proportion to the total aggregate content while the fine sand mixes had a 37.5 per cent sand proportion, (see Chapter 6).

The coarse sand mixes showed only slightly lower yield values (g) and almost identical plastic viscosity (h) values to those of the fine sand mixes, as shown in Figure 7.11. This similarity of yield (g) and plastic viscosity (h) values, is a further indication that the "Modified Maximum Density Theory" takes into account the quantitative effect the surface area of all the aggregate particles has on paste requirement and, therefore, results in optimum mix proportions.

Mixes with the lower fine sand content, i.e. 30 per cent compared to 37.5 per cent, showed a great reduction in the yield value (g) but again showed slightly higher plastic viscosity (h) values, as shown in Figure 7.11. It seems that lower sand contents do not reduce the "stickiness" of the mixes, (see Table 3.23), and that the term "sticky" consistency has, in this case, wrongly been related to the yield (g) value rather than to the plastic viscosity (h) value.



(a) Yield (g) value.



(b) Plastic viscosity (h) value.

Figure 7.11: The effect of the fineness modulus and proportion of sand on the yield (g) and plastic viscosity (h) values for OPC concrete mixes with a slump of approximately 150 mm.

(III) The effect of the type of superplasticiser.

In order to investigate the effect of the type of superplasticiser on the yield (g) and plastic viscosity (h) values of concrete with a slump of 150 mm, it was first necessary to determine the Conplast M1 dosage required for water-binder ratios between 0.38 and 0.20. Details of the mixes used for this series of tests are shown in Table 7.9. Those tested using the two point apparatus are shown in Table 7.7.

The Conplast M1 was surprisingly found to be somewhat more efficient than the Conplast 430, as shown in Figure 7.12. It has also been found with Conplast M1, as with Conplast 430, that above a certain superplasticiser dosage CSF has a water reducing effect. The higher efficiency of Conplast M1 is in contradiction to two other investigators' suggestions(20,132) that sulphonated naphthalene formaldehyde condensates are more efficient than those with a melamine base, (see Section 3.2.3, Part III). Others(31,32) however have suggested that the efficiency of superplasticisers can be variable depending on the admixture-binder interaction. It appears from Figure 7.12 that the efficiency of the superplasticisers may also depend on the water-binder ratio. A decreasing water-binder ratio requires a higher superplasticiser dosage, and the relative efficiencies appear to vary at different superplasticiser dosages. At superplasticiser dosages of 0.5 and 2.3 per cent solids by weight of cement the Conplast M1 and Conplast 430 are equally efficient. At dosages between 0.5 and 2.3 per cent the Conplast M1 appears to be slightly more efficient, but as has been shown in Chapter 6, it is less efficient at higher dosages than 2.3 per cent.

However, despite equal slumps obtained using the two superplasticisers the force required for the tamping rod to go through the concrete during the slump test was noticeably higher for mixes with Conplast M1. Also, when the cone of the slump test apparatus was lifted the concrete with conplast M1 slumped at a much lower rate than the corresponding mixes with Conplast 430. These differences were shown by the two point test results, both yield (g) and plastic viscosity (h) values being increased, as shown in Figure 7.13.

MIX No. :	1.	2.	3.	4.	5.	6.	7.	8.
Cement content (kg/m³)	421	460	510	525	379	414	459	491
CSF content (kg/m³)					42	46	51	55
Level of cement replacement (%)					10	10	10	10
Total binder content (kg/m³)	421	460	510	525	421	460	510	546
Coarse aggregate[(1)] (kg/m³)	1115	1115	1115	1115	1115	1115	1115	1115
Fine aggregate[(2)] (kg/m³)	670	670	670	670	670	670	670	670
Free water (kg/m³)	160	147	133	126	160	147	133	120
Total water (kg/m³)	184	171	157	150	184	171	157	144
Free water-binder ratio	0.38	0.32	0.26	0.24	0.38	0.32	0.26	0.22
Superplasticiser dosage[(3)] (%)	0.50	0.65	1.15	2.30	0.6	0.75	0.95	1.8
Slump (mm)	155	140	150	155	155	145	155	140

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1.
(3) Dosage is expressed as a percentage of Conplast M1 solids by weight of total binder.

Table 7.9: Mixes tested to determine the relation between water-binder ratio and superplasticiser (Conplast M1) dosage required for concretes of 150 mm slump.

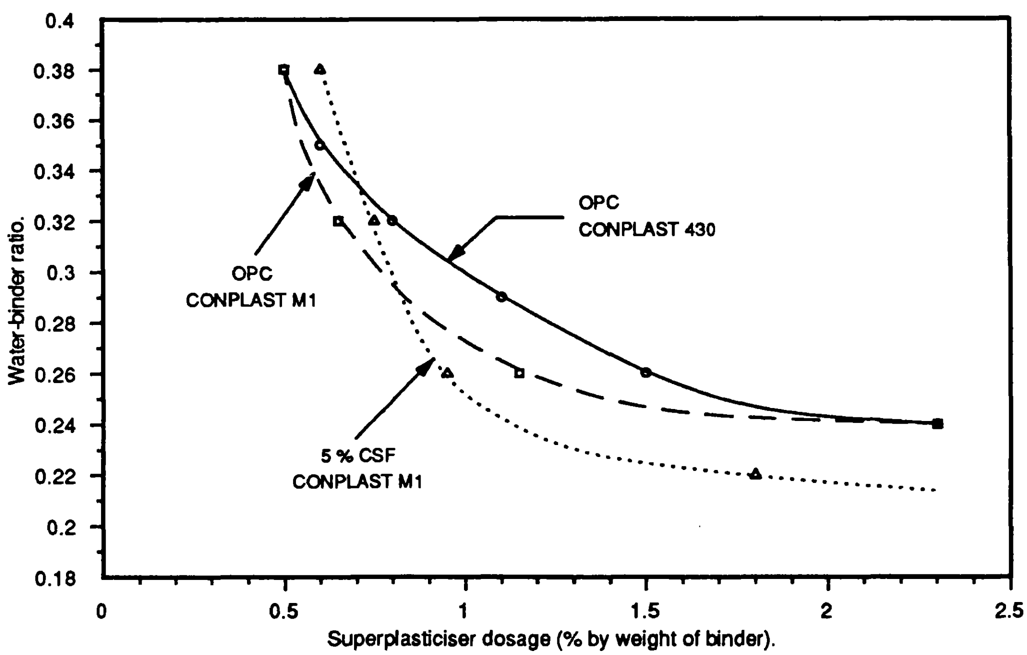
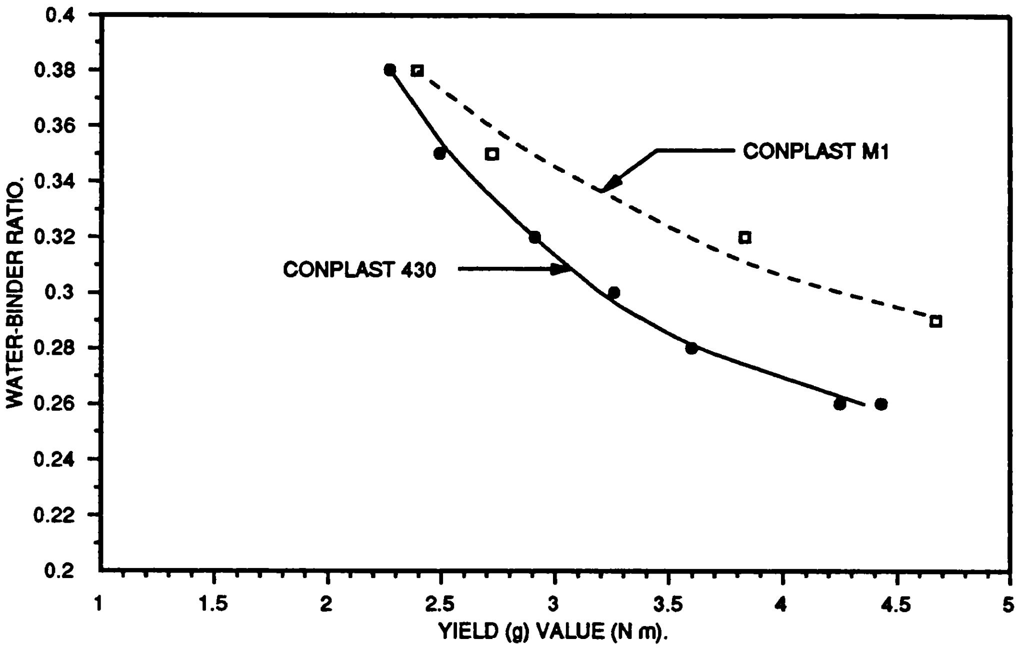
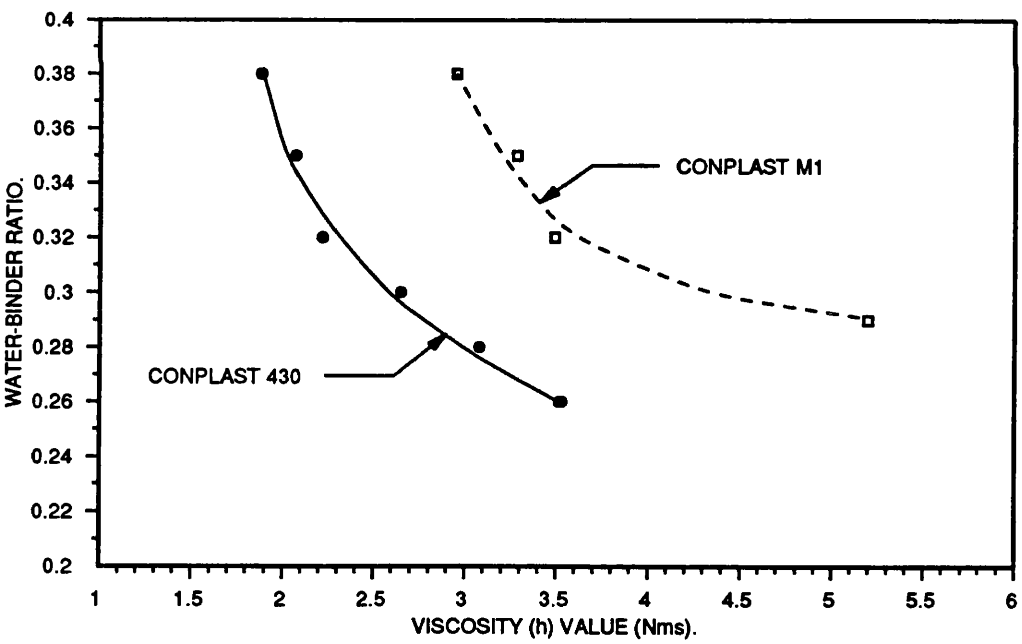


Figure 7.12: The effect of the superplasticiser type on the dosage required for concrete mixes with a slump of 150 mm.



(a) Yield (g) value.



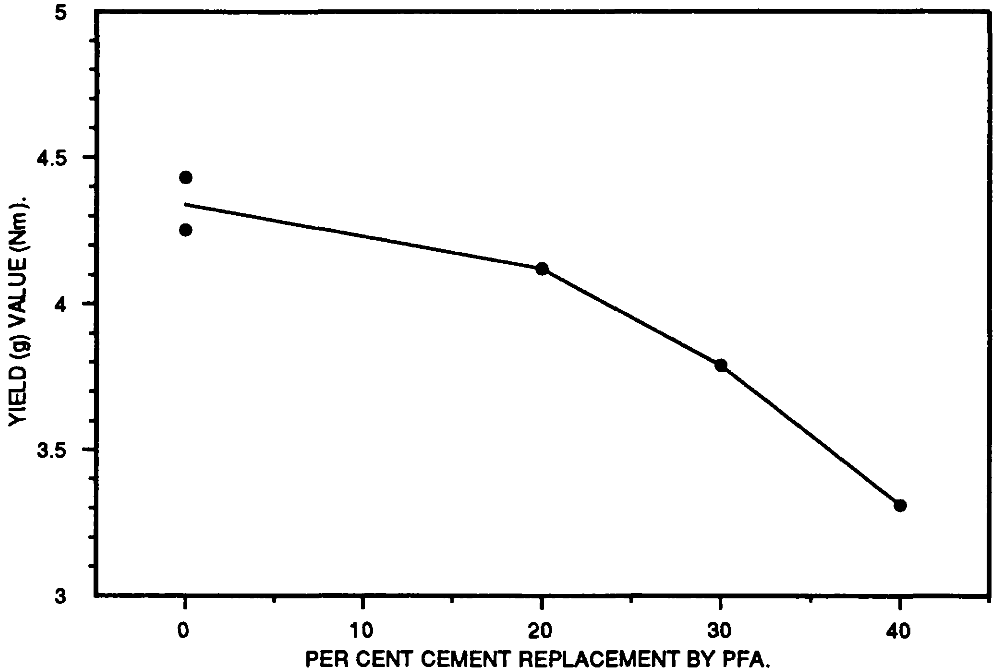
(b) Plastic viscosity (h) value.

Figure 7.13: The effect of the superplasticiser type on the yield (g) and plastic viscosity (h) values for OPC concrete mixes with a slump of approximately 150 mm.

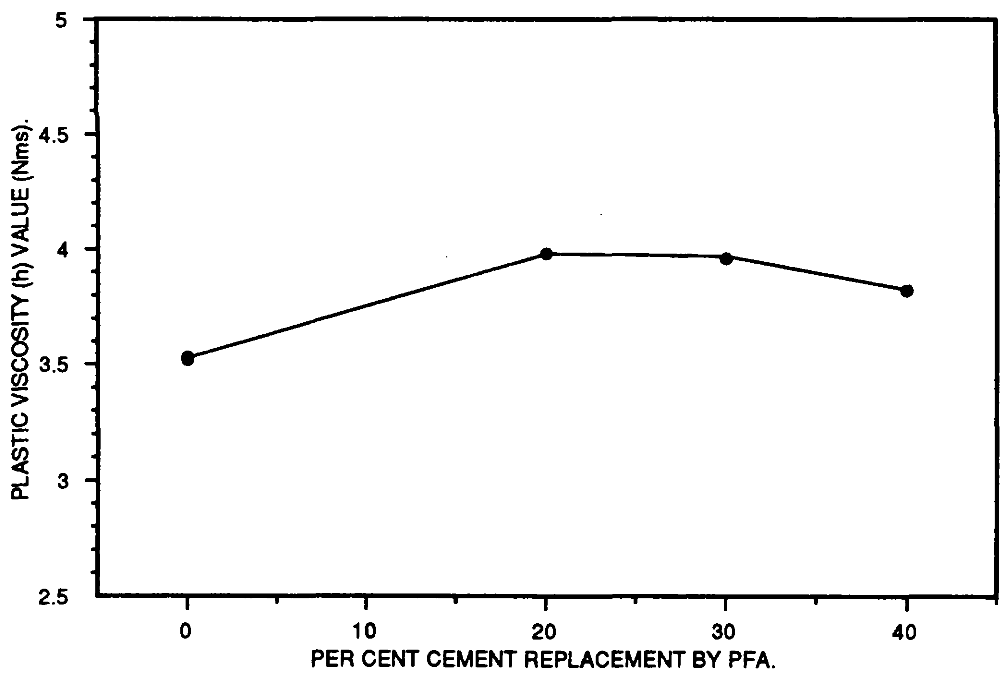
(IV) The effect of partial cement replacement by mineral admixtures.

Partial cement replacement by PFA at levels of 20, 30 and 40 per cent have been found to reduce the yield value of concrete at the low water-binder ratio of 0.26, as shown in Figure 7.14. The partial replacement of cement by PFA on an equal mass basis has the effect of increasing the volume of the paste. Owens(151), (see Section 3.2.6, Part I), has attributed the improved workability of PFA concrete to the increased proportion of the finer material (<45 μm). The increased volume of the paste is believed to have separated the aggregate particles even further and the increased proportion of the finer cementitious material (<45 μm) is believed to have improved the lubrication of the aggregate particles and has, therefore, resulted in lower yield (g) values even at constant slump. The yield (g) values of concrete at water-binder ratios of 0.38 and 0.32 were not, however, reduced, as shown in Figure 7.15a. At the low water-binder ratios the paste that separates the aggregate particles has a high solids content, and having optimised the mix proportions using the "Modified Maximum Density Theory" the amount of this paste has been kept at the minimum required for the lubrication of the aggregate particles. The beneficial effect of the increased volume of the paste and the increased proportion of the finer cementitious binder are, therefore, believed to have a more pronounced effect in reducing the yield (g) value of lower water-binder ratios where the paste used is a very dense one. A similar trend has also been found by Ivanhov and Zacharieva(153) using a ball immersed into vibrated concrete, (see page 115).

The plastic viscosity (h) value has been found to be higher than that of OPC concrete especially at the higher water-binder ratios, as shown in Figure 7.15b. Banfill(154), as mentioned in Section 3.2.6, Part I, showed that despite the higher slump values and therefore lower yield values obtained by increased cement replacement by PFA the plastic viscosity was, unchanged within experimental error. Consequently, if the concrete has the same slump it is expected to have a higher plastic viscosity. This has been found especially for the higher water-binder ratios used in my programme. It is believed that this increase is at least partly due to the change of the rheological properties of the paste, that have also been described by Banfill, (see page 119). However, this is only one of the factors that may affect the plastic viscosity (h) value of concrete mixes. The increased

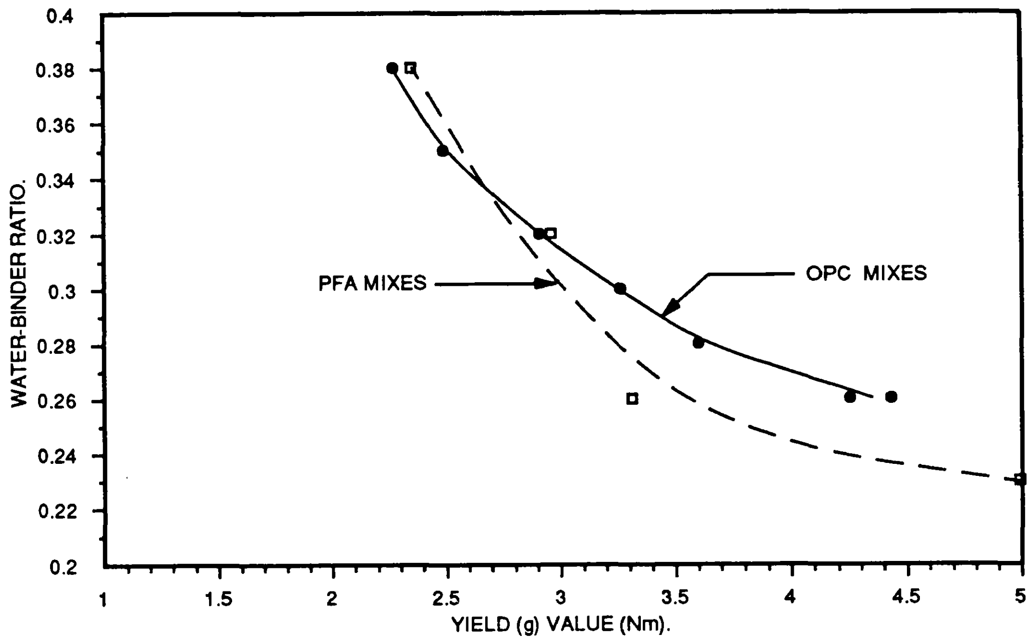


(a) Yield (g) value.

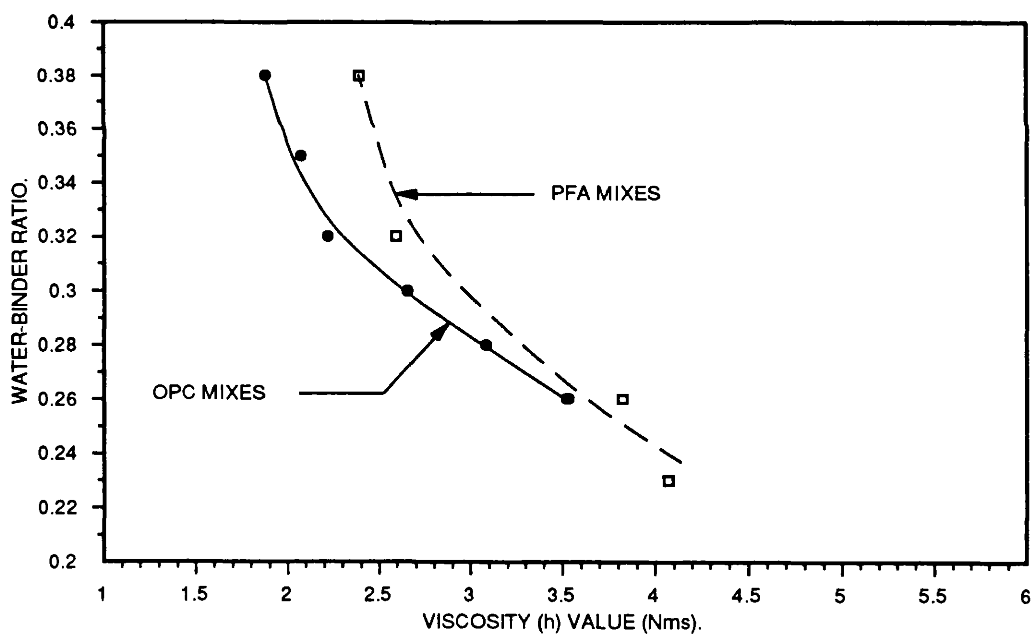


(b) Plastic viscosity (h) value.

Figure 7.14: The effect of the level of cement replacement by PFA on the yield (g) and plastic viscosity (h) values for concrete mixes with water-binder ratio of 0.26 and a slump of approximately 150 mm.



(a) Yield (g) value.



(b) Plastic viscosity (h) value.

Figure 7.15: The effect of 40 per cent cement replacement by PFA on the yield (g) and plastic viscosity (h) values for concrete mixes with a slump of approximately 150 mm.

volume of the paste and the increased proportion of the finer (<45 μm) cementitious binder may also affect the rheological properties of concrete. These are believed to have contributed to reducing the difference in the plastic viscosity between OPC and PFA mixes at the lower water-binder ratios. As with their effect on the yield (g) value, they appear to have a more pronounced effect on the plastic viscosity (h) value at the lower water-binder ratios, where the paste has a higher solids content.

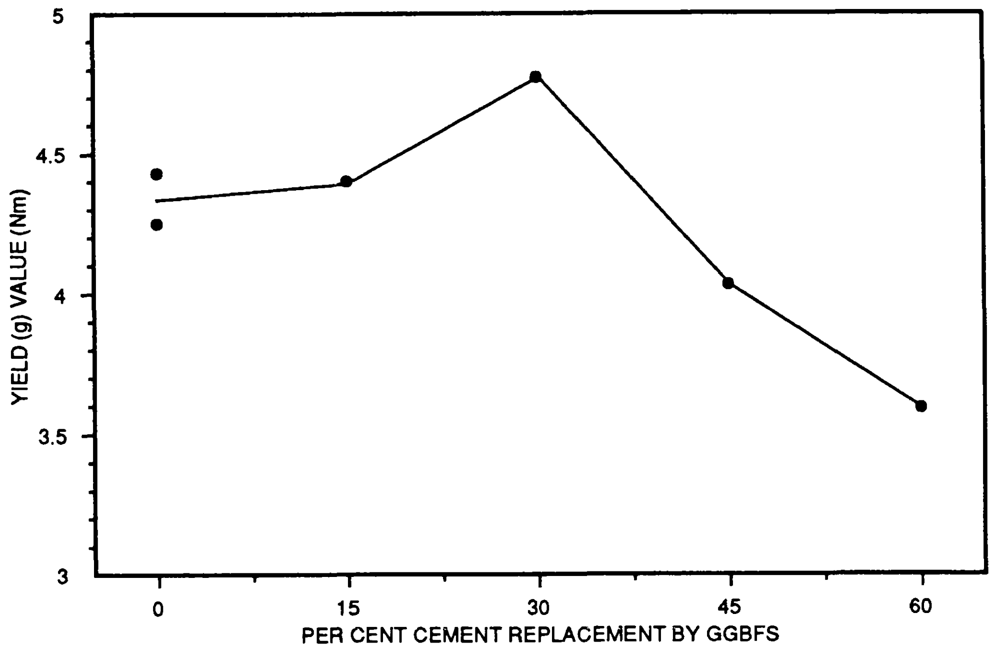
The results of mixes with partial cement replacement by GGBFS have been inconclusive as to whether GGBFS increases or reduces the yield (g) and plastic viscosity (h) values of concretes with equal slumps, as shown in Figure 7.16. The results seem to depend a lot on small variations of the slump. It may therefore be concluded that GGBFS has only a very small effect on the rheological properties of concrete.

The effect of partial cement replacement by CSF on concrete with a slump of 150 mm has also been studied. The CSF concrete appeared to be richer in cementitious content than OPC concrete despite the replacement being on an equal mass basis. This difference in appearance is the result of the different properties of CSF, which as mentioned in Section 3.2.6, Part III, are:

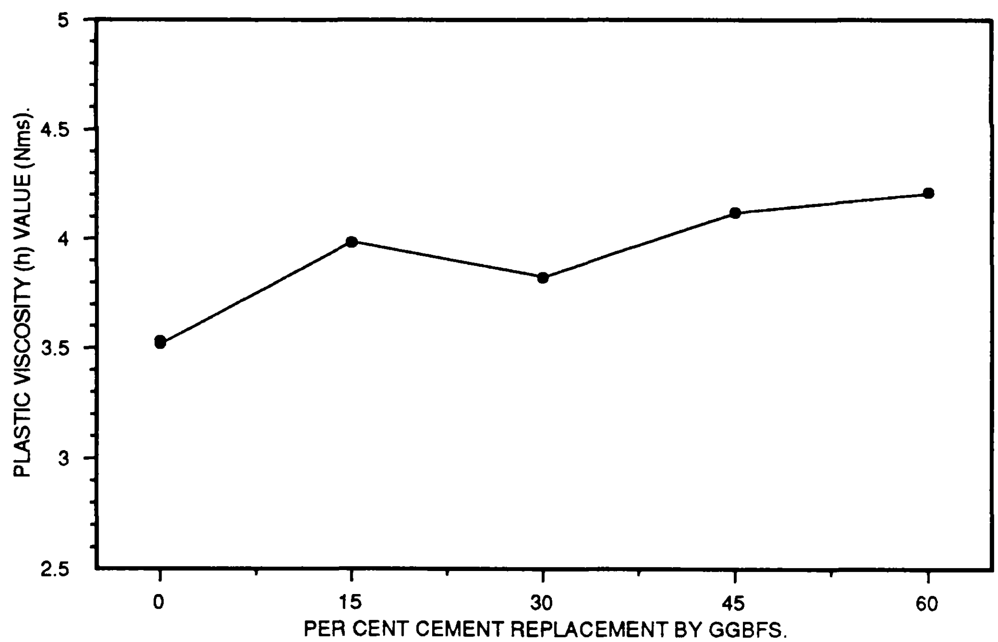
(i) the specific gravity of CSF is about 2.2 compared to 3.1 for ordinary Portland cement,

and, (ii) the specific area of CSF is approximately 20,000 m^2/kg, as compared to 432 m^2/kg for OPC, and therefore CSF has a lower average particle; approximately 100 times smaller than the average cement particle.

The force required for the tamping rod to go through the CSF concrete during the slump test was lower than OPC concrete despite the equal slumps of 150 mm. Also, when the cone of the slump test apparatus was lifted the OPC concrete slumped at a much lower rate than the corresponding CSF concrete. These differences were shown by the two point test results, both yield (g) and plastic viscosity (h) values for CSF concretes being lower than those with neat OPC, as shown in Figures 7.17 and 7.18. This is in agreement with Tachibana et al's work(138), (see Section 3.2.6, Part III), which showed that the electric energy consumed during mixing decreased by incorporating CSF in the high strength

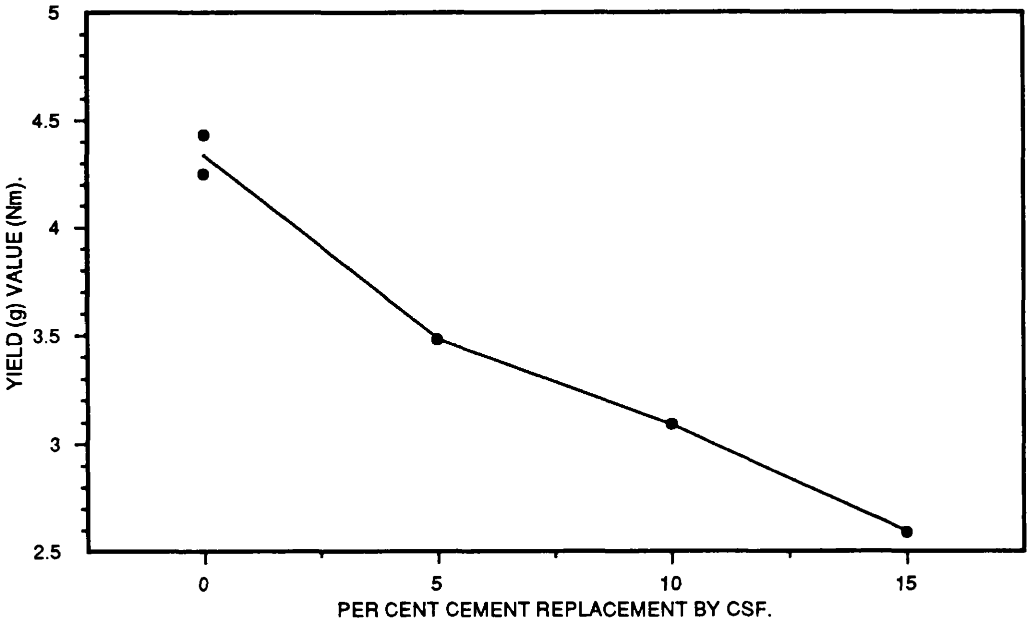


(a) Yield (g) value.

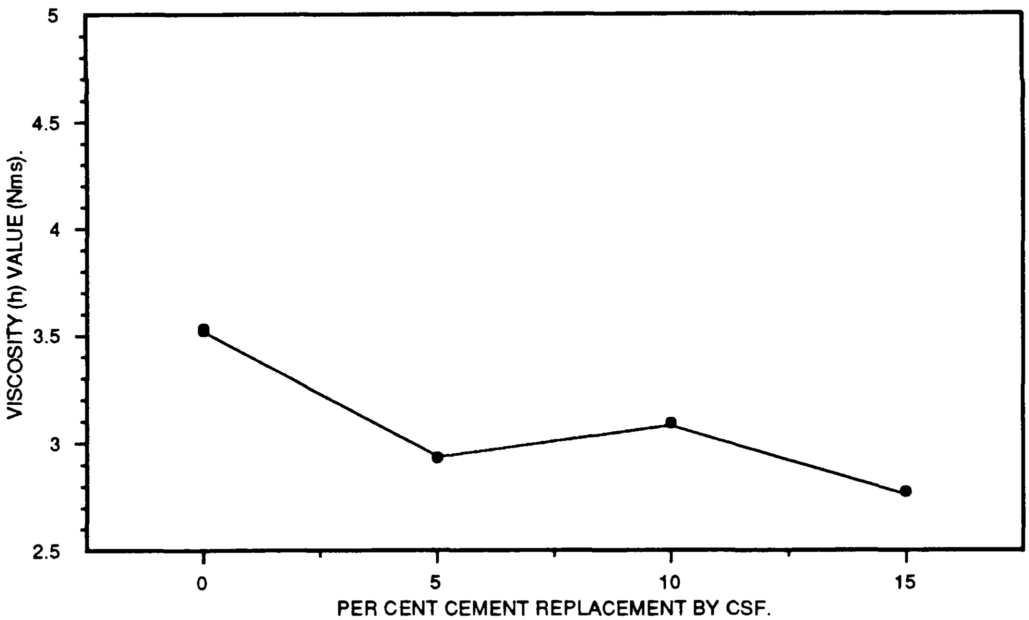


(b) Plastic viscosity (h) value.

Figure 7.16: The effect of the level of cement replacement by GGBFS on the yield (g) and plastic viscosity (h) values for concrete mixes with water-binder ratio of 0.26 and a slump of approximately 150 mm.

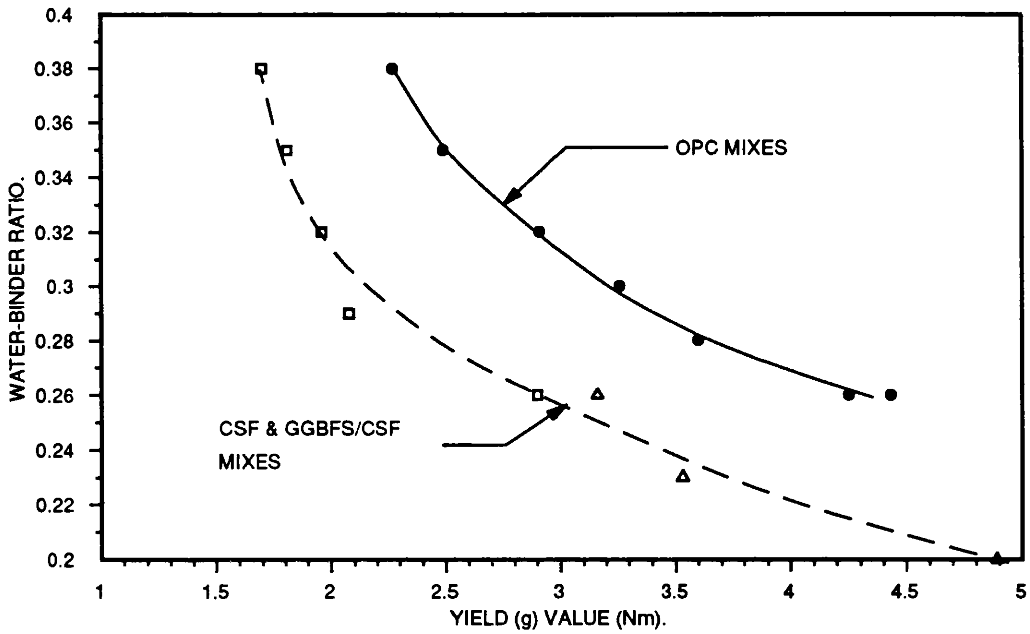


(a) Yield (g) value.

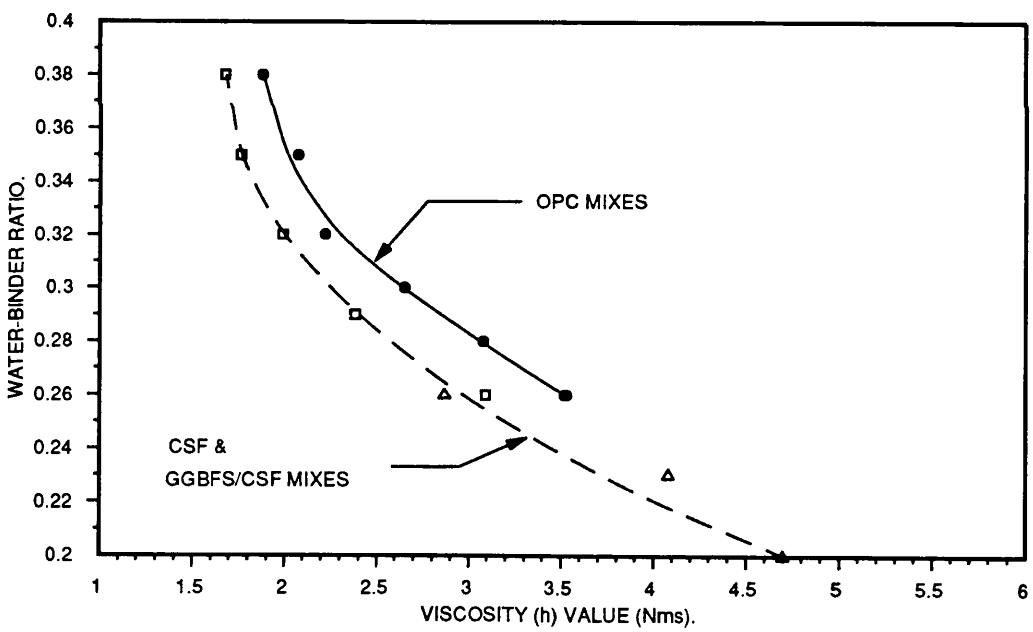


(b) Plastic viscosity (h) value.

Figure 7.17: The effect of the level of cement replacement by CSF on the yield (g) and plastic viscosity (h) values for concrete mixes with water-binder ratio of 0.26 and a slump of approximately 150 mm.



(a) Yield (g) value.



(b) Plastic viscosity (h) value.

Figure 7.18: The effect of 10 per cent cement replacement by CSF on the yield (g) and plastic viscosity (h) values for concrete mixes with water-binder ratio of 0.26 and a slump of approximately 150 mm.

concrete mixes, despite these having similar slumps after 3.5 minutes of mixing. They concluded that workability and constructibility can be improved by incorporating CSF.

Since GGBFS has been found not to affect significantly the behaviour of fresh concrete, mixes with a combination of 50 per cent GGBFS and 10 per cent CSF have been tested to extend the relation between water-binder ratio with yield (g) and plastic viscosity (h) values down to a water-binder ratio of 0.20, (Figure 7.18). As the water-binder ratio was lowered the yield (g) and plastic viscosity (h) values increased, as for OPC concrete, despite the concrete having an equal slump of 150 mm. Helland[80] also found, as mentioned in Section 2.3.3, that in spite of similar slumps a mix with an extremely high dosage of superplasticiser, i.e. low water-binder ratio, formed much greater internal resistance than a non-superplasticised mix, i.e. high water-binder ratio. The non-superplasticised concrete was found to respond well to the poker vibrator, while the mix with an extremely high dosage of superplasticiser reacted badly. The problem of the higher viscosity was solved by adding an air-entraining admixture; the plastic viscosity (h) value then decreased to that of a normal water-based CSF concrete. The plastic viscosity (h) value of CSF concrete has, therefore, been shown by Helland to be directly related to how concrete will behave at the shear rates of high frequency poker vibrators. This cannot be reflected by the slump value which reflects the situation when the rate of shear is near zero.

The lower plastic viscosity (h) value of CSF concrete as compared to OPC concrete does not, however, seem to correspond to a better behaviour of the CSF concrete to vibration. Casting of cubes required a higher amplitude of vibration for CSF than for OPC concrete. Sellevold and Nilsen[177] have, as mentioned in Section 3.2.6, Part III, also stated that the "static" and "dynamic" behaviour of normal strength CSF concrete do not relate in the same way as for OPC concrete, i.e. the slump measure does not predict the response to vibration in the usual way. For practical purposes they recommended that the slump should be increased by 20 to 30 mm for a CSF concrete to obtain the same workability as that for normal concrete. Because of this behaviour of CSF concrete workers have described CSF concrete as "sticky" or "plastic".

The above behaviour of CSF concrete can best be explained using Ritchie's subdivisions of the rheology of fresh concrete, i.e. stability, compactibility, and mobility, as shown in Figure 7.1[243]. The report by ACI Committee 309[81] on the "Behaviour of Fresh Concrete During Vibration" described Tattersall's two point test as a single test for the determination of the "mobility characteristics" of fresh concrete. They also stated that although Ritchie's diagram points out the primary factors embraced by the term "rheology", it does not show any relationship between categories. For example, viscosity, cohesion, and the angle of internal resistance may affect mixture stability and compactibility.

Helland's results, (see page 41), have shown that there is a relationship between mobility, i.e. yield (g) and plastic viscosity (h) value, and compactibility. This relationship was however determined for a particular binder and it does not seem to be valid when concretes with different binders are compared, i.e. an OPC concrete having higher yield (g) and plastic viscosity (h) values than CSF concrete responds better to vibration.

It is worth noting here that the term plastic viscosity (h) value does not correspond to the "viscosity" as has been described by Ritchie[243], but corresponds to the "internal resistance to shear". Ritchie refers to the viscosity of the matrix, i.e. the paste, and not that of concrete. He claims that it is important to be able to measure the viscosity of the paste fraction of the mixture in order to achieve a better understanding of a mixture's flow characteristics. Ivanhov and Zacharieva[153] have also stated that: "The flow properties of concretes are associated with those of the interstitial paste which contains the cement. This is especially applied to fresh concrete under vibration". However, Banfill[154] has, as mentioned in Section 3.2.6, Part III, shown that it is not only the changing rheology of the paste that affects the yield (g) value of concrete, but also the increased volume of paste, which reduces interparticle contact. He showed that the change in the rheology of the paste was the main factor affecting the yield (g) value of a rich concrete mix (mixes C and D, shown in Table 3.7) while it was the combination of the increased volume and changing rheology of the paste that affected the yield (g) value of a lean mix (mix A, shown in Table 3.7). In these mixes the plastic viscosity (h) value remained constant which was consistent with the plastic viscosity of the paste, shown in Table 3.6. Mix B, which he did not comment on, yields useful information. While the plastic viscosity of the paste remains unchanged with increasing PFA replacement of cement, concrete mix B shows a decrease in the plastic viscosity (g) value. This mix differs from the others in that it has a low cement content in combination with a high sand content. It is suggested that because of the high surface area of the aggregate in mix B, the main factor that affected the plastic viscosity (h) value of the concrete was the increased volume of the paste, which reduced interparticle contact, and not the rheology of the paste. My experiments on PFA concrete have also shown, as mentioned earlier, that the rheology of the paste has a more pronounced effect on the plastic viscosity (h) value at the higher water-binder ratios. Its effect is reduced at the lower water-binder ratios when the effect of the increased volume of paste and the increased proportion of the finer (<45 μm) cementitious material becomes more pronounced.

According to ACI Committee 309(81) (italics indicate my additions) the internal resistance of concrete to shear depends on:

1. the shape and texture of the aggregate, *and also the total surface area of the aggregate,*
2. the richness of the mixture, *i.e. not only the cementitious content but also its specific area,*
3. the water-cement ratio, *or water-binder ratio,*

and, 4. the type of cement (*or binder*) used, *i.e. the rheological properties of the paste.*

It may therefore be concluded that mobility, as defined by the yield (g) and plastic viscosity (h) values, may be influenced more by the richness of the mixture while compactibility, i.e. the response of fresh concrete to vibration, may be infuenced more by the rheological properties of the paste. The possibility of combinations of the above factors that affect the internal resistance of concrete to shear will require large populations of test data before definitive relationships between mobility and compactibility are established.

7.7 CONCLUSIONS.

Partial cement replacement by PFA or GGBFS results in an improvement of the workability. Advantage can therefore be taken of this to either reduce the superplasticiser dosage or reduce the water-binder ratio used in a concrete and yet to maintain the same workability as OPC concrete.

The use of CSF in concrete reduces the workability at low superplasticiser dosages and therefore requires a further addition of superplasticiser to retain the same slump. However, above a certain superplasticiser dosage, believed to be that required to reduce the cohesion of CSF particles sufficiently to disperse them, CSF has a water-reducing effect.

Having determined the effect of PFA, GGBFS, and CSF on the workability of concrete, their effectiveness in reducing the adiabatic temperature rise was assessed and this is described in the next chapter.

CHAPTER 8.
ADIABATIC TEMPERATURE RISE DURING HYDRATION.

8.1 INTRODUCTION.

Temperature considerations, described in Section 2.3.4, require the development of mixtures which will give a minimum of heat generation but still have the required strength at the specified age of testing. Prerequisite to this is the measurement of:

(i) either the temperature rise in a simulated large pour, isothermal heat output, or the adiabatic temperature rise of concretes,

and, (ii) their strength development.

The data from the adiabatic temperature rise measurements can be used to predict the temperature cycling the concrete will undergo in actual structural members. It is hoped that the results of the adiabatic tests described in this chapter will, in the near future, provide the basis for determining the relation between the compressive strengths of test specimens subjected to the predicted temperature cycling with those cured under standard temperature conditions. The compressive strength development of the concretes is more conveniently included in the next chapter.

The experimental procedure for the measurement of the adiabatic temperature rise is first described followed by the results of tests to measure the effects of such parameters as:

(i) water-cement ratio,

(ii) casting temperature,

(iii) level of cement replacement by PFA, GGBFS and CSF,

and, (iv) high levels of replacement by PFA or GGBFS in combination with 10 per cent CSF.

8.2 EXPERIMENTAL PROCEDURE.

8.2.1 Preparation of samples.

The mixing procedure for concrete described in Chapter 5 was adopted. Where fresh concrete temperatures of 10^0C after 5 minutes of mixing were required, all the materials were kept for 24 hours at 7^0C in a climatic cabinet. The same procedure was adopted for mixes cast at 30^0C except that the temperature of the climatic cabinet was set at 30^0C. The materials used for concrete cast at 20^0C were taken directly from the storing bins which were kept at room temperature.

Fresh concrete was, after the mixing time of 5 minutes, compacted in a plywood mould, shown in Figure 8.1, whose internal dimensions were 210 x 210 x 210 mm. Two of the mixes, one with 10 per cent PFA and the other with 10 per cent GGBFS, were cast during the hot weather of summer and the temperature after 5 minutes of mixing was found to be about 35^0C. The concrete was therefore kept longer in the pan of the mixer and the outside of the pan was hosed with tap water until the concrete temperature fell to about 30^0C.

The time taken to compact the concrete in the mould and then place it in the climatic cabinet, where it was to be tested, was usually 15 minutes, i.e. a total of 20 minutes elapsed since the contact of water with the binder. The temperature of the fresh concrete cast at 10^0C was found to increase very quickly after mixing, and the time taken for compacting and placing the concrete into the climatic cabinet was therefore shortened to 5 minutes.

8.2.2 Adiabatic temperature measurements.

The adiabatic temperature rise due to hydration of cement is the temperature rise which will occur if fresh concrete is stored in a perfectly insulated environment, i.e. one from which no heat loss can occur. To achieve this state it is necessary to either heavily

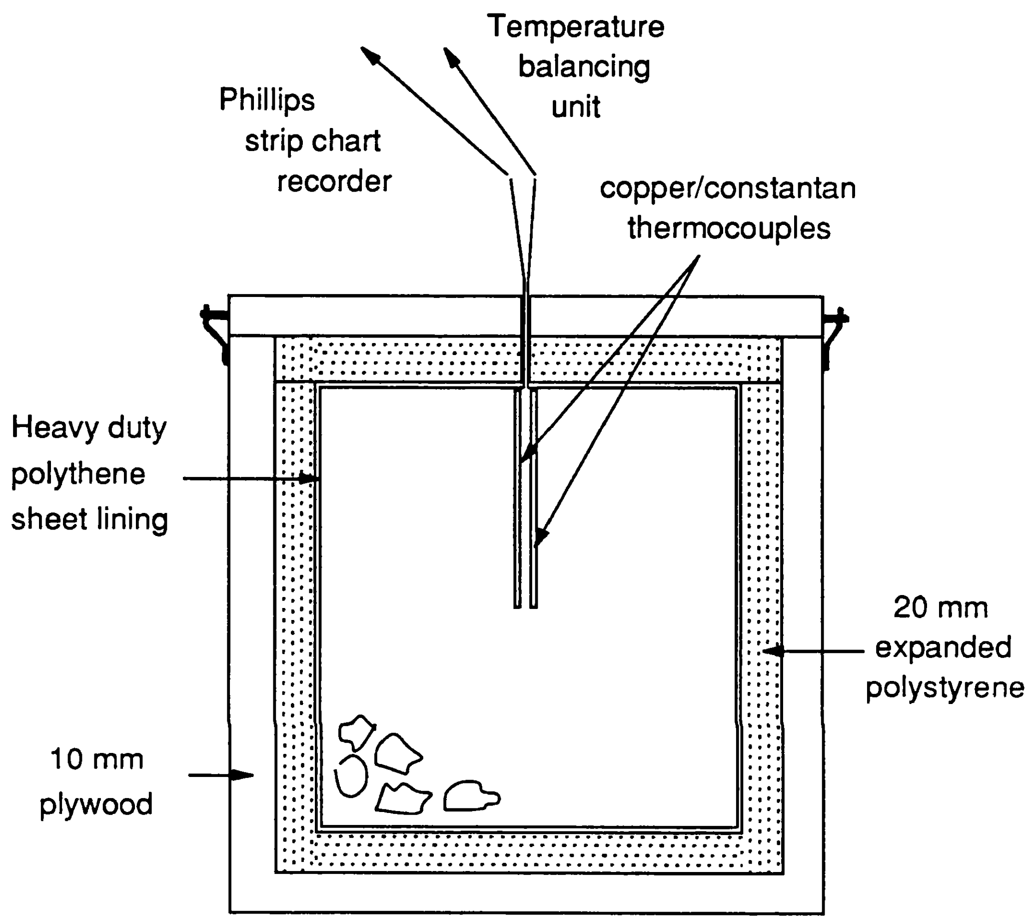


Figure 8.1: Plywood mould used in adiabatic tests.

insulate the concrete or alternatively to ensure that the environment in which the concrete is stored is at the same, or nearly the same, temperature as the concrete. The latter approach was adopted in my research programme.

Concrete was cast, as described above, in a plywood mould lined with 20 mm expanded polystyrene for insulation and heavy duty polythene to prevent moisture loss, as shown in Figure 8.1. The specimen was then placed in a Fisons climatic cabinet, and two copper/constantan thermocouples were inserted in it through a hole in the top of the mould. The first one was connected to a temperature balancing unit that was incorporated in the climatic cabinet, while the second was connected to a Phillips strip chart recorder. Two more copper/constantan thermocouples were used to monitor the temperature of the oven, and were again connected the same way as the other two. The balancing unit monitored the temperature difference between the oven and the concrete. It was set to activate the oven when the difference in the thermocouple output was 0.002 mV, until thermal compatibility had been restored. The actual temperature difference as determined from the strip charts was 1^0C. Some heat loss did therefore occur. The rate of temperature drop, occurring after the maximum temperature rise had been reached, was found to be 0.08^0C per hour, (an average of the first ten experiments), and was used to correct the adiabatic temperature rises. A more detailed investigation of the rate of heat loss, e.g., heating specimens to 80^0C for several days to complete the hydration and then monitoring the rate of temperature drop until the temperature reached the ambient, was felt unnecessary. Bamforth(93), despite having carried such a detailed investigation, only used a first order correction. This he advocated was justified because the heat loss was small particularly during the initial few days after casting when a large proportion of the hydration occurred.

8.3 RESULTS AND DISCUSSION.

The details of all the mixes whose adiabatic temperatures have been measured are shown in Table 8.1.

Mix No. :	1.	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.
Cement (kg/m^3)	439	510								275	184
PFA (kg/m^3)			51	102	204					184	
GGBFS (kg/m^3)						51	153	306			275
CSF (kg/m^3)									51	51	51
Level of cement replacement (%)			PFA=10	PFA=20	PFA=40	GGBFS=10	GGBFS=30	GGBFS=60	CSF=10	PFA=36 CSF=10	GGBFS=54 CSF=10
Total binder (kg/m^3)	439	510	510	510	510	510	510	510	510	510	510
Coarse aggregate (kg/m^3)	1095	1115	1115	1115	1115	1115	1115	1115	1115	1115	1115
Fine aggregate (kg/m^3)	655	670	670	670	670	670	670	670	670	670	670
Free water (kg/m^3)	167	133	133	133	133	133	133	133	133	133	133
Total water (kg/m^3)	190	157	157	157	157	157	157	157	157	157	157
Free water-binder ratio	0.38	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26
Superplasticiser dosage[(1)]	0.5	1.5	1.5	1.4	1.2	1.5	1.5	1.5	1.5	1.0	1.0
	Casting Temperature, (°C).										
Series A		11.2	11.8	10.2	12.6	11.6	10.2	10.8	11.8	11.8	10.2
Series B	21.8	21.8	23.8	21.9	22.5	19.0	24.5	19.8	23.0	21.5	21.8
Series C		30.8[(2)]	31.2[(3)]	30.5	32.8	30.2[(4)]	29.9	30.2	32.5	30.1	30.2

(1) The dosage is given as a percentage of the superplasticiser solids by weight of cementitious material.

(2) Repeated at mixing temperature of 38.8°C but cooled down to 30.1°C before casting, and it is referred to as Test 2.

(3) This specimen reached a mixing temperature of approximately 35°C and was cooled down before casting, (referred to as Test 1). The test was repeated with a mixing and casting temperature of 30.1°C, (referred to as Test 2).

(4) This specimen reached a mixing temperature of approximately 35°C and was cooled down before casting, (referred to as Test 1). The test was repeated with a mixing and casting temperature of 30.8°C, (referred to as Test 2).

Table 8.1: The details of the mixes used for adiabatic temperature rise measurements.

8.3.1 The combined effect of water-cement ratio and cement content.

The adiabatic temperature rise of OPC mixes with water-cement ratios of 0.38 and 0.26 and with cement contents of 439 and 510 kg/m^3, respectively, are shown in Figure 8.2. The difference was only 1.8^{0}C despite the cement content being higher by 71 kg/m^3 for the water-cement ratio of 0.26. The adiabatic temperature rises for 100 kg of cement are 10.8^{0}C and 9.6^{0}C for water-cement ratios of 0.38 and 0.26 respectively. These values are also much lower than the values obtained by Bamforth(93) for similar cement contents but higher water-cement ratios, as shown in Figure 8.3. Under an adiabatic condition all the heat of hydration will be transformed into an increase of the temperature of the concrete sample. The heat evolution per kg of cement (Q_a) can then be calculated from:

$$Q_a = \Delta T c \rho / C$$

where:	ΔT	is the maximum increase of temperature of the concrete,
	c	is the specific heat of the concrete,
	ρ	is the unit weight of the concrete,
and	C	is the cement content of the concrete.

The specific heat was not measured but the value found by Bamforth(93) and Aarsleff et al(82), i.e. c = 1.1 kJ/kg/^{0}C, was used. The heat evolutions per kg of cement for the mixes used in my research programme as well as values calculated from Bamforth's adiabatic temperature rise experiments are shown in Table 8.2. It can be seen that the values for the mixes used in my research programme indicate that the water-cement ratio has an influence on the heat evolution (Q_a). This is in agreement with the results by Helland(195), (see Figure 3.68), Parrott(16,32) and Aarsleff et al(82), (see Figures 2.9 and 2.10). However, the values calculated from Bamforth's experiments(93), which were all on mixes with water-cement ratios greater than 0.39, do not indicate this. Aarsleff et al(82) concluded, from microscopic examinations of concretes having very low water-cement ratios, that although unhydrated cement compounds exist for all values of the water-cement ratio, they appear to increase with decreasing water-cement ratio, particularly when it is reduced below 0.35. The heat evolution of cement is therefore expected to be greatly affected when the water-cement ratio is reduced below 0.35. This somewhat mitigates the temperature rise of high strength concrete, but these are still high, e.g., 50^{0}C for an OPC mix.

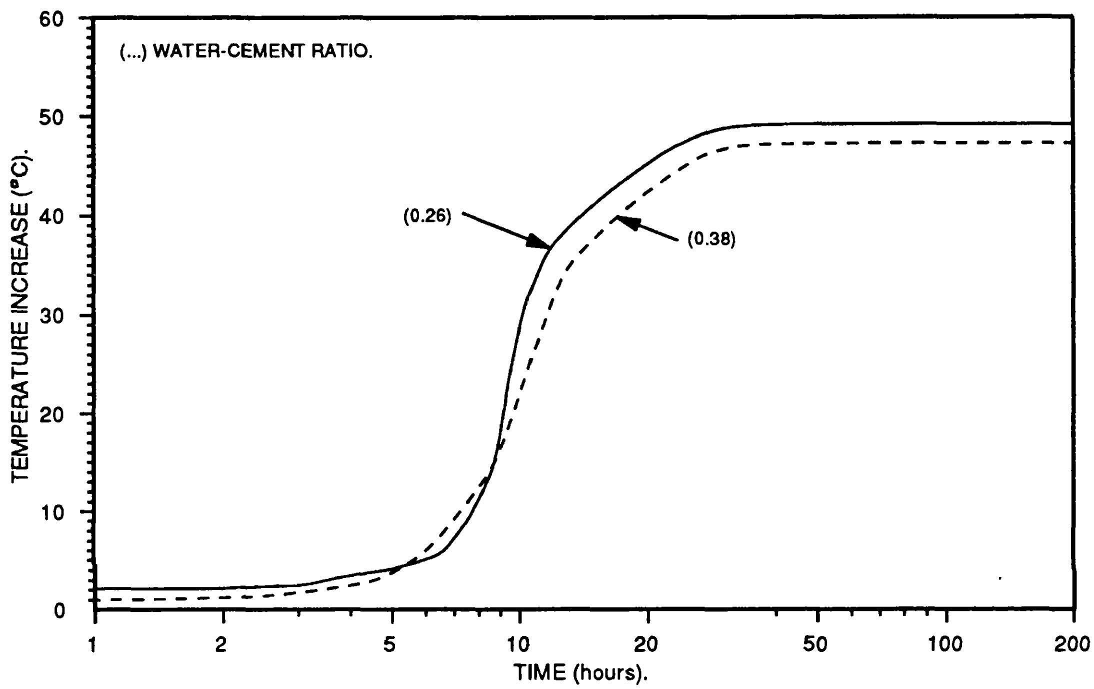
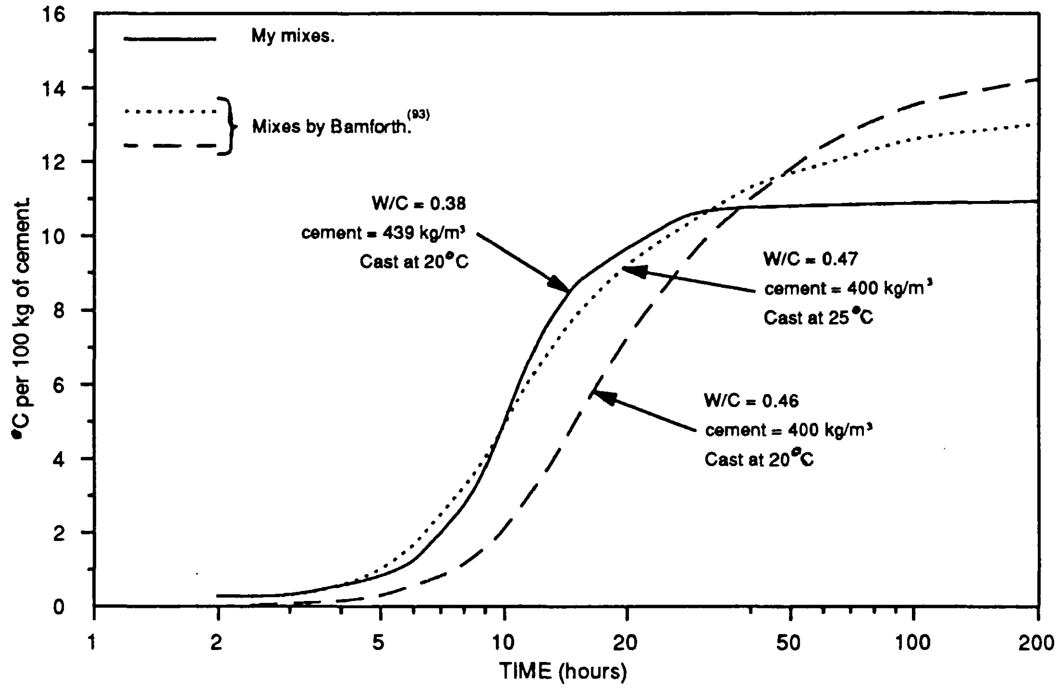
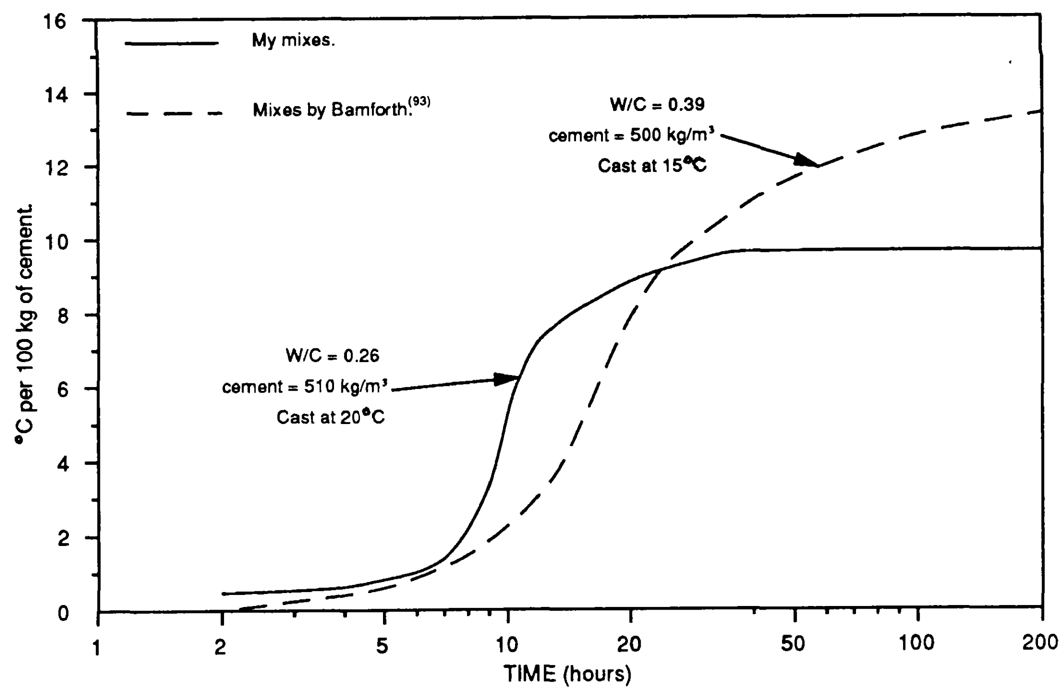


Figure 8.2: Adiabatic temperature rise of OPC mixes.



(a) Total cement content of approximately 400 kg/m³.



(b) Total cement content of approximately 500 kg/m³.

Figure 8.3: Adiabatic temperature rises for 100 kg of cement for various water-cement ratios.

No.	Cement content (kg/m³)	Water-cement ratio	Density (kg/m³)	Casting Temperature (°C)	Max. Temp. Rise (°C)	Q_a (kJ/kg/°C)
1.	439	0.38	2455	21.8	47.4	291.3
2.	510	0.26	2505	21.8	49.2	265.8
3*.	150	1.27	2325	15.5	32.0	281.3
4*.	300	0.64	2360	15.5	56.5	354.8
5*.	400	0.46	2365	16.0	67.0	331.7
6*.	400	0.47	2368	27.0	87.0	390.7
7*.	500	0.39	2360	17.0	82.0	337.5

* Bamforth's adiabatic temperature rise experiments(93).

Table 8.2: The heat evolution per kg of cement for concrete mixes under adiabatic condition.

8.3.2 The effect of the casting temperature.

The adiabatic temperature rises of the OPC mixes with water-cement ratio of 0.26 and nominal casting temperatures of 10, 20 and 30^{0}C are shown in Figure 8.4. The rise is very small during the first few hours after mixing. This period has been referred to, e.g., by Bamforth[93], as the dormant period and has, herein, been quantitatively defined as the time taken for the adiabatic temperature to increase by 10^{0}C. As can be seen from Figure 8.5a, the dormant period is decreased by increasing the casting temperature of the concrete. The peak temperature has herein been defined as the highest temperature rise recorded after which the rate of generation of heat by hydration was lower than the small heat losses occurring during the almost adiabatic test. The peak temperature also decreased with increase in the nominal casting temperature, (Figure 8.5b). Bamforth[93] has attributed this to the higher initial rate of hydration at higher mixing temperatures which may have significantly modified the hydration process, (see page 53).

8.3.3 The effect of partial cement replacement by pulverised fuel ash (PFA).

The effect of partial cement replacement by PFA at levels up to 40 per cent is shown in Figures 8.6, 8.7 and 8.8 for concretes with a water-binder ratio of 0.26 and nominal casting temperatures of 10, 20 and 30^{0}C. At the low levels of cement replacement by PFA, i.e. 10 and 20 per cent, there has, in most cases, been a small increase of the dormant period, (Figure 8.9a), whilst the peak temperatures of the PFA concretes were comparable, or slightly higher than those of the OPC concrete, (Figure 8.9b). This indicates that PFA mixes, under certain conditions, can have a potentially higher level of total heat output during hydration than neat OPC mixes.

The mix with 10 per cent PFA cast at 20^{0}C appeared to have an abnormal adiabatic temperature rise, i.e. it had a shorter dormant period than the OPC concrete and a lower peak temperature rise than the 20 per cent PFA concrete. The casting temperature of the 10 per cent PFA concrete, i.e. 23.8^{0}C, was however slightly higher than those of the OPC mix, i.e. 21.8^{0}C, and of the 20 per cent PFA mix, i.e. 21.9^{0}C. As has been shown in the

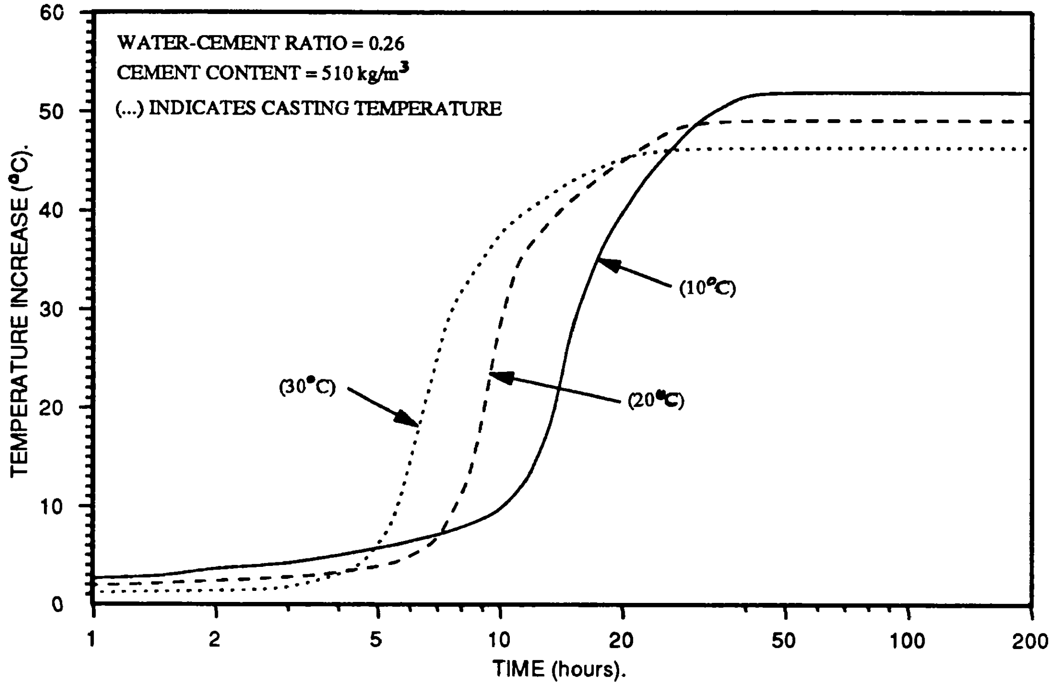
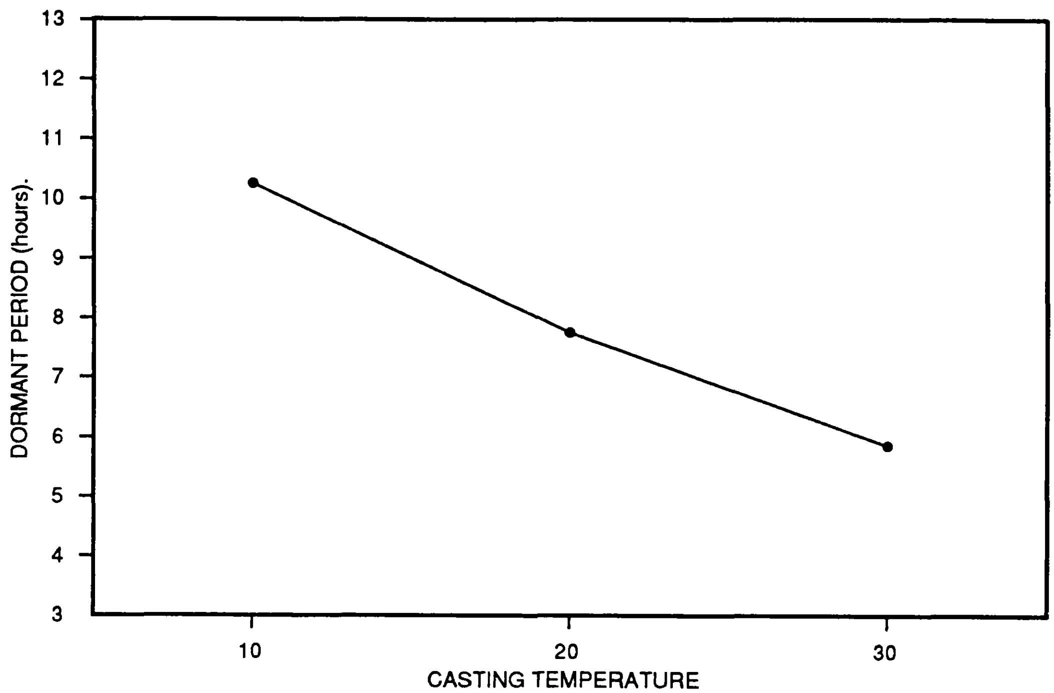
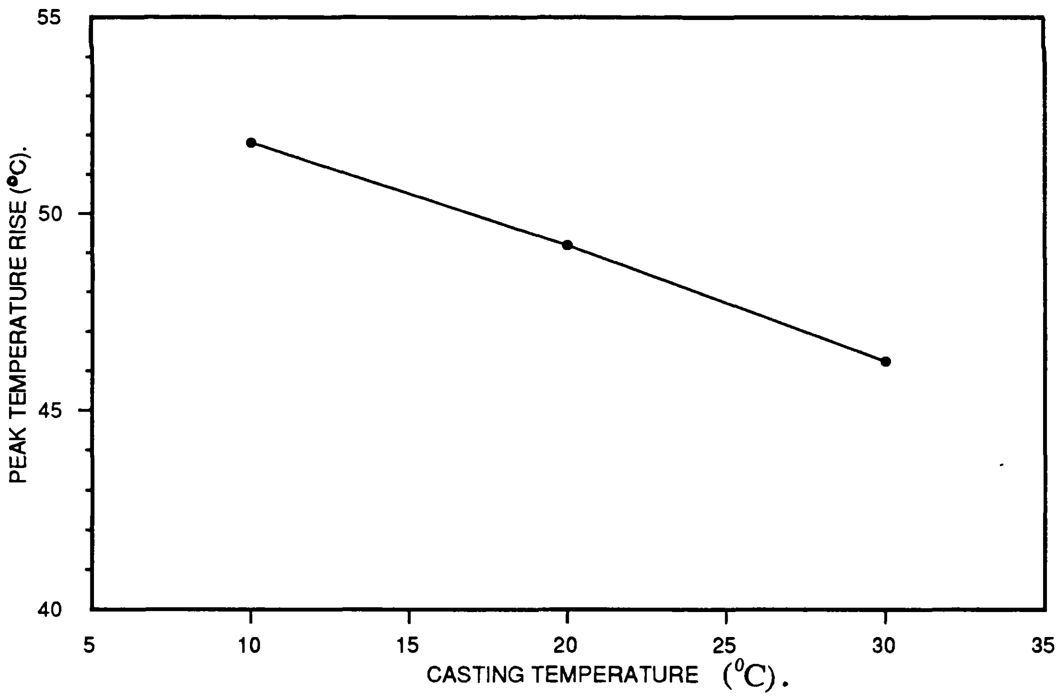


Figure 8.4: The effect of the casting temperature on the adiabatic temperature rise of OPC mixes.



(a) Dormant period.



(b) Peak temperature.

Figure 8.5: The effect of the casting temperature on the dormant period and peak temperature rise of OPC mixes.

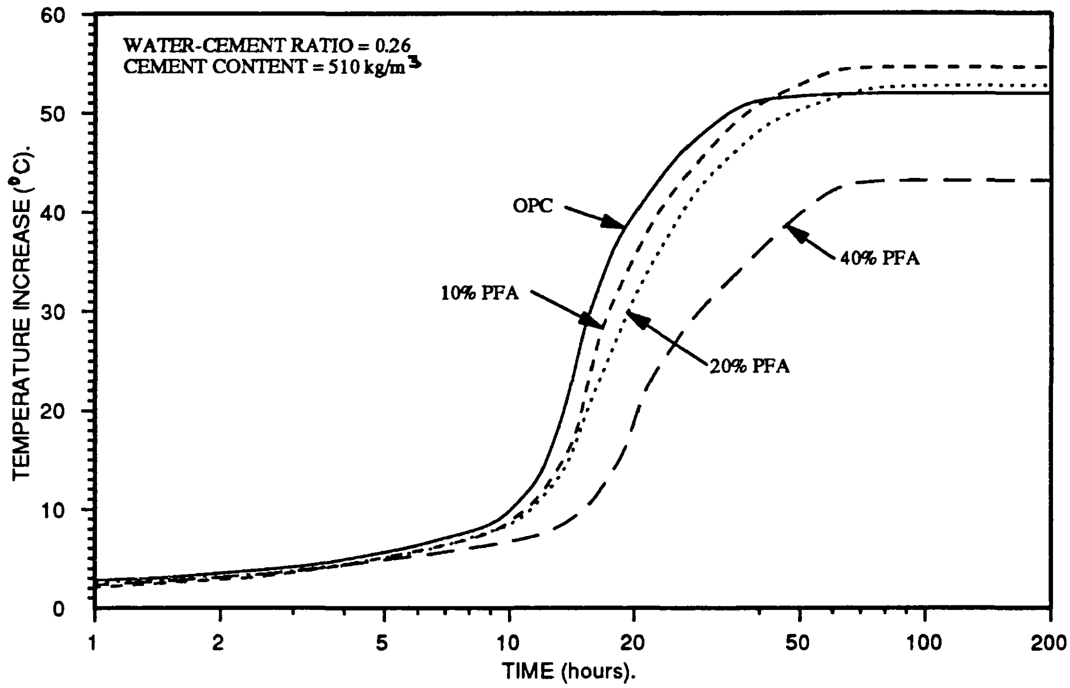


Figure 8.6: The adiabatic temperature rises of PFA mixes cast at 10°C.

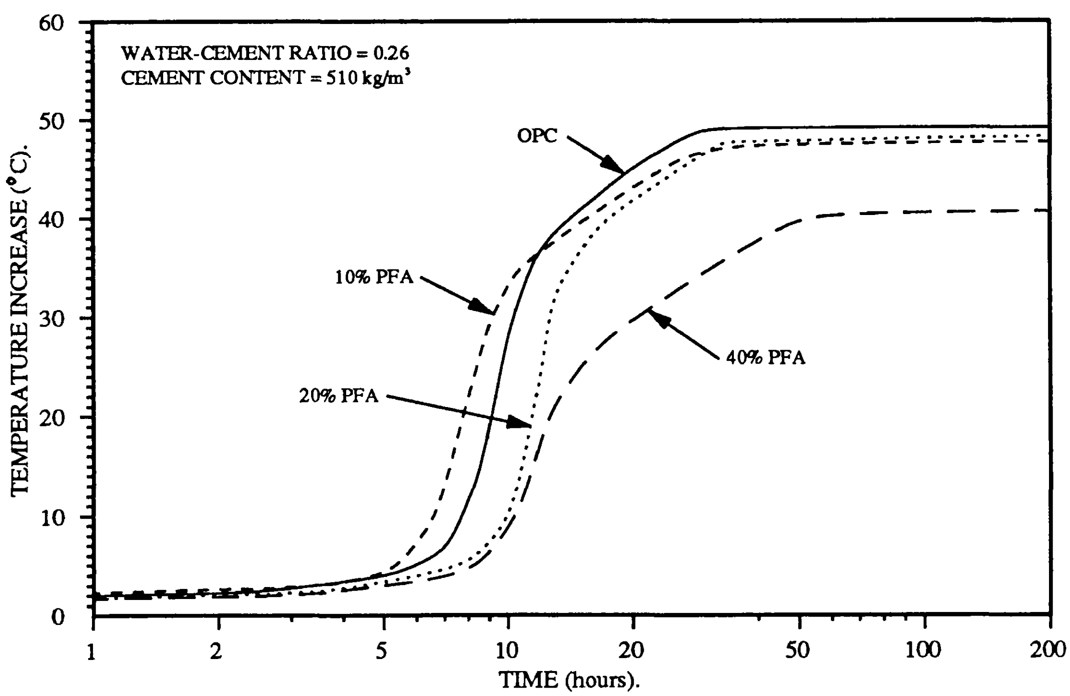


Figure 8.7: The adiabatic temperature rises of PFA mixes cast at 20°C.

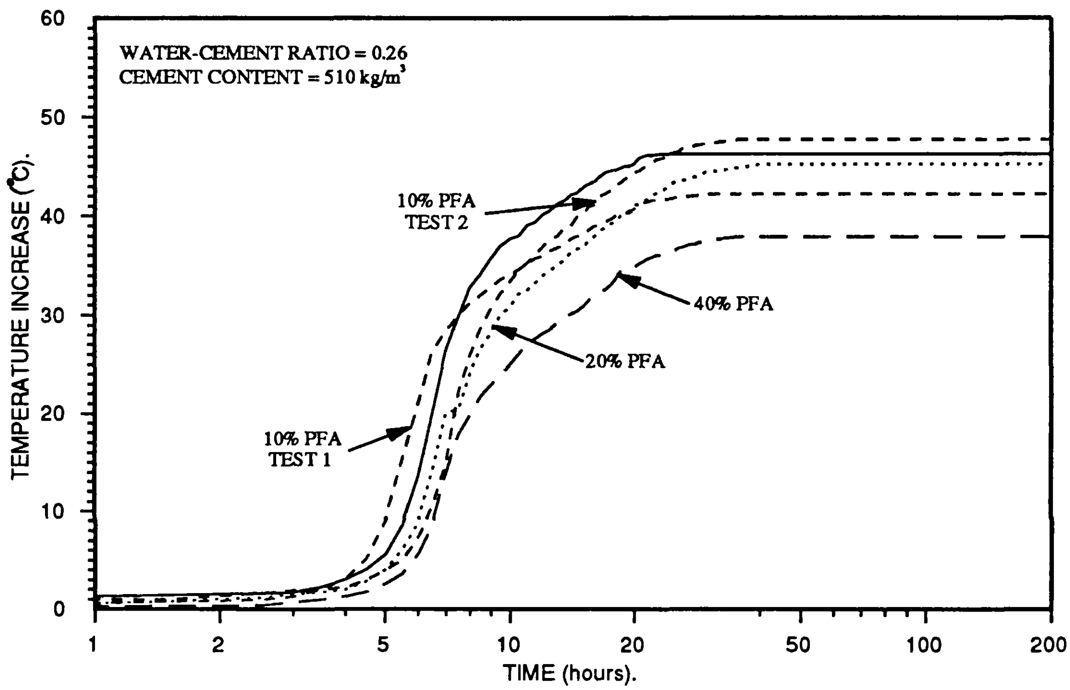
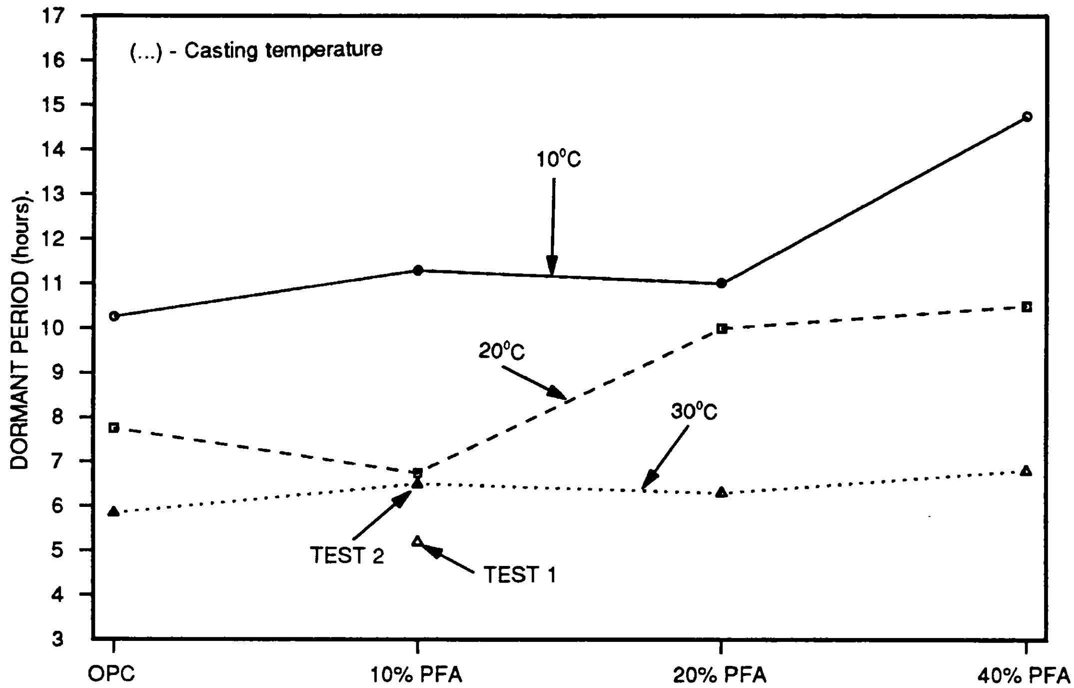
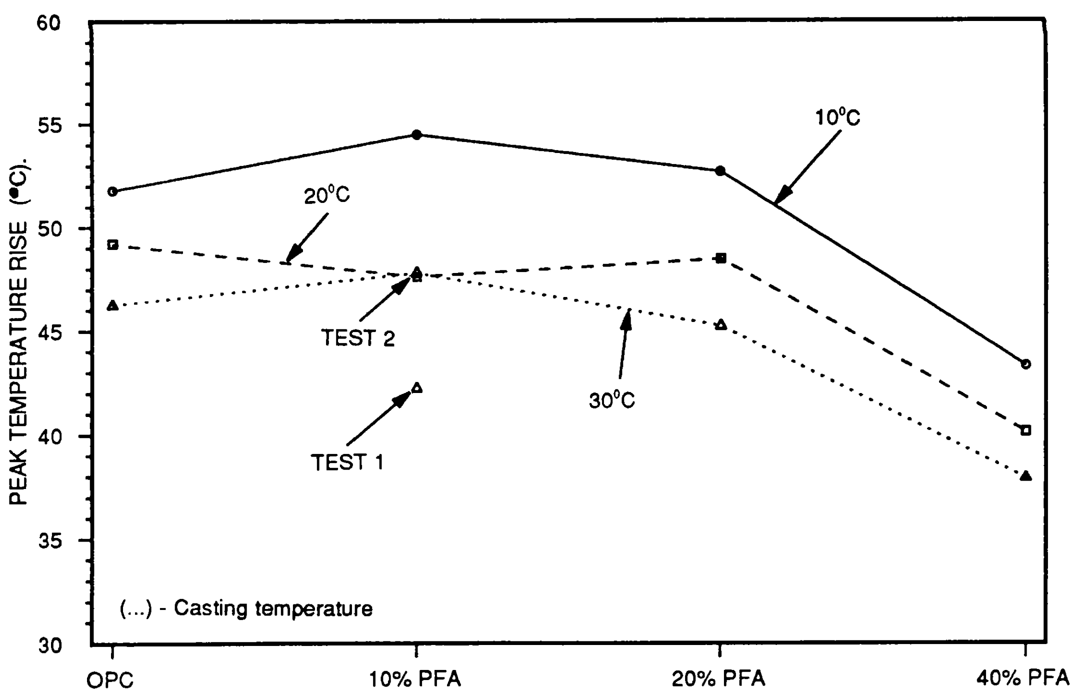


Figure 8.8: The adiabatic temperature rises of PFA mixes cast at 30°C.



(a) Dormant period.



(b) Peak temperature.

Figure 8.9: The effect of the casting temperature on the dormant period and peak temperature rise of PFA mixes.

previous section the effect of a higher mixing temperature is to shorten the dormant period and also decrease the peak adiabatic temperature rise of the concrete.

The shorter dormant period and lower peak temperature have also been found for one of the two tests with 10 per cent PFA concrete cast at the nominal temperature of 30^{0}C, shown in Figure 8.8 as Test 1. The difference in the actual casting temperatures of the OPC and PFA concretes was not significant, i.e. the difference was only 0.4^{0}C. The 10 per cent PFA concrete was one of the two mixes whose temperature after the initial mixing was found to be around 35^{0}C and was therefore cooled down to approximately 30^{0}C, as has been explained in Section 8.2.1, before casting. The test was therefore repeated under more controlled conditions so as to achieve a concrete temperature of as close to 30^{0}C as possible without the need to cool the concrete. The results of the second test, shown in Figure 8.8 as Test 2, conformed with the general trend, i.e. 10 per cent PFA resulted not only in a longer dormant period but also in a higher peak temperature rise than the corresponding OPC concrete. This indicated that the effect of a higher mixing temperature, despite having been lowered before casting, can have a significant effect. This is further discussed in the next section.

At the higher level of cement replacement by PFA of 40 per cent the adiabatic temperature rise was reduced significantly, by 16.4, 18.4 and 18.0 per cent for the nominal casting temperatures of 10, 20 and 30^{0}C, respectively, (Figure 8.9). Bamforth(93) has reported similar values, i.e. with 30 per cent PFA the reduction in the adiabatic temperature rise was 10-15 per cent, increasing to 30 per cent with 50 per cent PFA, (see page 122).

8.3.4 The effect of partial cement replacement by ground granulated blast furnace slag (GGBFS).

The effect of partial cement replacement by GGBFS at levels up to 60 per cent is shown in Figures 8.10, 8.11 and 8.12 for concretes with a water-binder ratio of 0.26 and nominal casting temperatures of 10, 20 and 30^{0}C. The peak temperatures of mixes with low level

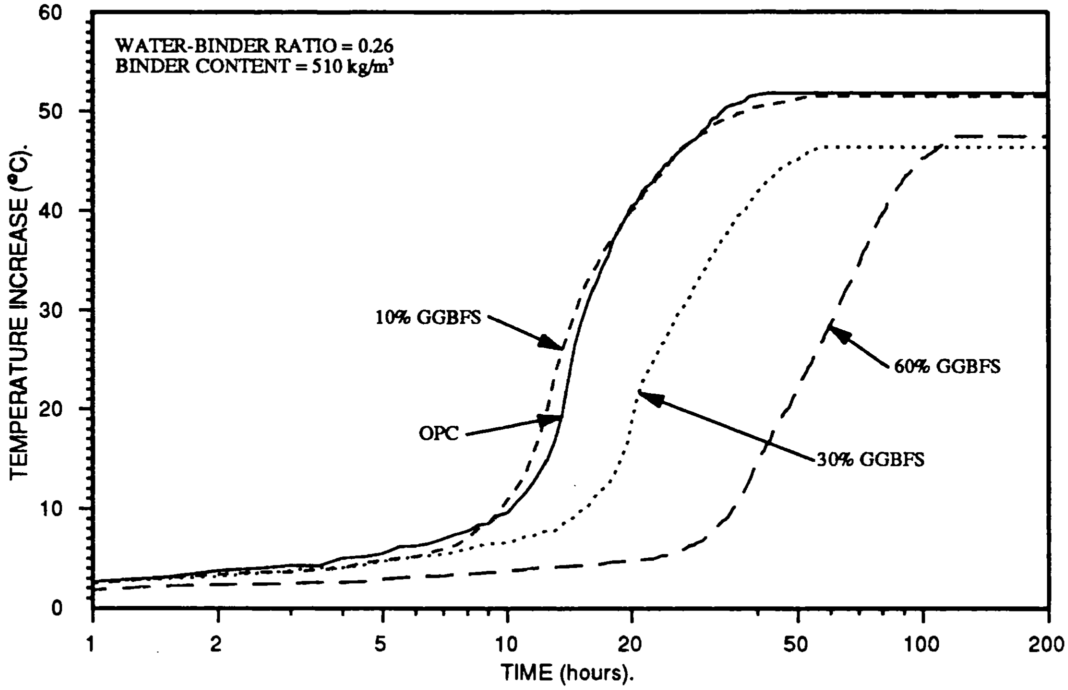


Figure 8.10: The adiabatic temperature rises of GGBFS mixes cast at 10^{0}C.

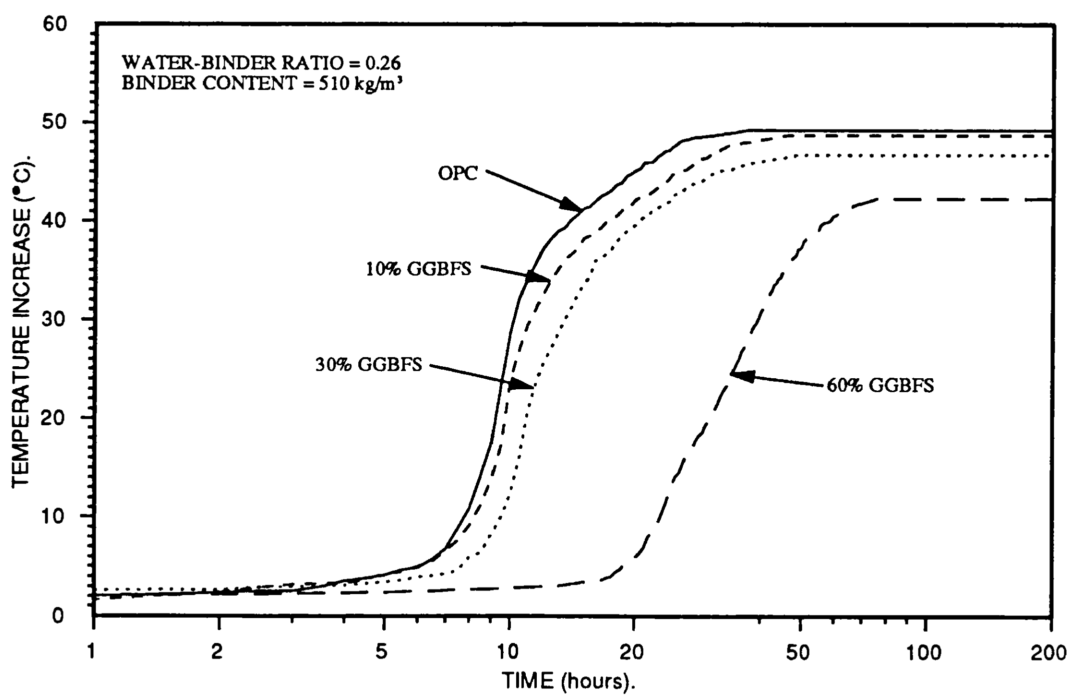


Figure 8.11: The adiabatic temperature rises of GGBFS mixes cast at 20^{0}C.

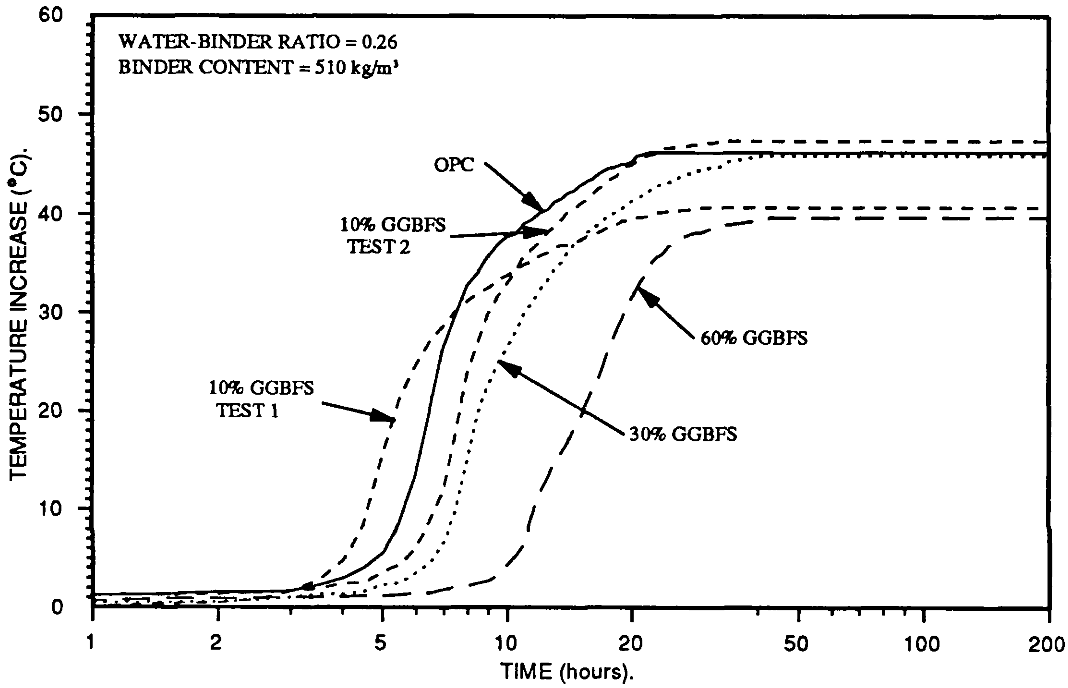


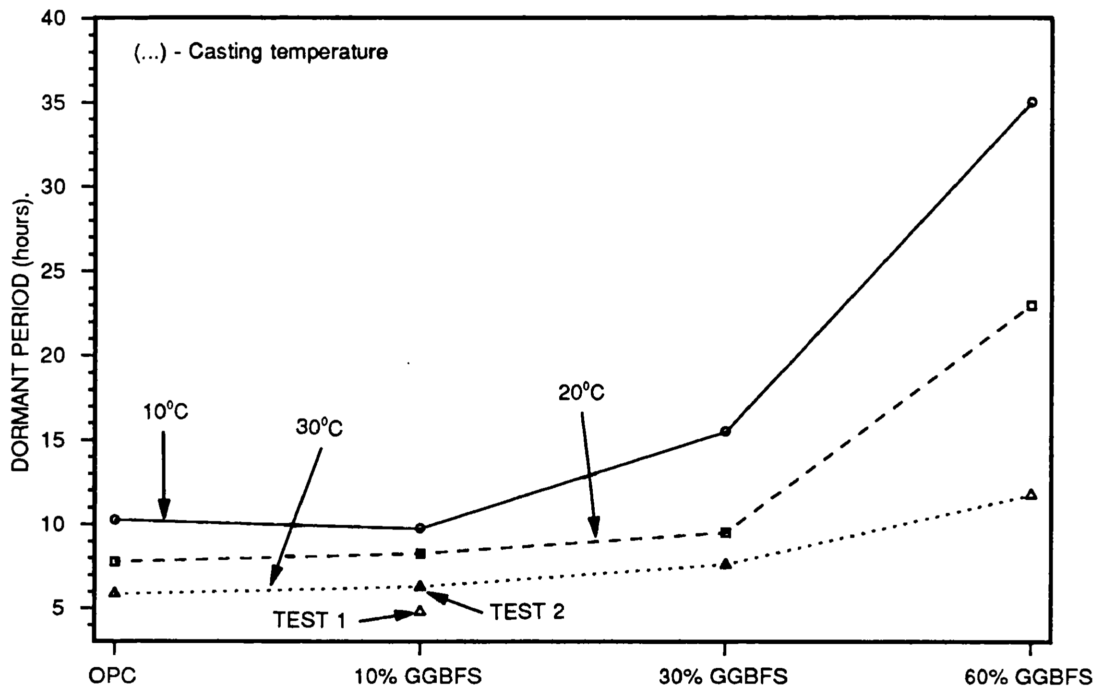
Figure 8.12: The adiabatic temperature rises of GGBFS mixes cast at 30^{0}C.

of cement replacement of 10 per cent, were comparable and even slightly higher than those of the OPC concrete. This indicates that GGBFS, as with PFA, under certain conditions can produce a potentially higher level of total heat of hydration than OPC.

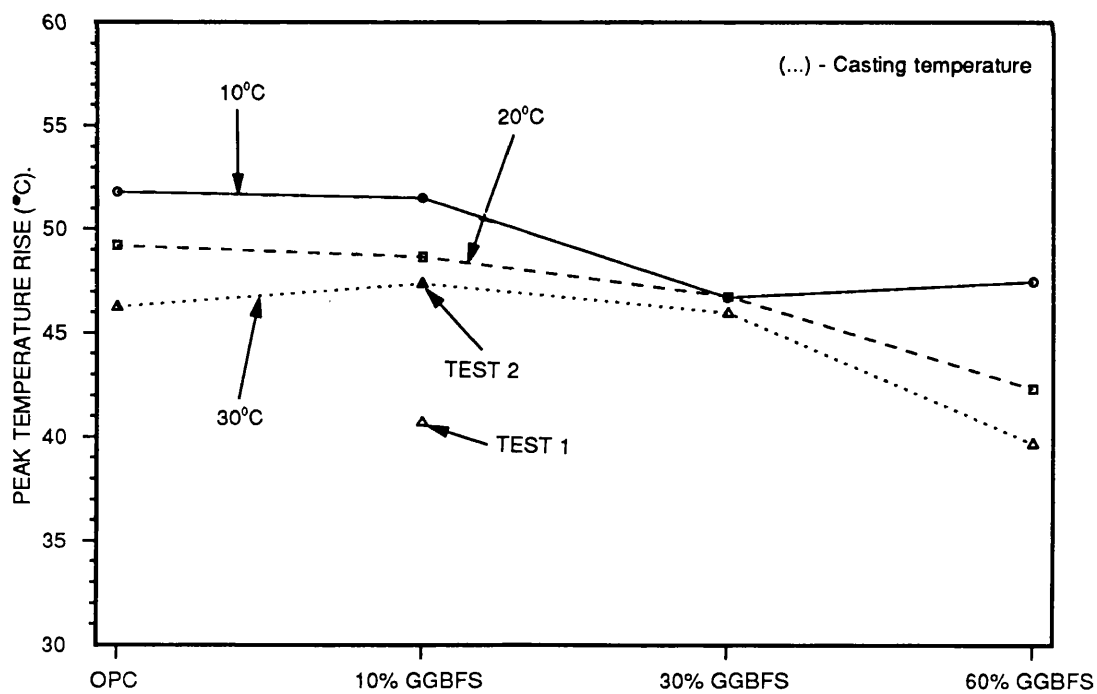
The shorter dormant period and lower peak temperature, found for 10 per cent PFA cast at the nominal temperature of 30^{0}C after having been cooled down from a temperature of 35^{0}C, has also been found for the 10 per cent GGBFS concrete that had to be cooled down, shown in Figure 8.13 as Test 1. The test was therefore repeated under more controlled conditions so as to achieve a concrete temperature of as close to 30^{0}C as possible without the need to cool the concrete. The results of the second test, shown in Figure 8.13 as Test 2, conformed with the general trend, i.e. 10 per cent GGBFS resulted not only in a longer dormant period but also in a higher peak temperature than the corresponding OPC concrete.

The effect of a higher mixing temperature, that despite having been lowered before casting, on the adiabatic temperature rise was further investigated for an OPC concrete mixed at 38.8^{0}C but cooled down to 30.2^{0}C before casting. As shown in Figure 8.14 the cooling of the concrete had no significant effect on the adiabatic temperature rise of the OPC concrete, i.e. the casting temperature was the important parameter and not the mixing temperature. It may be that a higher mixing temperature affects the adiabatic temperature rise of GGBFS and PFA concretes while it has no effect on OPC concretes. This may be important in the prediction of temperature rises in concretes that although mixed at ambient temperature are cooled down using liquid nitrogen cooling. Further results are however needed before these effects are substantiated.

The mixes with 30 per cent GGBFS extended the dormant period, (Figure 8.13a), but resulted in only very small decreases of the peak adiabatic temperature rises, (Figure 8.13b). It is clear that to achieve any appreciable reduction in the adiabatic temperature rise higher replacement levels of GGBFS are necessary, e.g. a 60 per cent cement replacement by GGBFS resulted in 8.4, 14.1 and 14.3 per cent reductions of the adiabatic temperature rises for the nominal casting temperatures of 10, 20 and 30^{0}C, respectively. This is in agreement with Bamforth[93] who has also recommended levels higher than 60



(a) Dormant period.



(b) Peak temperature.

Figure 8.13: The effect of the casting temperature on the dormant period and peak temperature rise of GGBFS mixes.

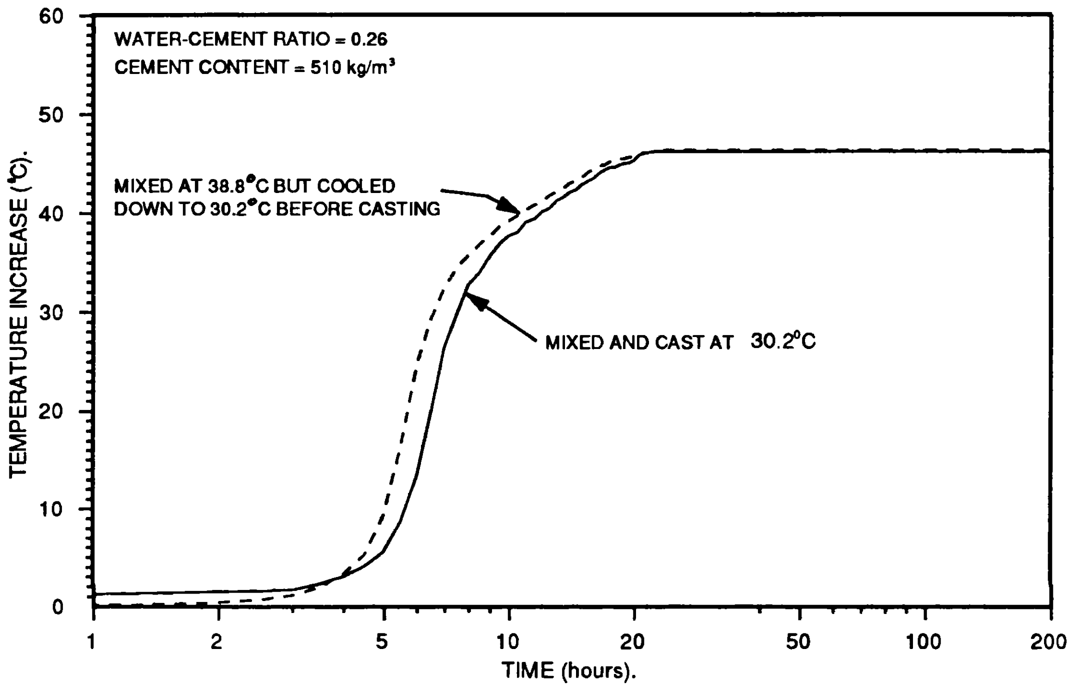


Figure 8.14: The effect of a higher mixing temperature on the adiabatic temperature rise of OPC mixes cast at 30°C.

per cent to achieve any appreciable reductions in the adiabatic temperature rises.

8.3.5 The effect of partial cement replacement by condensed silica fume (CSF).

The effect of partial cement replacement by CSF at the level of 10 per cent for concretes with water-binder ratio of 0.26 and nominal casting temperatures of 10, 20 and 30^{0}C is shown in Figures 8.15, 8.16, and 8.17. Partial cement replacement by 10 per cent CSF shortens the dormant period of the concretes at nominal mix temperatures of 20 and 30^{0}C, (Figure 8.18a). It does not however affect significantly the peak temperatures of the mixes, (Figure 8.18b). The shorter dormant period may be due to CSF particles acting as nucleation sites for the cement hydration products to form. The above effect has not been found for concrete mixed at 10^{0}C. The CSF concrete had a higher peak temperature than the OPC mix, i.e. CSF concrete had a peak temperature of 57^{0}C as compared to 51.8^{0}C for OPC concrete. No explanation can be given for this.

The equal or higher peak temperatures recorded are in contradiction to Helland's conclusion[195] that at the low water-cement ratios the CSF gives very little contribution to the heat of hydration, (see page 148). He has used mixes where 15 per cent of CSF was an additive and not a replacement material, and he consequently compared concretes with similar water-cement ratios but not water-binder ratios, i.e. water-(OPC + CSF) ratios. The resulting water-binder ratios of the CSF concretes were therefore lower than the corresponding OPC concretes. The reductions in the total evolution of heat of hydration could therefore be due to the effect of the water-binder ratio which has been described in Section 8.3.1, and has also been noted by Helland himself.

8.3.6 The effect of partial cement replacement by CSF in combination with PFA or GGBFS.

Partial cement replacement by 10 per cent CSF in combination with 36 per cent PFA, or 54 per cent GGBFS, have been investigated for concretes with water-binder ratios of 0.26

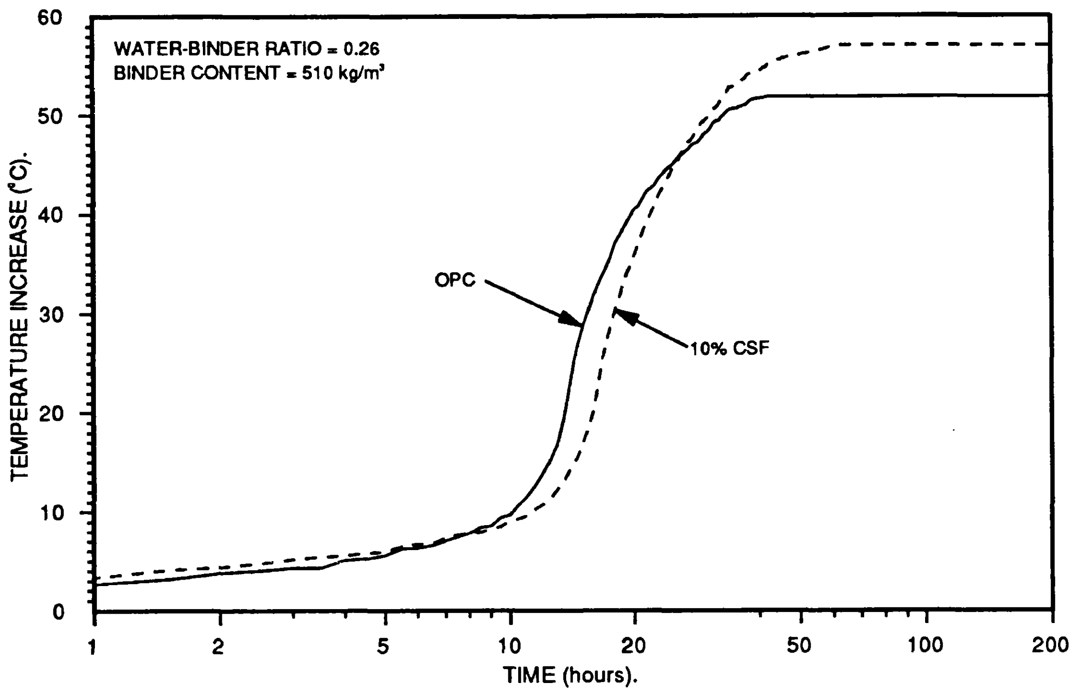


Figure 8.15: The adiabatic temperature rises of CSF mixes cast at 10°C.

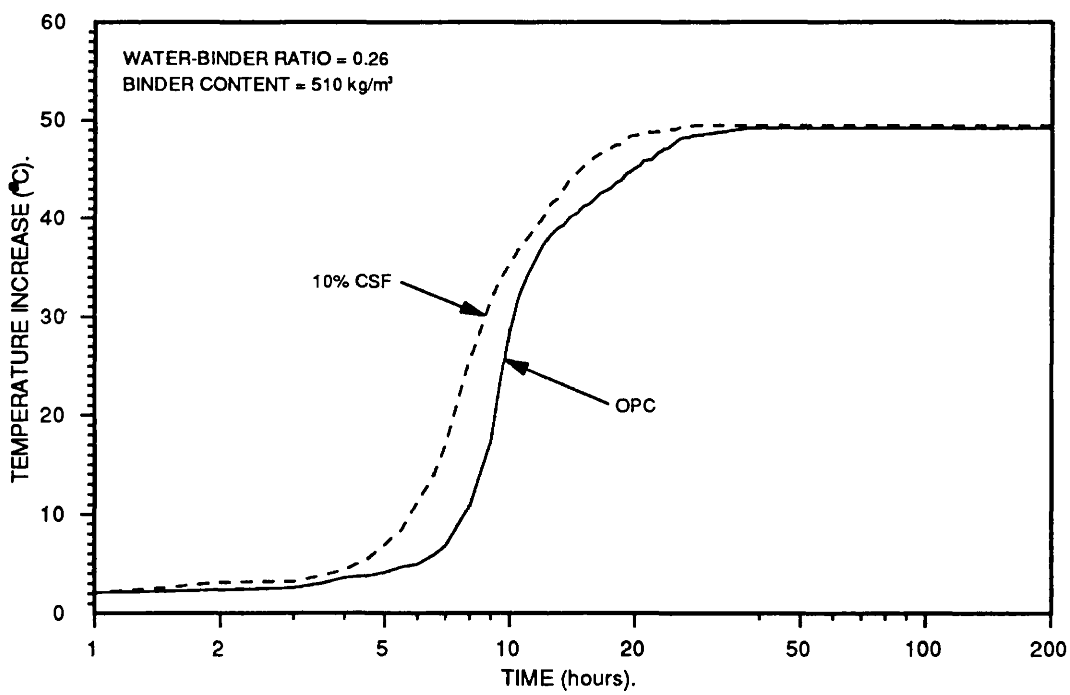


Figure 8.16: The adiabatic temperature rises of CSF mixes cast at 20°C.

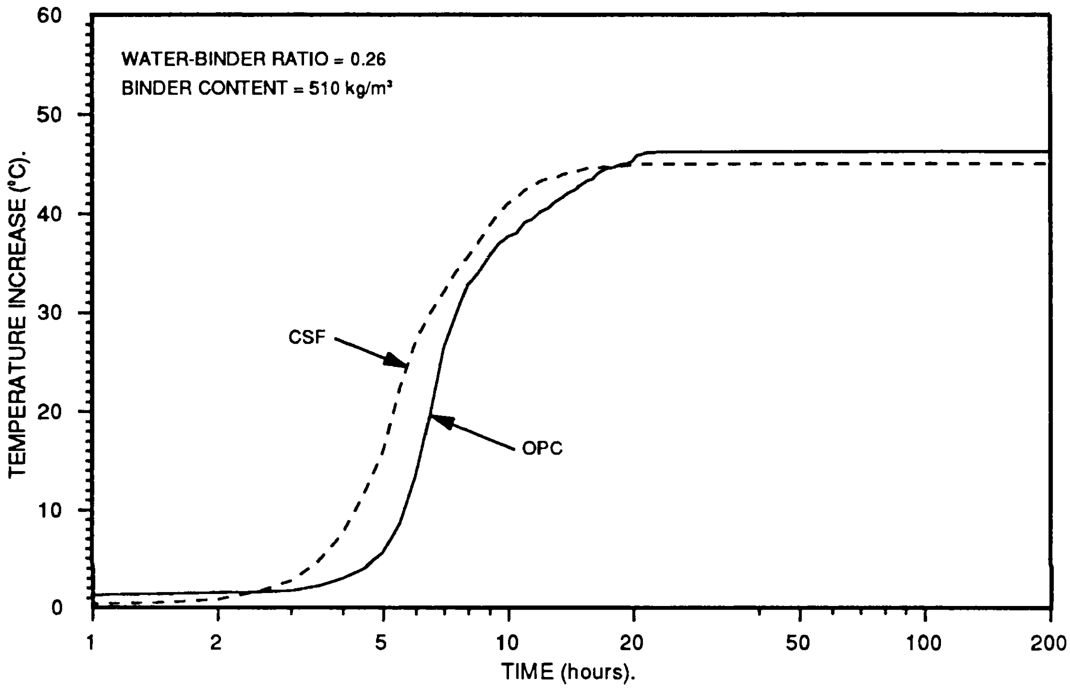
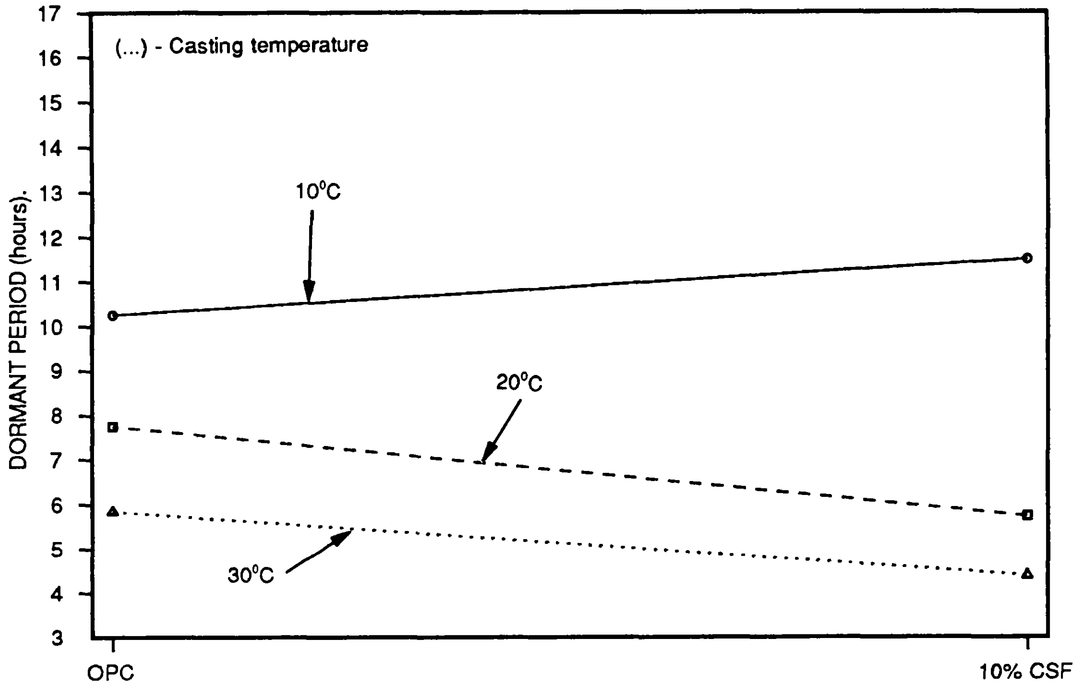
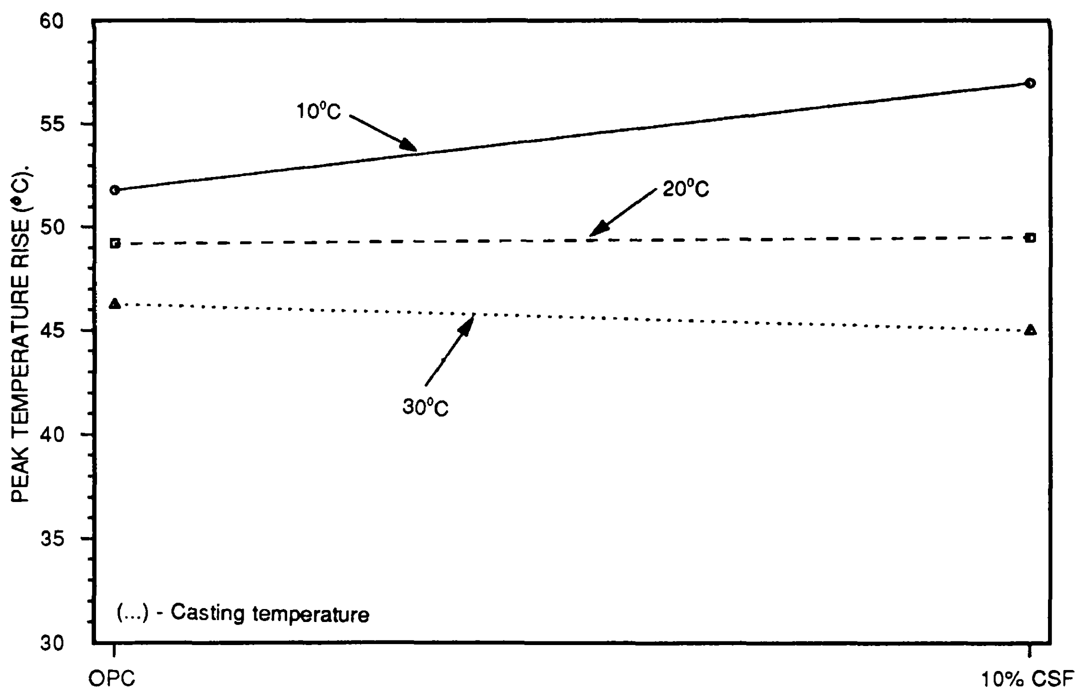


Figure 8.17: The adiabatic temperature rises of CSF mixes cast at 30°C.



(a) Dormant period.



(b) Peak temperature.

Figure 8.18: The effect of the casting temperature on the dormant period and peak temperature rise of CSF mixes.

and nominal casting temperatures of 10, 20 and 30°C. The results are shown in Figures 8.19, 8.20 and 8.21 for PFA mixes, and Figures 8.22, 8.23 and 8.24 for GGBFS mixes. It can be seen that despite the lower cement contents of PFA/CSF and GGBFS/CSF mixes as compared to 40 and 60 per cent PFA and GGBFS concretes, respectively, the dormant periods of the former are shortened, (Figures 8.25a and 8.26a), but the peak temperatures are also lower, (Figures 8.25b and 8.26b). A possible explanation for the shorter dormant periods, that has also been found for CSF mixes, could again be that CSF particles act as nucleation sites for the cement hydration products to form. The lower peak temperatures of PFA/CSF and GGBFS/CSF mixes could be due to the lower cement contents of these mixes.

8.4 CONCLUSIONS.

High levels of cement replacement by PFA and GGBFS, e.g., 40 and 60 per cent, respectively, are required to reduce significantly the adiabatic temperature rise of high strength concrete mixes. These mixes must still have the required strength at the specified age of testing. Their compressive strength development is described next.

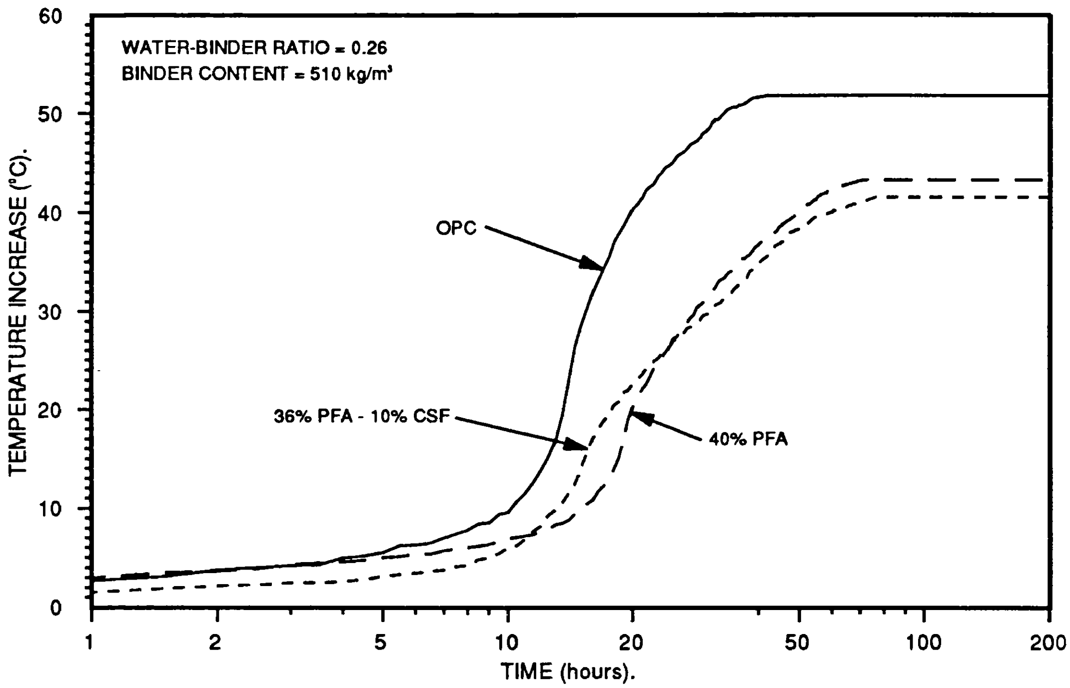


Figure 8.19: The adiabatic temperature rises of CSF-PFA mixes cast at 10^0C.

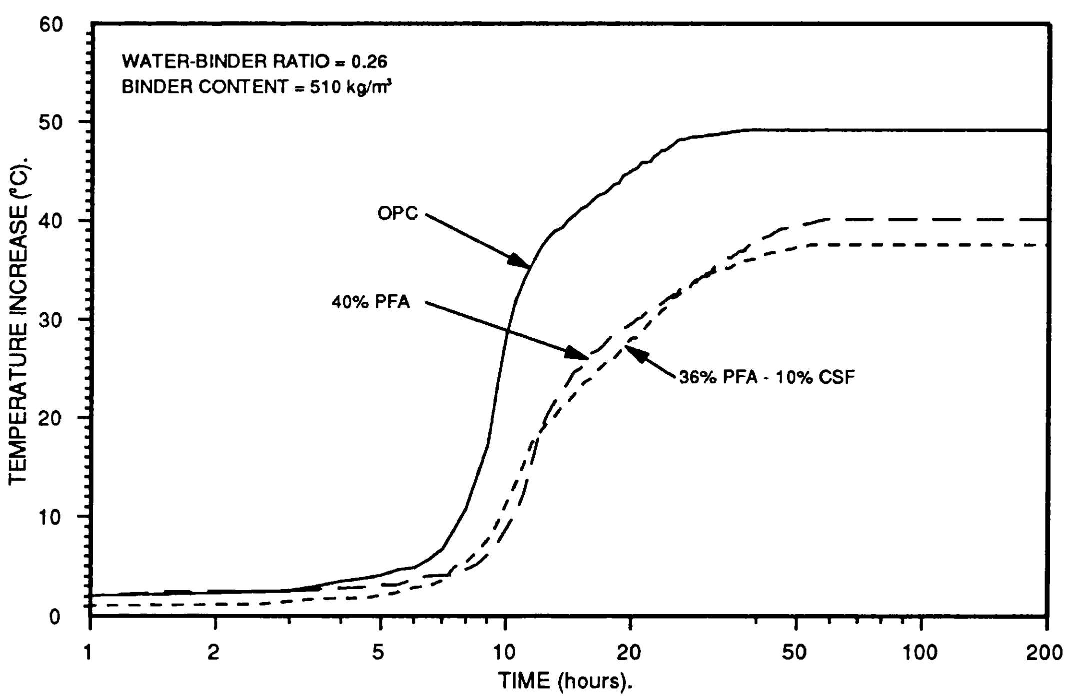


Figure 8.20: The adiabatic temperature rises of CSF-PFA mixes cast at 20^0C.

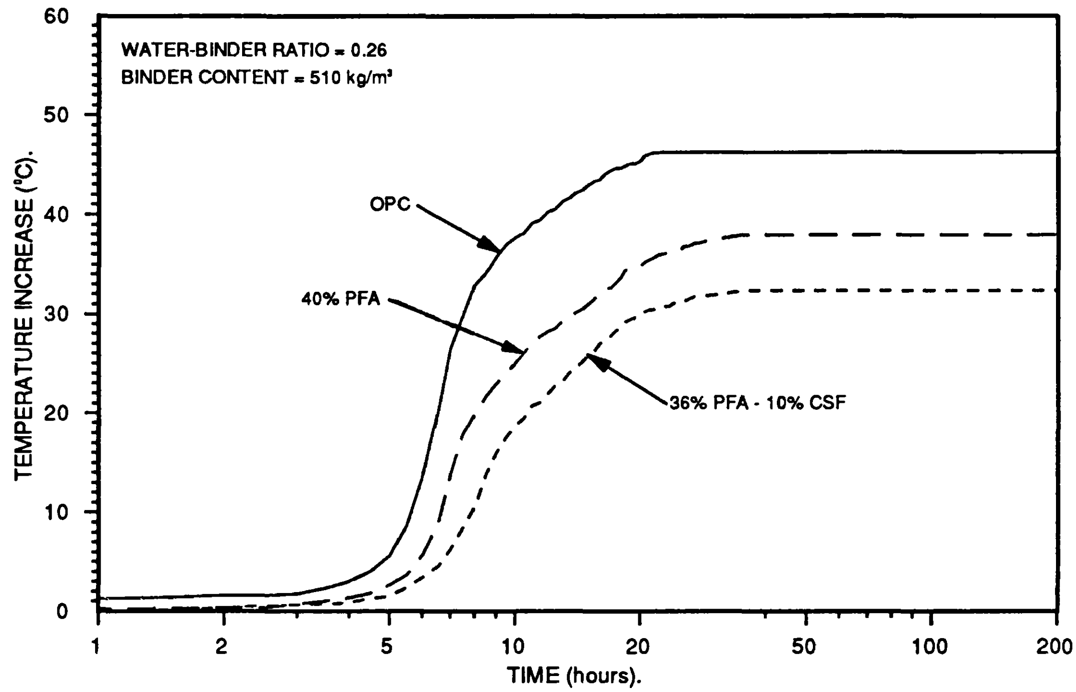


Figure 8.21: The adiabatic temperature rises of CSF-PFA mixes cast at 30°C.

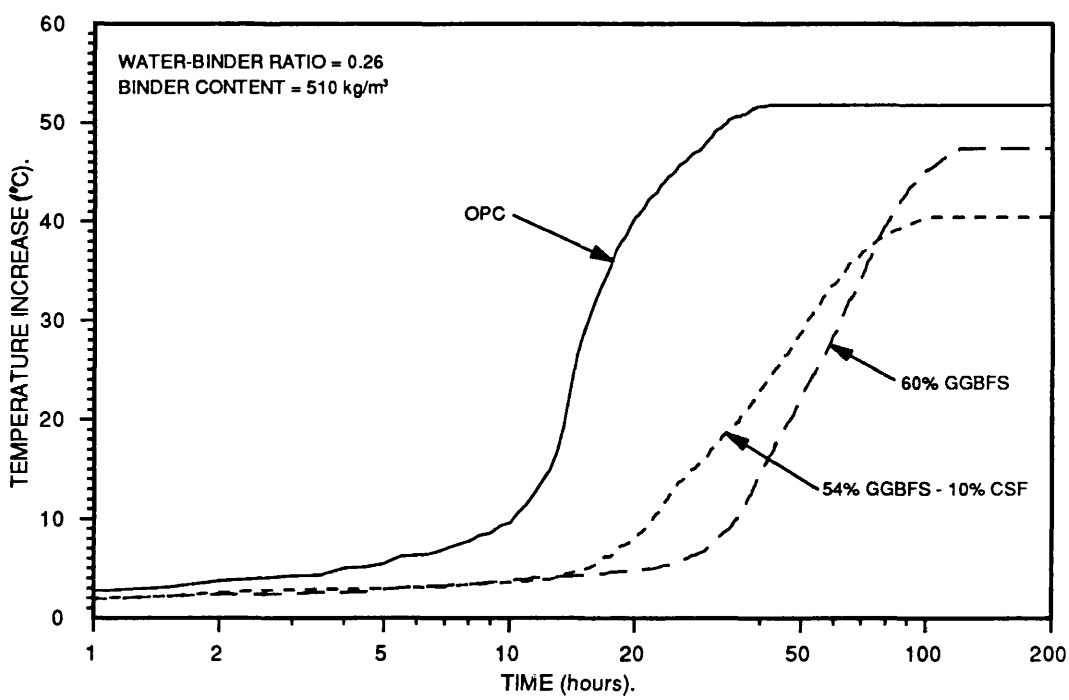


Figure 8.22: The adiabatic temperature rises of CSF-GGBFS mixes cast at 10°C.

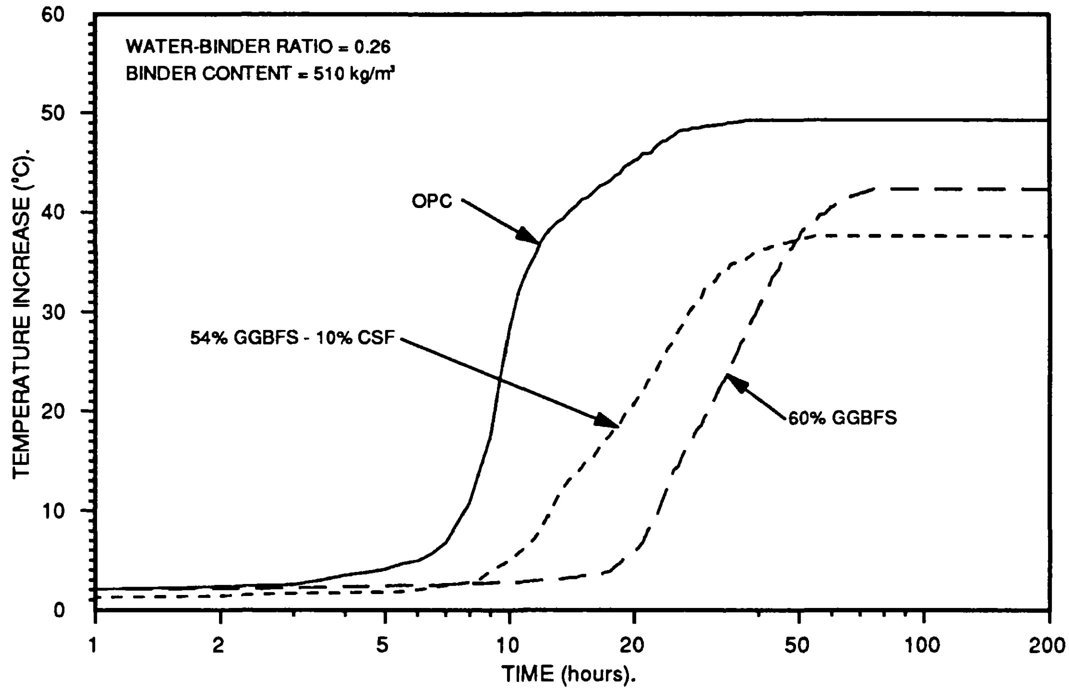


Figure 8.23: The adiabatic temperature rises of CSF-GGBFS mixes cast at 20^0C.

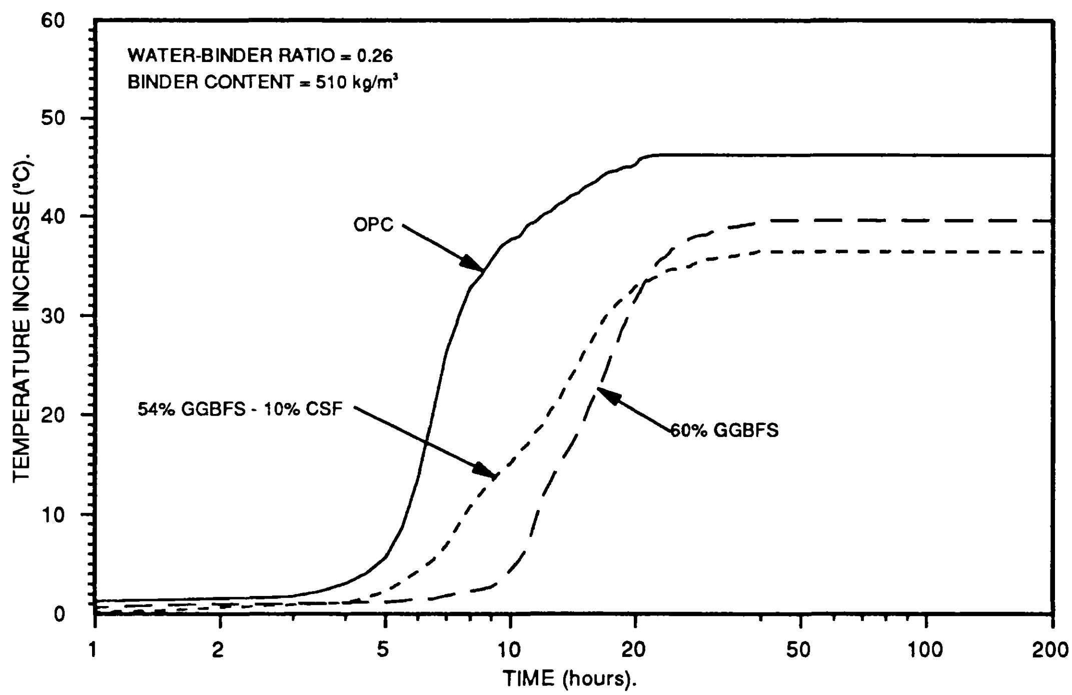
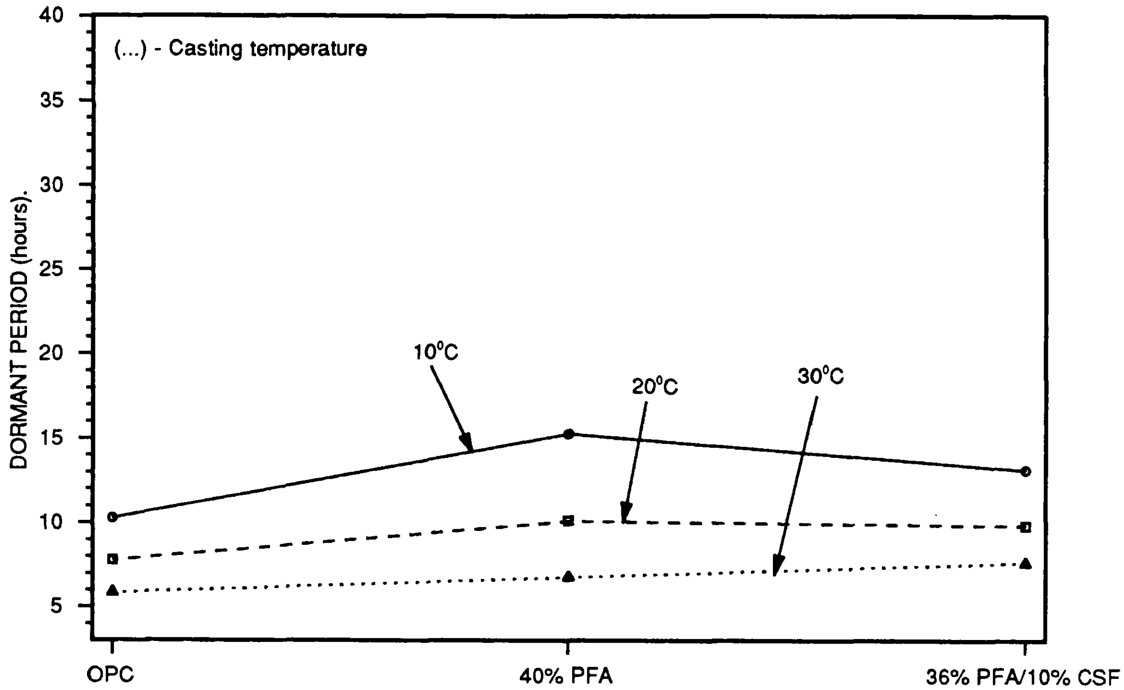
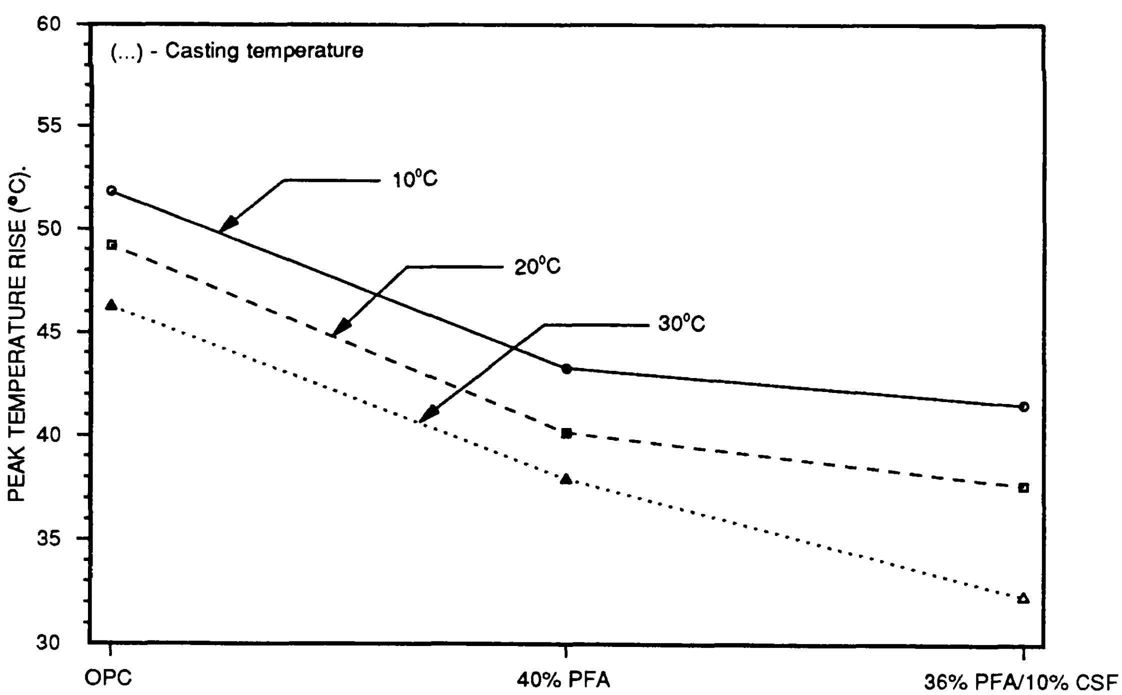


Figure 8.24: The adiabatic temperature rises of CSF-GGBFS mixes cast at 30^0C.

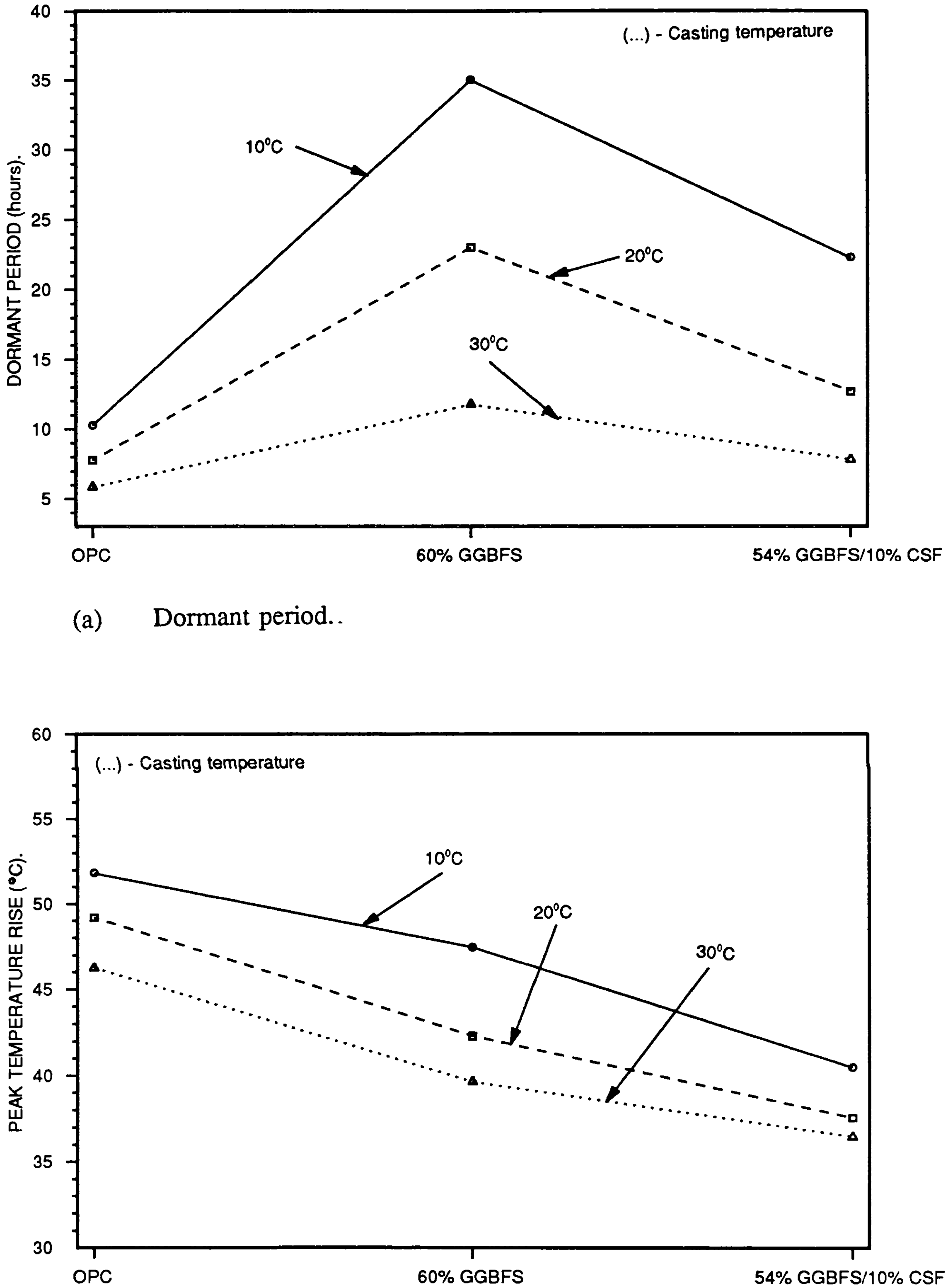


(a) Dormant period.



(b) Peak temperature.

Figure 8.25: The effect of the casting temperature on the dormant period and peak temperature rise of CSF-PFA mixes.



(a) Dormant period..

(b) Peak temperature.

Figure 8.26: The effect of the casting temperature on the dormant period and peak temperature rise of CSF-GGBFS mixes.

CHAPTER 9.
STRENGTH DEVELOPMENT CHARACTERISTICS OF HIGH STRENGTH CONCRETE.

9.1 INTRODUCTION.

The results of the compressive strength tests described in Chapter 4, Section 4.2.4, are presented in this chapter. The parameters studied may be divided in four sections:

1. The effect of coarse aggregate properties and mix design procedure. Three types of aggregate, i.e. gravel, limestone and granite, with different crushing values and surface textures and maximum sizes of 20 and 10 mm were used. Mixes were designed according to the "Maximum Density Theory", since at this stage of the work, its modified version had not yet been developed. Later, the modified version was used to design mixes with 10 mm granite aggregate.

2. The effect of partial cement replacement by pulverised fuel ash (PFA) at replacement levels of 10, 20 and 40 per cent, ground granulated blast furnace slag (GGBFS) at replacement levels of 10, 30 and 60 per cent, condensed silica fume (CSF) at replacement levels of 5, 10 and 15 per cent. The effect of combinations of CSF with GGBFS or PFA has also been investigated.

3. Investigation of the effect of maximum aggregate size with mixes designed according to the "Modified Maximum Density Theory". Mixes with ordinary Portland cement (OPC) and with 10 per cent cement replacement by CSF have been tested.

4. The effect of increasing fineness modulus of sand. Sands with fineness moduli of 2.1 and 2.73 have been used in mixes with ordinary Portland cement (OPC), and with 10 per cent cement replacement by CSF.

The water-binder ratios of the above mixes were between 0.20 and 0.38.

9.2 EXPERIMENTAL PROCEDURE.

The procedures for mixing concrete, casting and curing specimens were as described in Chapter 5. Although it was desired to retain the slump of the concrete mixes above 100 mm, even at low water-binder ratios for ease of consolidation, this was not found possible with all the mixes designed according to the "Maximum Density Theory", (see Section 6.3.2). The mixes designed according to its modified version all had slumps above 100 mm and some were even above 200 mm, despite aiming for a slump of approximately 150 mm. The superplasticiser dosage required for this was estimated before the more accurate "water-binder ratios versus superplasticiser dosage", described in Chapter 7, had been determined. Sufficient cubes were cast for sets of five to be tested for density and compressive strength. The test procedures for these measurements were also described in Chapter 5.

9.3 RESULTS AND DISCUSSION.

Detailed data of all the mixes used can be found in Appendix 5.

9.3.1 The effect of coarse aggregate type and size, and mix design procedure.

OPC mixes with three types of coarse aggregate, i.e. gravel, limestone and granite with maximum sizes of 20 and 10 mm were tested to determine the effect of crushing value, surface texture and maximum size on the relationship between water-cement ratio and compressive strength at the age of 28-days. These mixes were designed according to the "Maximum Density Theory", at a constant voids overfill of 5 per cent.

The compressive strength results of 20 and 10 mm gravel, limestone and granite are shown in Figures 9.1 to 9.3, and there is no clear indication that the smallest size of coarse aggregate produces the highest strength. These results showed a large scatter within some of the sets of five cubes, e.g., the 10 mm limestone mix with a water-cement ratio of 0.34 had a range of 18 N/mm^2 and a standard deviation of 6.7 N/mm^2. The densities of the cubes also varied from 2375 to 2448 kg/m^3. This was due to incomplete compaction which was attributed to the fact that the cube moulds were placed on the vibrating table without fixing them. A frame was therefore made that clamped the cube moulds to the vibrating table, and was used for all of the tests whose results are given from now on. Its efficiency in reducing the scatter of the results is discussed later in this section.

The relationship between water-cement ratio and 28-day average compressive strength for the different aggregates of the same maximum size is shown in Figures 9.4 and 9.5, superimposed on data collected by Parrott[32], (see page 206), for concrete with a wide variety of binders, admixtures and aggregates.

The strength of the gravel mixes is just below the strength envelope determined by Parrott. Gravel aggregate also produced lower strengths than limestone despite having a lower crushing value (17.2 per cent as compared to 20.7 per cent for limestone, see page 238). The strengths obtained using gravel aggregates, however, conform to Nagataki's relation, (see page 94), between the compressive strength and the crushing value of aggregate, as shown in Figure 9.6. Limestone, despite the high aggregate crushing value of 20.7 per cent, has produced much higher compressive strengths than predicted by this relation. The most likely reason for this is the difference in surface texture and particle shape between gravel, i.e. rounded aggregate, and crushed stone. A greater mechanical bond can develop with angular particles, (see page 96). Indeed, observation of the concrete made with gravel aggregate after testing showed, as with the preliminary tests described in Section 6.2, that the failure was initiated at the mortar-aggregate interface, i.e. in the transition zone. This prompted the study of three additional gravel mixes, one was an OPC mix and two were with partial cement replacement by 5 and 10 per cent CSF. The compressive strength results are shown in Figure 9.7, and it can be seen that 5 per cent CSF has increased the 28-day compressive strength from 67 to 81 N/mm^2. The failure mode of the specimens also changed; the concrete failed explosively and it

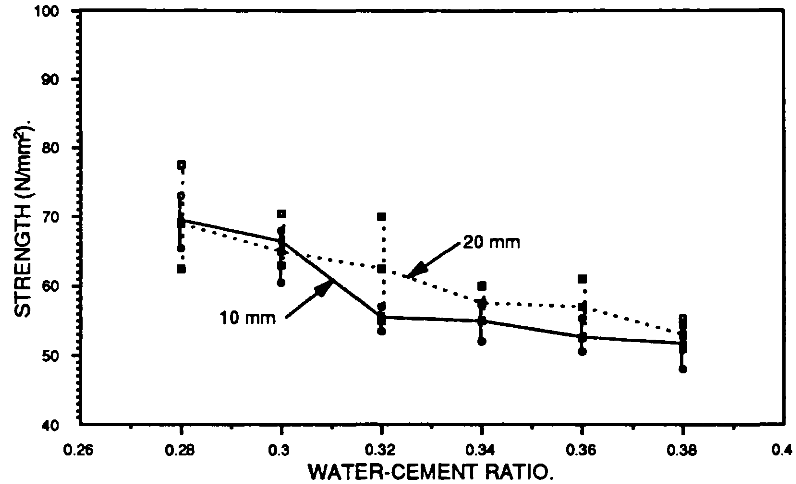


Figure 9.1: 28-day compressive strengths of gravel mixes.

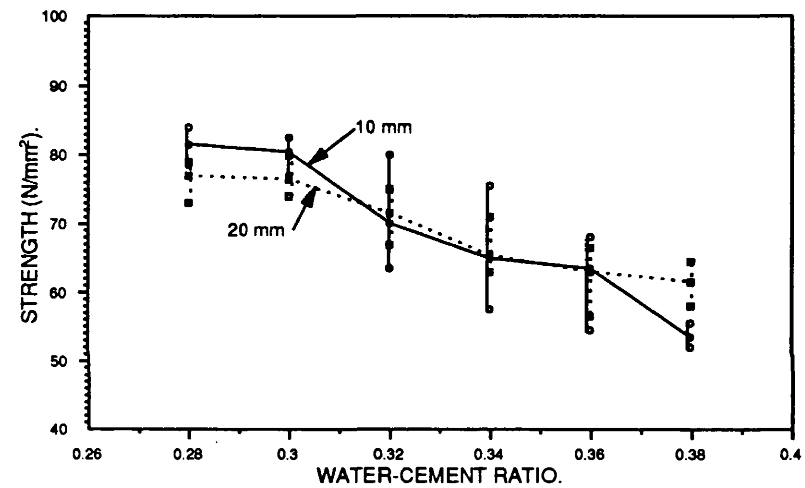


Figure 9.2: 28-day compressive strengths of limestone mixes.

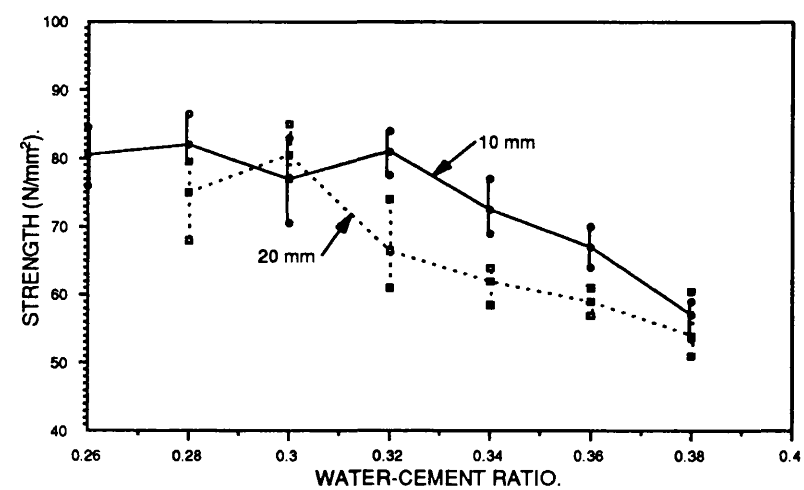


Figure 9.3: 28-day compressive strengths of granite mixes.

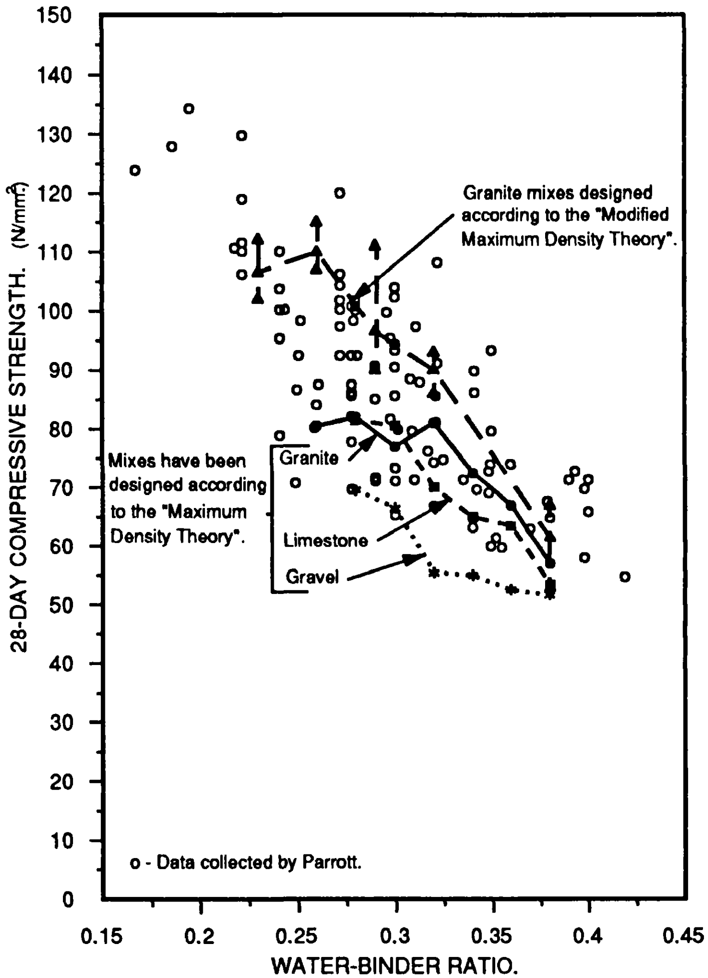


Figure 9.4: The relationship between water-cement ratio and 28-day average compressive strength for mixes with 10 mm maximum aggregate size.

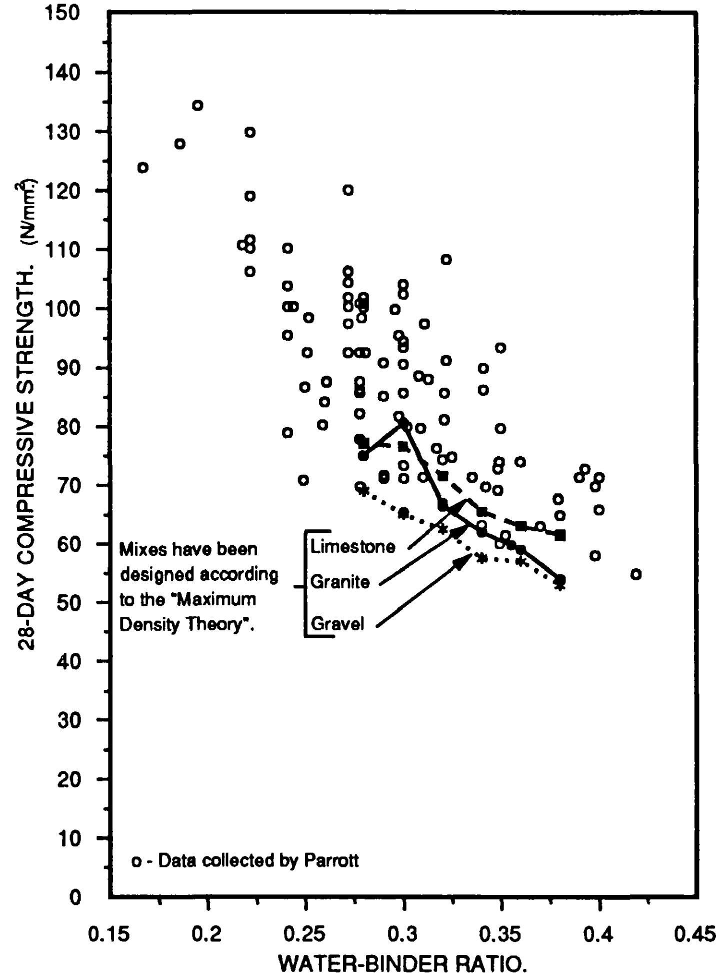


Figure 9.5: The relationship between water-cement ratio and 28-day average compressive strength for mixes with 20 mm maximum aggregate size.

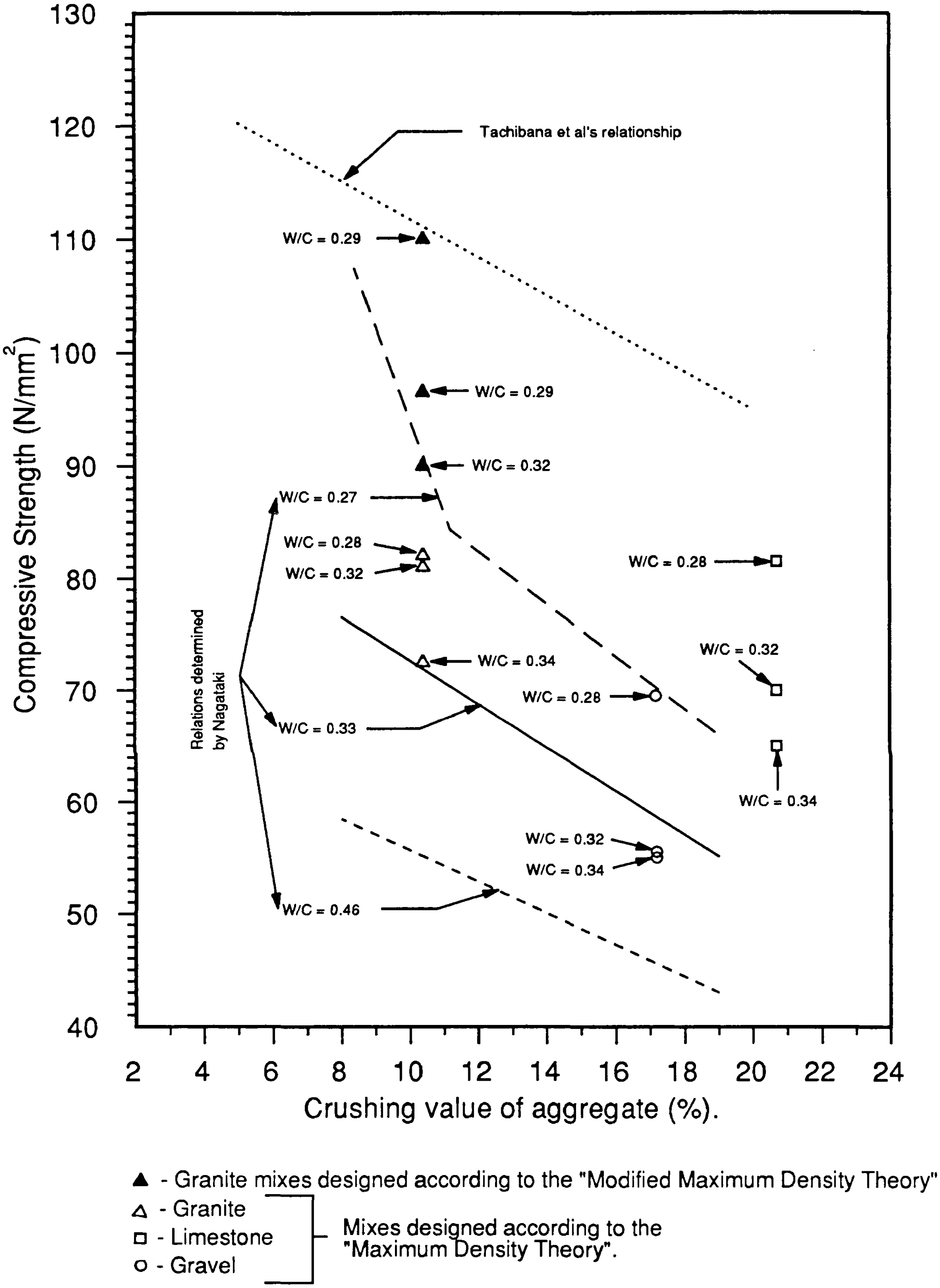


Figure 9.6: 28-day compressive versus crushing value of aggregate.

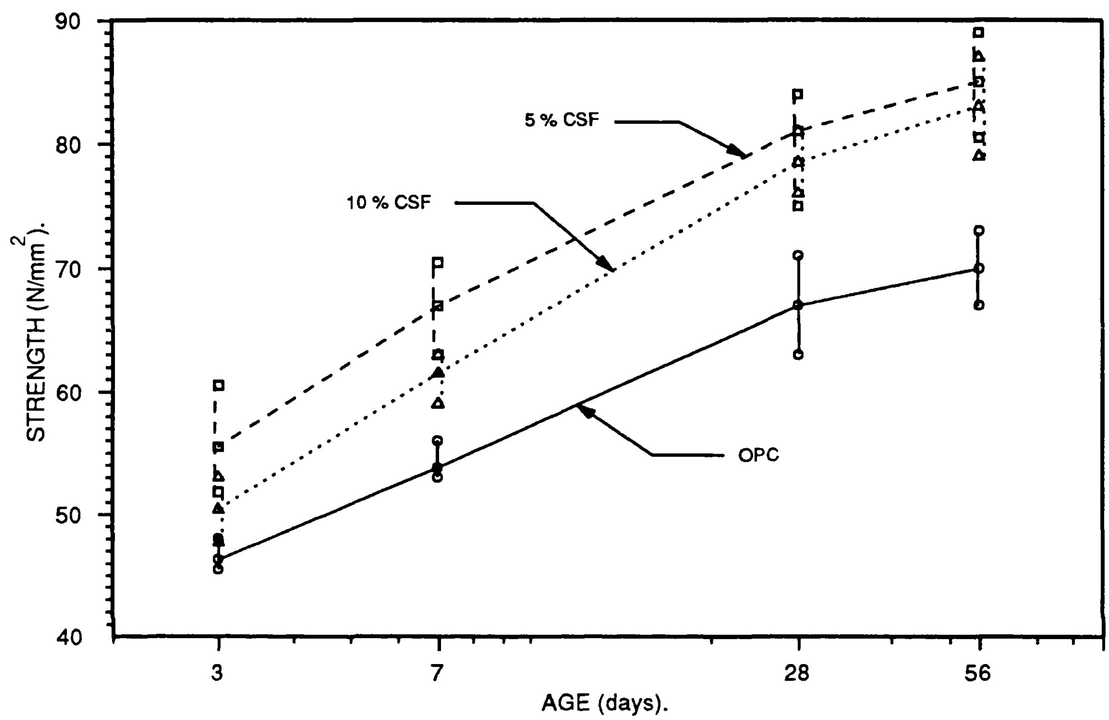


Figure 9.7: The effect of CSF on the compressive strength development of concrete mixes with 20 mm gravel.

ruptured through the coarse aggregate. This is in agreement with several researchers[180,186,187,189] that have shown that the microstructure of the transition zone can be improved by the use of CSF, (see page 140). The use of CSF may also have been the reason that Tachibana et al[138] found a comparatively small decrease in compressive strength due to the increase in the crushing value, (see page 94). Their results are also shown in Figure 9.6. It may therefore be concluded that the relation between the compressive strength and the crushing value is affected both by the surface texture of the aggregate as well as by the use of CSF.

The strength of the limestone and granite mixes, designed according to the "Maximum Density Theory", are in the lower part of the strength envelope determined by Parrott[32], as shown in Figures 9.4 and 9.5. It is also apparent that the strength of these mixes is unlikely to be increased above 80 N/mm^2 by lowering the water-cement ratio below 0.30.

Two other investigators[76,104] have also found that, for some aggregates, a point is reached beyond which further increases in cement content, with consequent reduction in water-cement ratio, produce no increase in the compressive strength of the concrete. This apparently is due to having reached the limit of the bonding potential of that cement-aggregate combination, which ACI Committee 363 called the "intrinsic aggregate strength", i.e. no matter how much stronger the paste is made, failure is initiated in the transition zone, (see page 96).

Indeed, it appears from Figures 9.4 and 9.5, that the bonding potential of the cement-aggregate combination has increased resulting in compressive strengths of 110 N/mm^2 when the mix proportions for 10 mm granite mixes were optimised using the "Modified Maximum Density Theory", that has been described in Chapter 6. While the strengths of granite mixes designed according to the "Maximum Density Theory" are as predicted by Nagataki's relation, (see Figure 9.6), the strengths of granite mixes designed according to the modified theory are much higher. Their strength is also higher in the strength envelope determined by Parrott[32], (see Figure 9.4), despite being neat OPC concrete mixes without any mineral admixtures. This improvement was at first attributed to the change in the casting procedure; the cube moulds were clamped to the vibrating table as described earlier. Since this was aimed at reducing the scatter of results within the sets of five cubes, it might have also increased the average compressive strength. However, the results did not show any reduction in the scatter, e.g., the mix with a water-cement ratio of 0.39 had a range of 21 N/mm^2 and a standard deviation of 8.8 N/mm^2. Other possible reasons for this improvement of strengths are:

1. Mixes designed according to the "Maximum Density Theory" required superplasticiser dosages above 2.0 per cent solids by weight of cement in order to enable water-cement ratios lower than 0.30 to be used, e.g., 20 mm limestone mixes at a water-cement ratio of 0.28 required a superplasticiser dosage of 2.5 per cent of solids by weight of cement for a slump of 20 mm. The high superplasticiser dosage in combination with a low workability may have caused the entrainment of air with resulting strength reductions. Optimisation of mix proportions using the "Modified Maximum Density Theory" made possible the use of lower water-cement ratios, as low as 0.23, without requiring excessive superplasticiser dosage in order to retain the slump above 100 mm.

2. The failure planes of the mixes designed according to the "Maximum Density Theory" showed a weak bond not only between the mortar and the aggregate but also between the paste and the sand. The "Modified Maximum Density Theory" has shown that there is a critical percentage overfill of voids by the paste above which the concrete "fluidifies", i.e. the slump suddenly increases from less than 100 mm to above 150 mm with only a 1 per cent increase in the overfill, (see page 265). This amount of overfill is required to lubricate the aggregate sufficiently, thus allowing the particles to move past each other easily. It appears that this amount is not only critical in "fuidifying" the concrete but is also the critical amount that will sufficiently cover all the surface of the aggregate and therefore bind them together.

The compressive strength of 110 N/mm^2 with a 100 mm cube appears to be the highest that can be achieved with only ordinary Portland cement and 10 mm granite aggregate.

According to other workers[35,143], (see pages 205 and 222), the bonding potential of the cement-aggregate combination can only be increased by incorporating mineral admixtures.

It was unfortunate that soon after I completed this first series of experiments I was informed that the ARC depot at Greenwich from where I received the limestone had closed down. Although their West Drayton depot could supply me with limestone its source was different. This would have meant repeating the preliminary tests with resulting delays in the rest of the research programme. The very high strengths that have been obtained with OPC concrete using 10 mm granite aggregate once the mix proportions were optimised according to the "Modified Maximum Density Theory", led to the subsequent use of this aggregate in this investigation.

9.3.2 Pulverised fuel ash (PFA).

Three water-binder ratios were considered essential to determine not only the contribution of PFA to the strength of concrete but also to show if this contribution is affected by the water-binder ratio. The water-binder ratios of 0.38, 0.32 and 0.26, were chosen as they corresponded to 28-day OPC concrete strengths of 60, 90 and 110 N/mm^2, (Figure 9.4).

Three basic mix proportioning approaches have been used, for normal strength PFA concrete, with a view to obtaining either reduced heat of hydration or, more recently, overcoming difficulties encountered in getting acceptable levels of strength in concrete at early ages.These are:

1. partial replacement of cement,
2. addition of PFA as fine aggregate,

and, 3. partial replacement of both cement and fine aggregate.

The first approach, i.e. partial replacement of cement, has been favoured for the production of high strength concrete as it produces acceptable strengths and, at the same time, it may, as has been shown in Section 8.3.3, reduce the heat of hydration of the mixes. For example, the procedure used by Cook[58] to determine economical mixture proportions for the concrete used in the 75-storey Texas Commerce Tower was replacement of Portland Cement with Class C (high calcium) PFA on a "pound-for-pound" basis, (see page 54). Levels of cement replacement by PFA that have usually been used for the production of high strength concrete have been between 10 and 40 per cent (see Table 2.2, page 28).

The mix proportions of concretes studied in this programme were optimised for OPC mixes according to the "Modified Maximum Density Theory". Partial replacement of cement by PFA was adopted and the levels of replacement were 10, 20 and 40 per cent. The strength development characteristics of the resulting concretes are shown in Figures 9.8 to 9.10. Only the average strengths are shown for clarity, and this has subsequently been adopted for all the figures. The ranges of strengths can be obtained from the detailed results that can be found in Appendix 5.

It can be seen from Figures 9.11 to 9.13, showing the compressive strengths expressed as a percentage of the 28-day strength of the reference OPC concrete, that the major strength gain of OPC concrete is mainly within the first 28-days. Although concretes made with partially replacing OPC by PFA have lower 28-day compressive strengths they gain considerable strengths at later ages. For cement replacement levels by PFA of 10 and 20 per cent and water-binder ratios of 0.38 and 0.32 the strength is equal or even higher than that of OPC concrete at 56-days. This is in agreement with Berry and Malhotra[(150)] who found that for high strength concrete, PFA functions by providing increased strength at late ages of curing (56 to 91-days). This they concluded from the strength development of concretes used for the Water Tower Place and the River Plaza, (see Table 2.2, page 28), which both had a water-binder ratio of approximately 0.32, the cement replacement level by PFA was 10 per cent, and the 28-day compressive strength was around 70 N/mm^2.

However, for the low water-binder ratio of 0.26, PFA concrete, even at low cement replacement levels of 10 and 20 per cent and despite its considerable strength development beyond 28-days, does not equal the compressive strength of OPC concrete even after 180-days. There is therefore an indication that PFA performs more efficiently

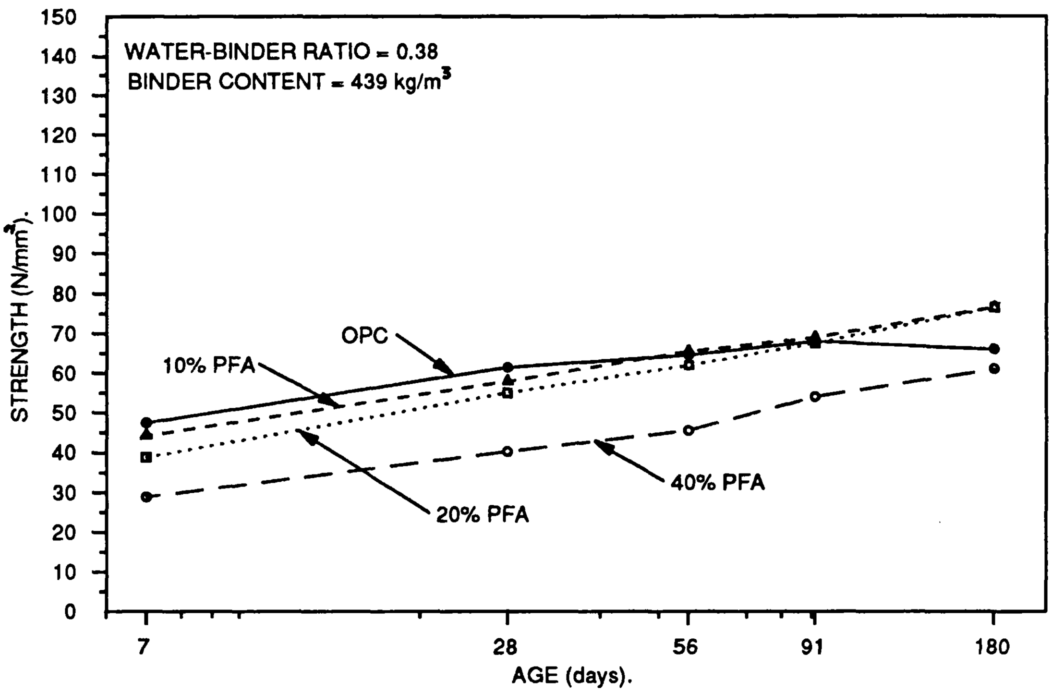


Figure 9.8: The strength development characteristics of PFA concretes with a water-binder ratio of 0.38.

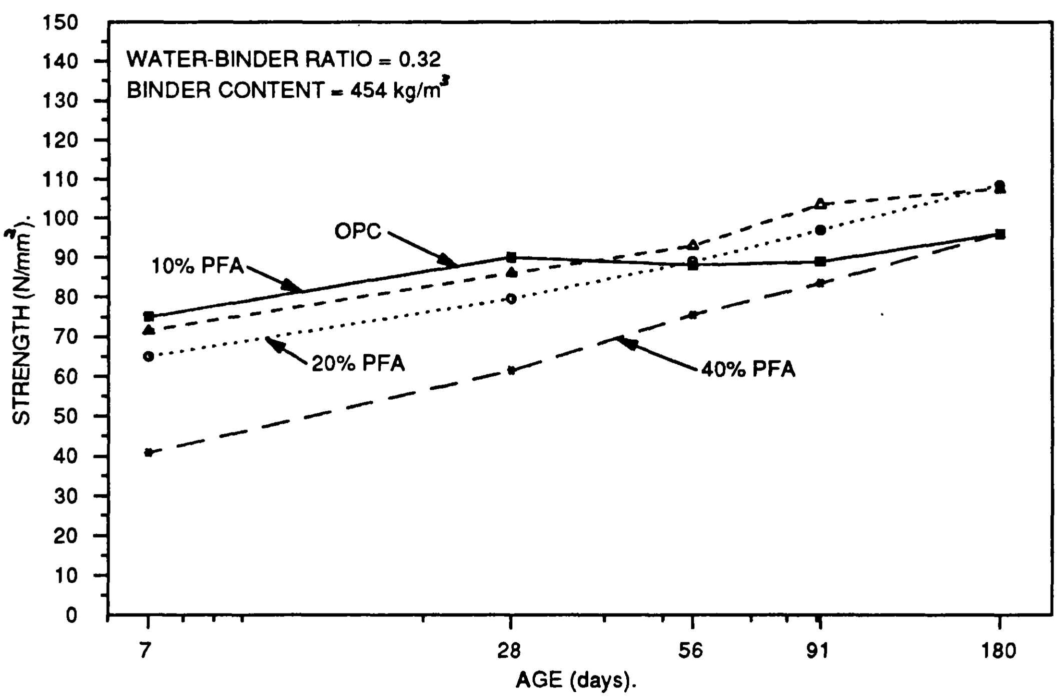


Figure 9.9: The strength development characteristics of PFA concretes with a water-binder ratio of 0.32.

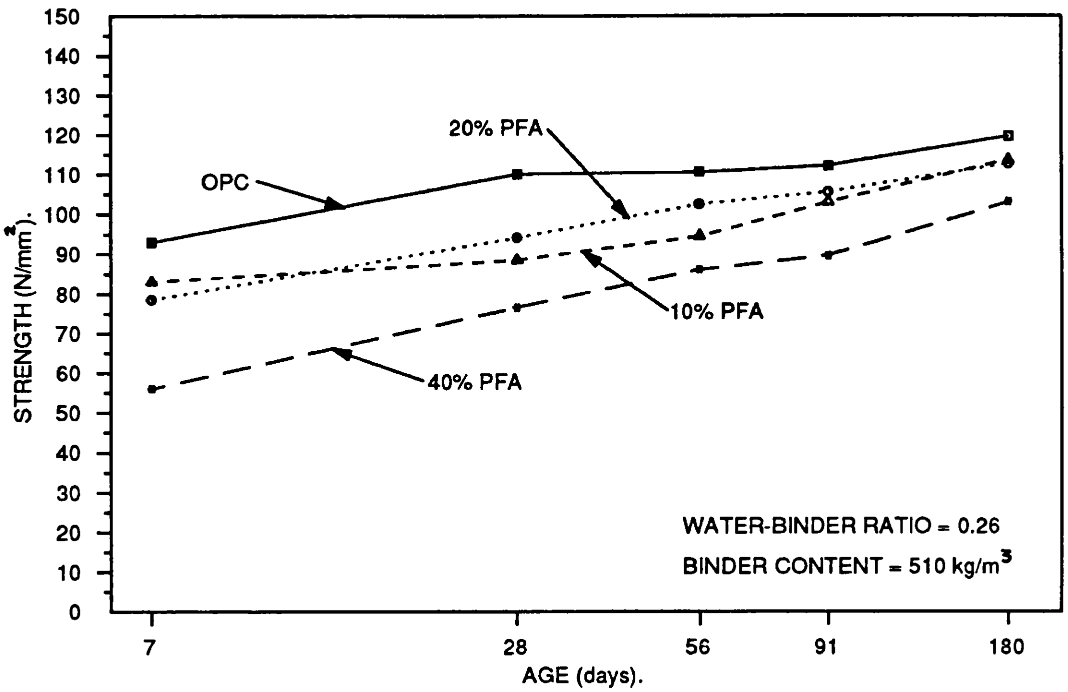


Figure 9.10: The strength development characteristics of PFA concretes with a water-binder ratio of 0.26.

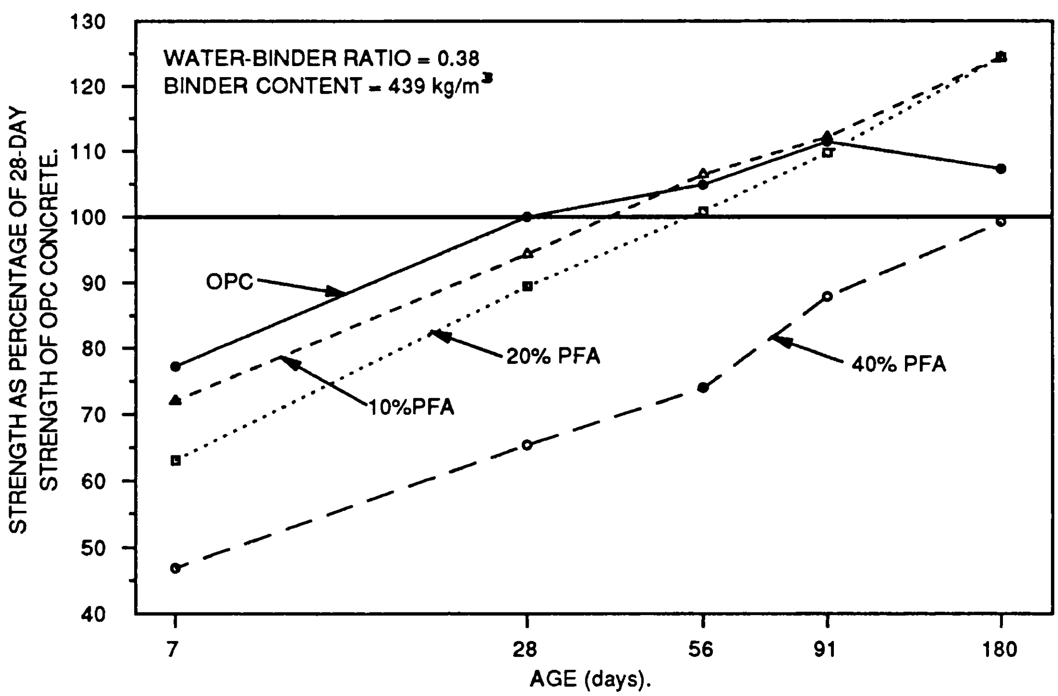


Figure 9.11: Compressive strengths of PFA concretes, (with a water-binder ratio of 0.38), expressed as a percentage of the 28-day strength of the reference OPC concrete.

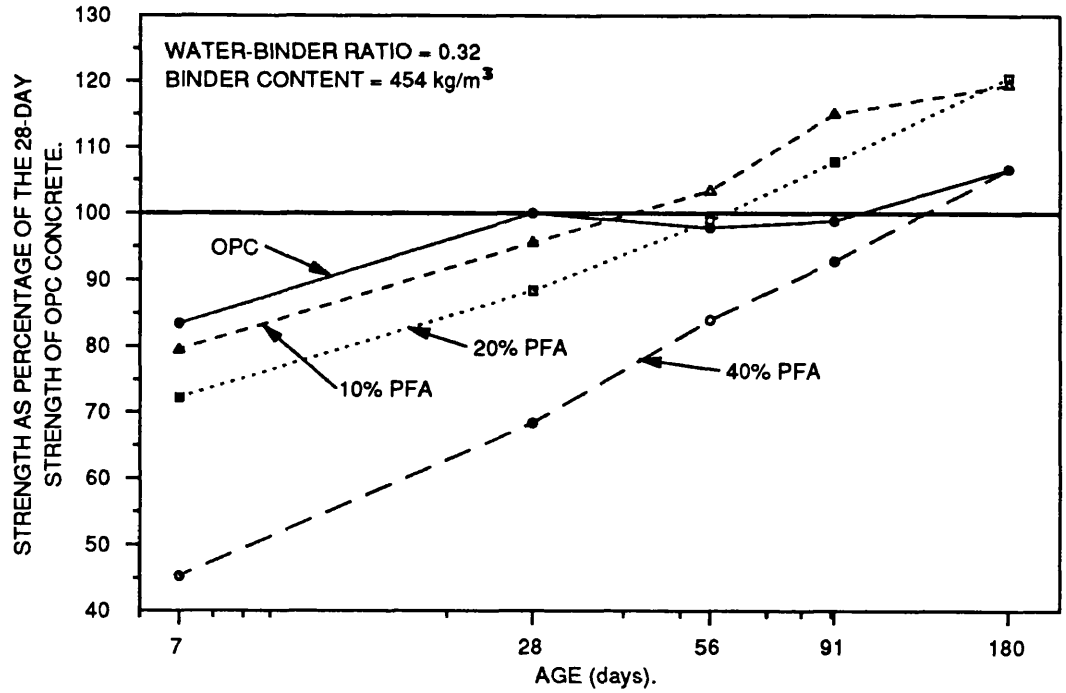


Figure 9.12: Compressive strengths of PFA concretes, (with a water-binder ratio of 0.32), expressed as a percentage of the 28-day strength of the reference OPC concrete.

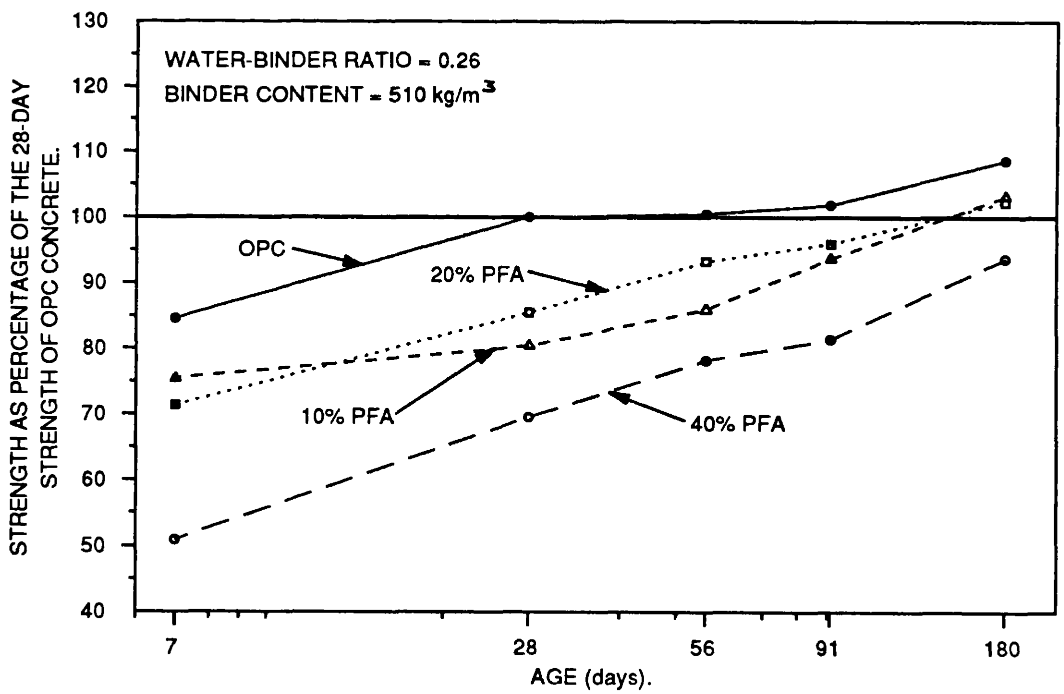


Figure 9.13: Compressive strengths of PFA concretes, (with a water-binder ratio of 0.26), expressed as a percentage of the 28-day strength of the reference OPC concrete.

at high water-binder ratios. This effect of the water-binder ratio has also been found for GGBFS and CSF, as will be reported in the next two sections. Possible reasons for this effect will be given for CSF concrete for which the literature offers more information. Some of these may also apply for PFA but more research is required before they can be substantiated.

Cement replacement levels by PFA of 40 per cent do not equal the strength of OPC concrete, at any of the water-binder ratios used, even after 180-days.

9.3.3 Ground granulated blast furnace slag (GGBFS).

The same three water-binder ratios, i.e. 0.38, 0.32 and 0.26, as have been used for PFA concrete, were also chosen in order to study the effects of partially replacing OPC with GGBFS.

In some cases, GGBFS has been used in proportions of up to 70 per cent by mass of the total cementitious material, (see page 129). For this reason, higher cement replacement levels than those used for PFA concrete have been studied. These were 10, 30 and 60 per cent. The strength development of these concretes is shown in Figures 9.14 to 9.16.

10 per cent cement replacement by GGBFS at the water-binder ratios of 0.38 and 0.32 exhibited 28-day compressive strengths equal to the OPC concrete and greater strengths beyond 28-days. 30 per cent replacement at the water-binder ratio of 0.32 exhibited a 28-day compressive strength almost equal to the OPC concrete and greater strengths beyond 28-days. 30 per cent replacement at the water-binder ratio of 0.38 exhibited abnormally low strength development, even lower than the concrete with 60 per cent replacement. Even mixes with 60 per cent replacement, water-binder ratios of 0.38 and 0.32, exhibited strengths higher than the corresponding OPC concrete at the ages of 180-days and 56-days respectively.

However, for the low water-binder ratio of 0.26, GGBFS concrete, as was the case with

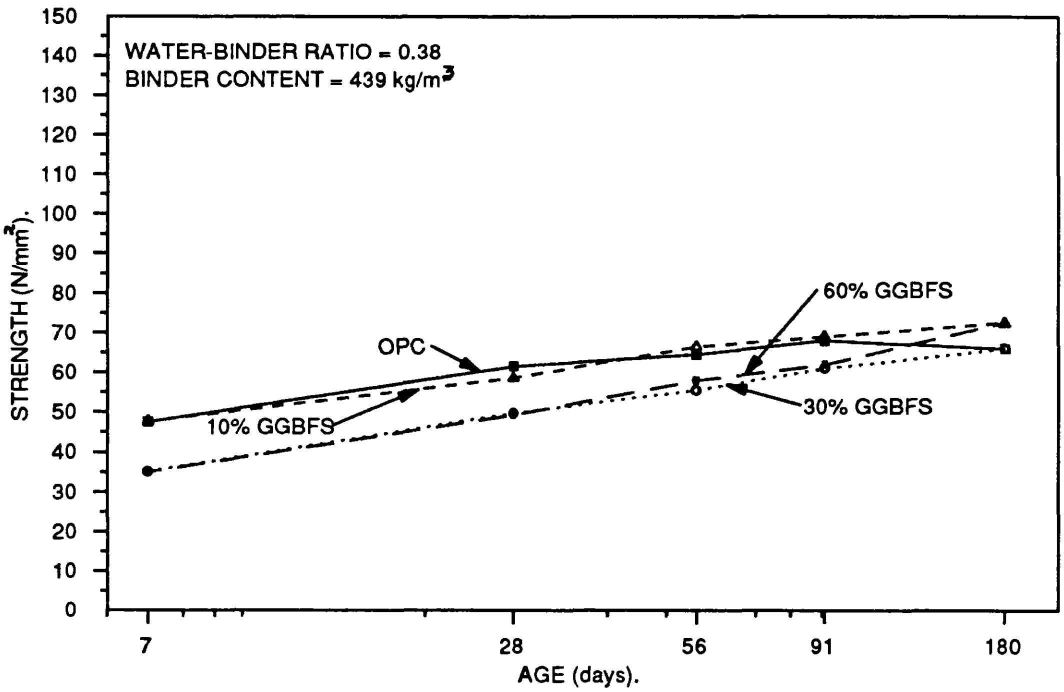


Figure 9.14: The strength development characteristics of GGBFS concretes with a water-binder ratio of 0.38.

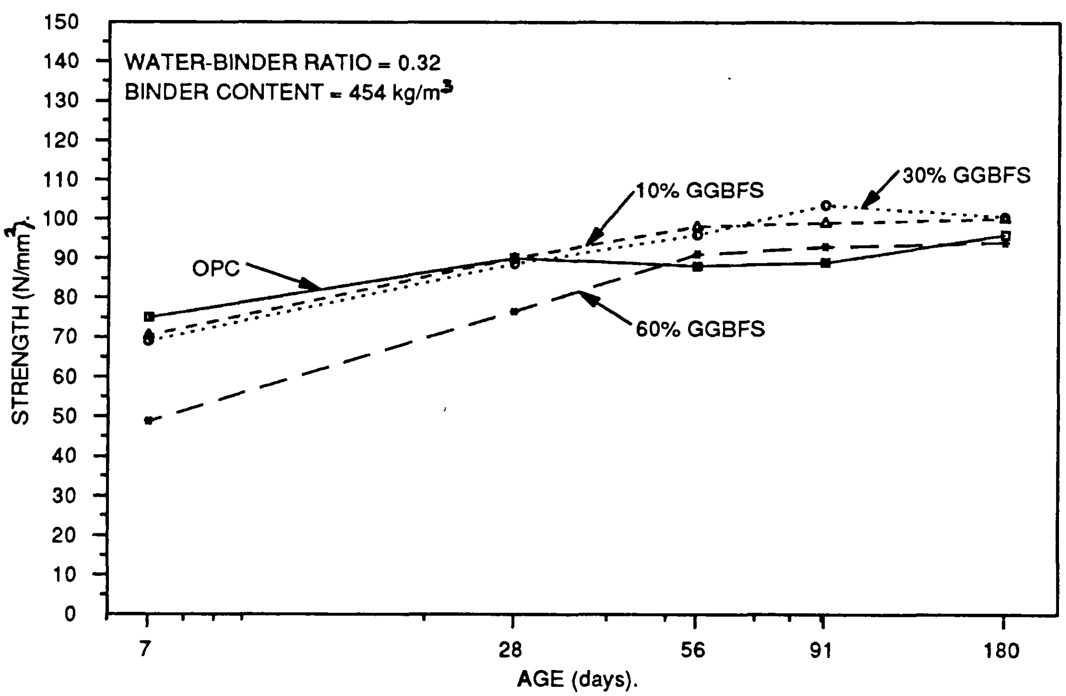


Figure 9.15: The strength development characteristics of GGBFS concretes with a water-binder ratio of 0.32.

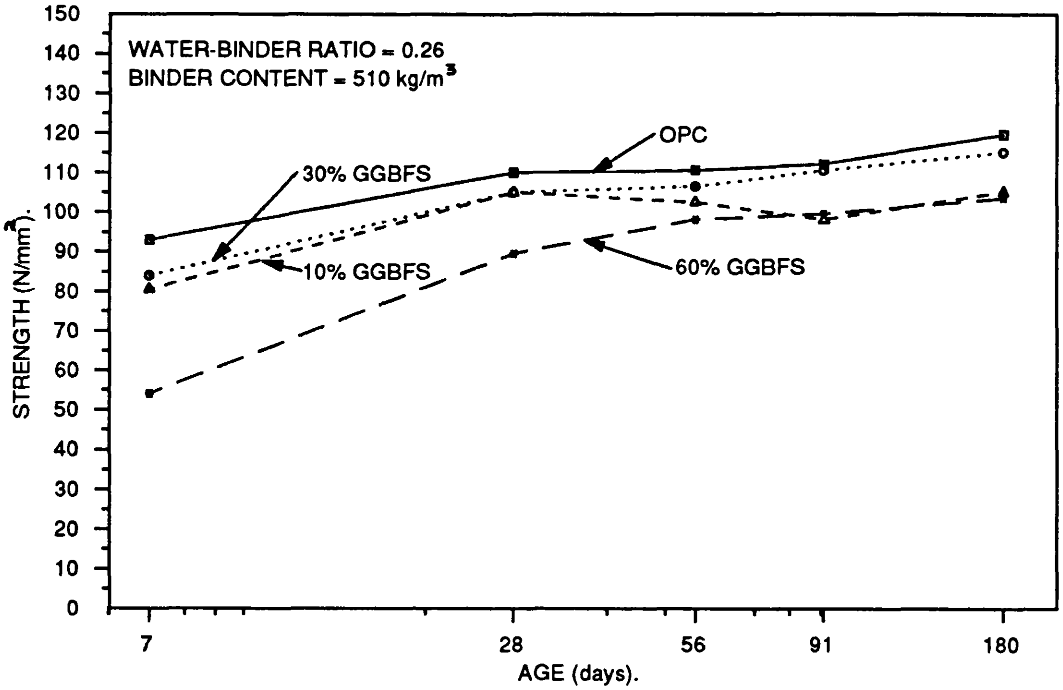


Figure 9.16: The strength development characteristics of GGBFS concretes with a water-binder ratio of 0.26.

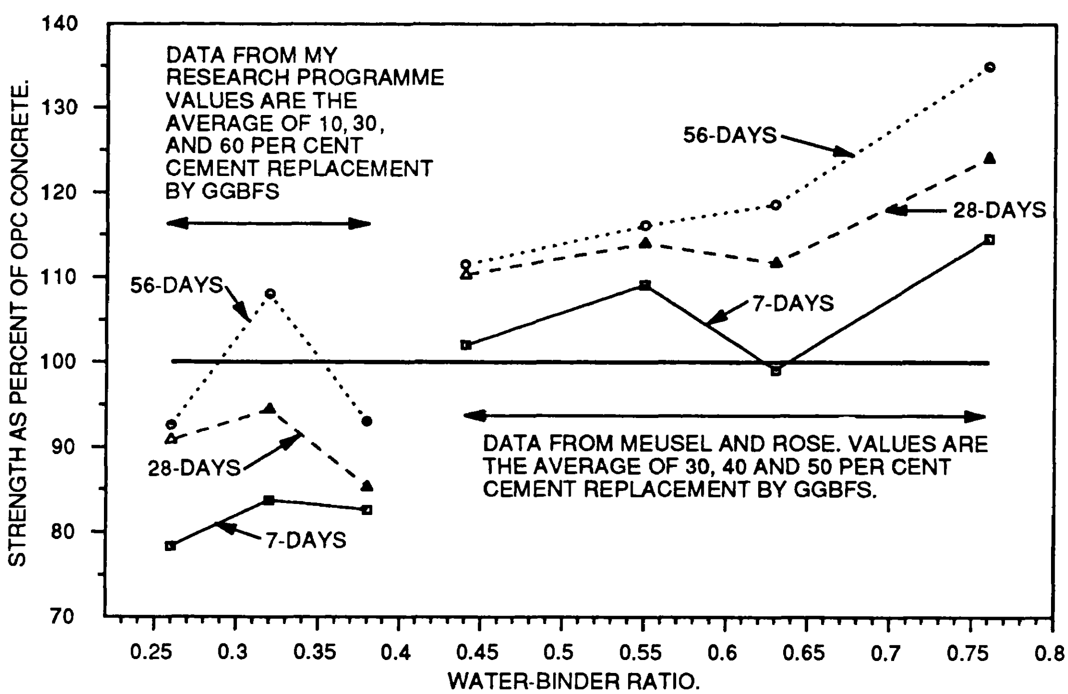


Figure 9.17: The effect of water-binder ratio on the compressive strength of GGBFS concrete.

PFA concrete, does not equal the compressive strength of OPC concrete even after 180-days. The performance of GGBFS in concrete is, therefore, shown to be affected by the water-binder ratio of the concrete. Meusel and Rose's relation[162] of the effect of water-binder ratio on compressive strength of GGBFS concrete is shown in Figure 9.17 and using data from my research programme, the relation has been extended down to a water-binder ratio of 0.26. Fulton and Malhotra[166] have also reported that the percentage of strength gain achieved with GGBFS and pelletized blast-furnace slag, respectively, is greater in concrete mixtures which have high water-binder ratios. However, neither of them gave any explanations for this trend.

9.3.4 Condensed silica fume (CSF).

The same three water-binder ratios, i.e. 0.38, 0.32 and 0.26, as have been used for PFA and GGBFS concretes, were also chosen in order to study the effects of partially replacing OPC with CSF on the strength development of concrete up to 180-days. In order to define more clearly the performance of CSF in concrete two additional water-binder ratios, i.e. 0.29 and 0.23, were also studied.

The strength development characteristics of concretes with OPC partially replaced by CSF at levels of 5, 10 and 15 per cent are shown in Figures 9.18 to 9.22. It is seen from these figures that the compressive strength of concrete incorporating CSF increased compared to the OPC mix. The increase in strength achieved by higher levels of cement replacement by CSF reaches an approximate ceiling level at about 10 per cent, (Figures 9.23 to 9.27). Increasing the replacement level from 10 to 15 per cent results in very small strength improvement. For this reason the compressive strength data of concrete with 10 per cent CSF have been used to show the general shape of the curve for strength versus the water-binder ratio, (see Figure 9.28). This also includes data from Sellevold and Radjy[172], after having been modified to consider CSF as partial replacement of cement and not as an additive. Their data, determined for water-binder ratios higher than 0.45, show that the shape of the 28-day compressive strength versus water-binder ratio curve for 8 per cent CSF level is the same as that for reference concrete, but it shifts to a substantially higher

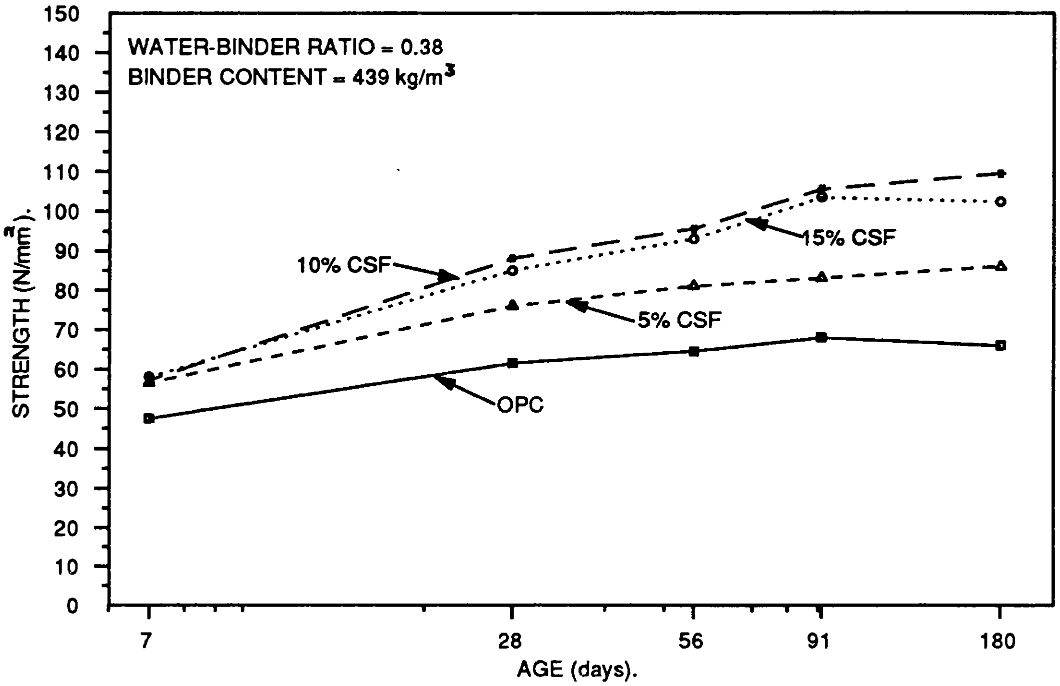


Figure 9.18: The strength development characteristics of CSF concretes with a water-binder ratio of 0.38.

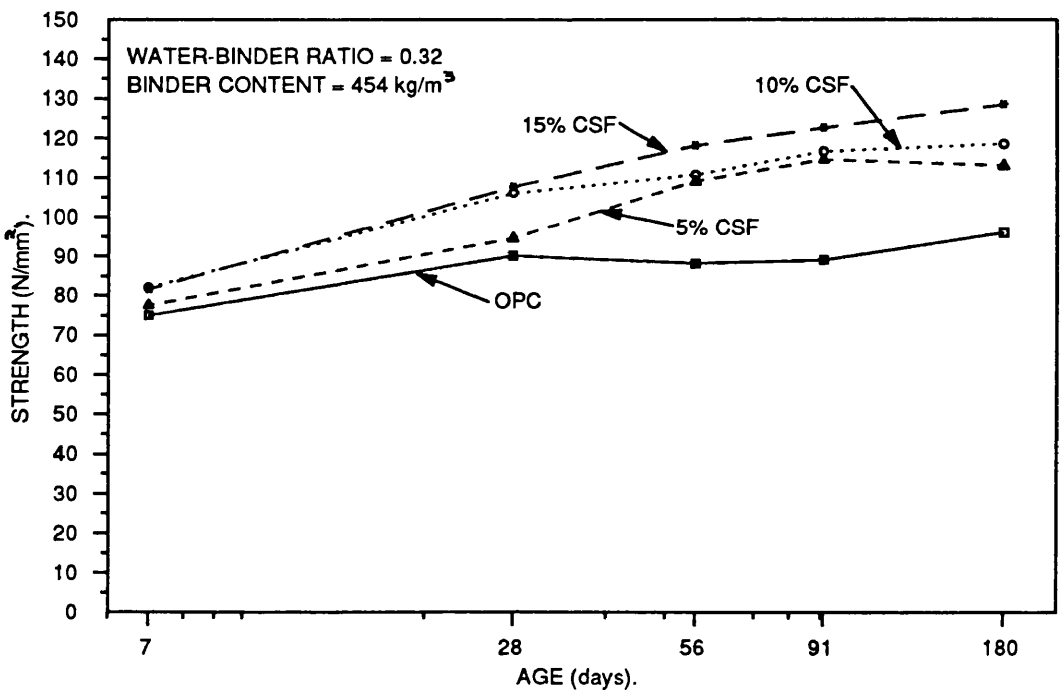


Figure 9.19: The strength development characteristics of CSF concretes with a water-binder ratio of 0.32.

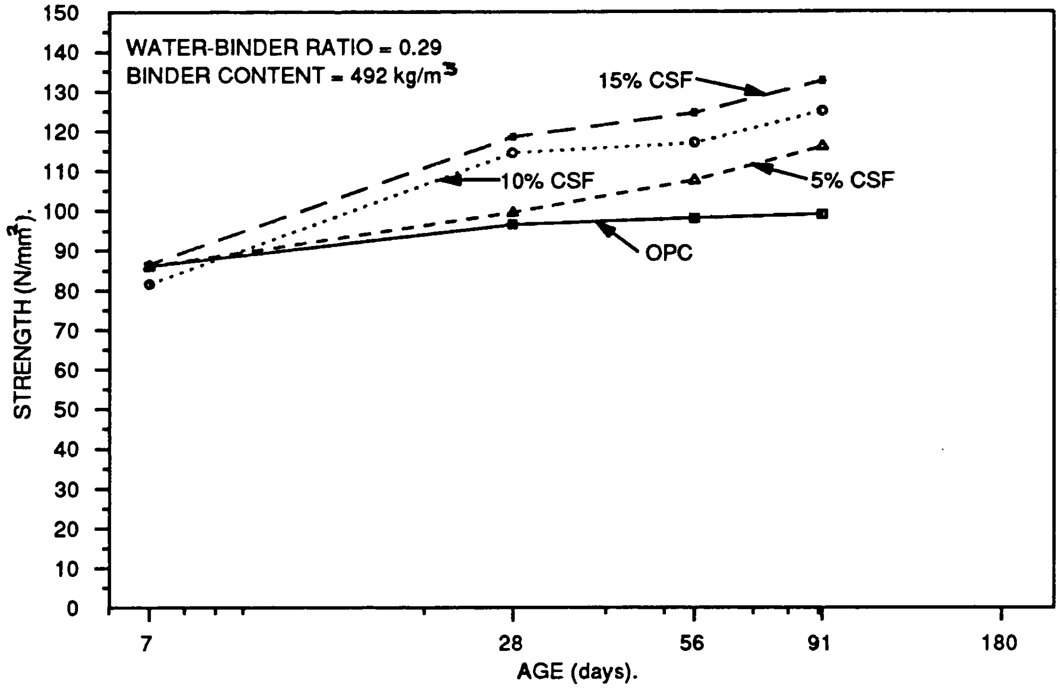


Figure 9.20: The strength development characteristics of CSF concretes with a water-binder ratio of 0.29.

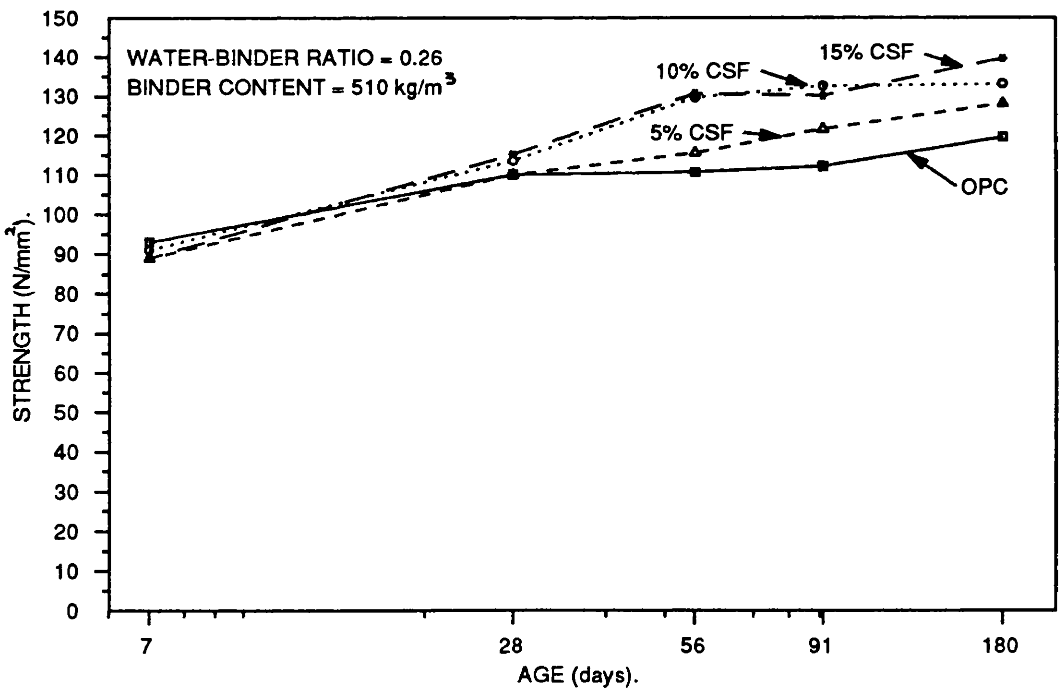


Figure 9.21: The strength development characteristics of CSF concretes with a water-binder ratio of 0.26.

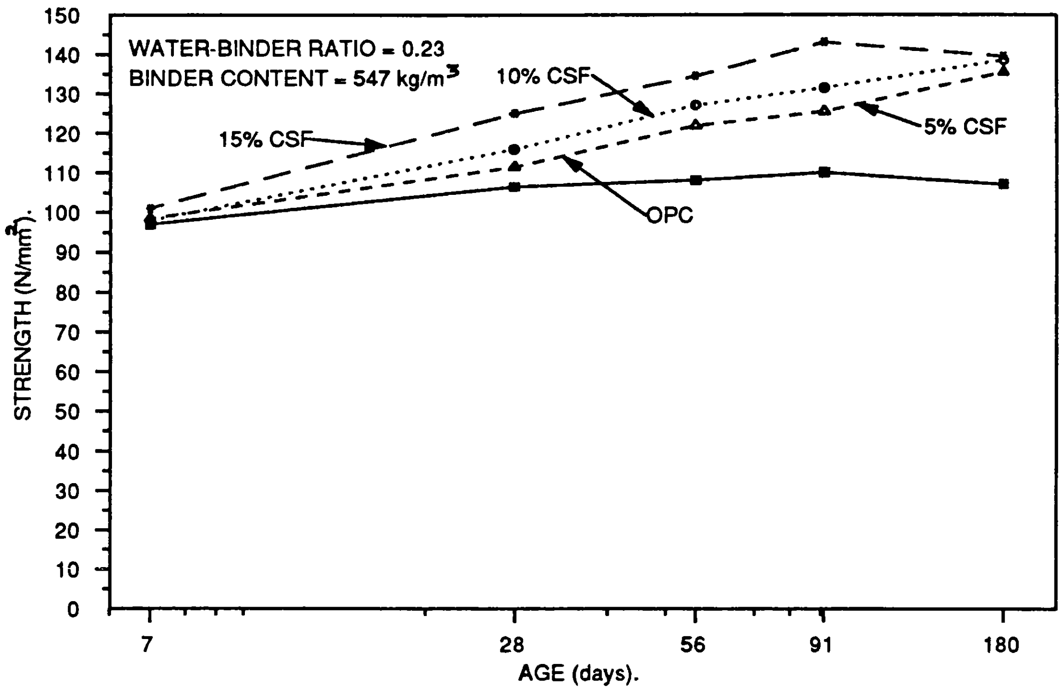


Figure 9.22: The strength development characteristics of CSF concretes with a water-binder ratio of 0.23.

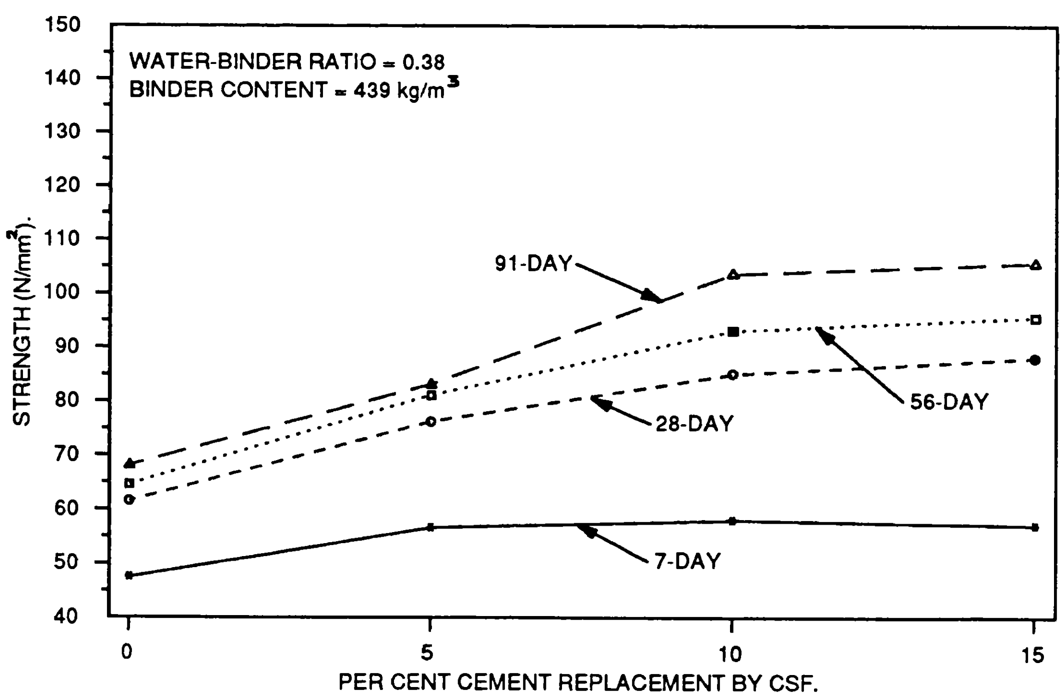


Figure 9.23: The increase in strength achieved by 5, 10 and 15 per cent cement replacement by CSF at a water-binder ratio of 0.38.

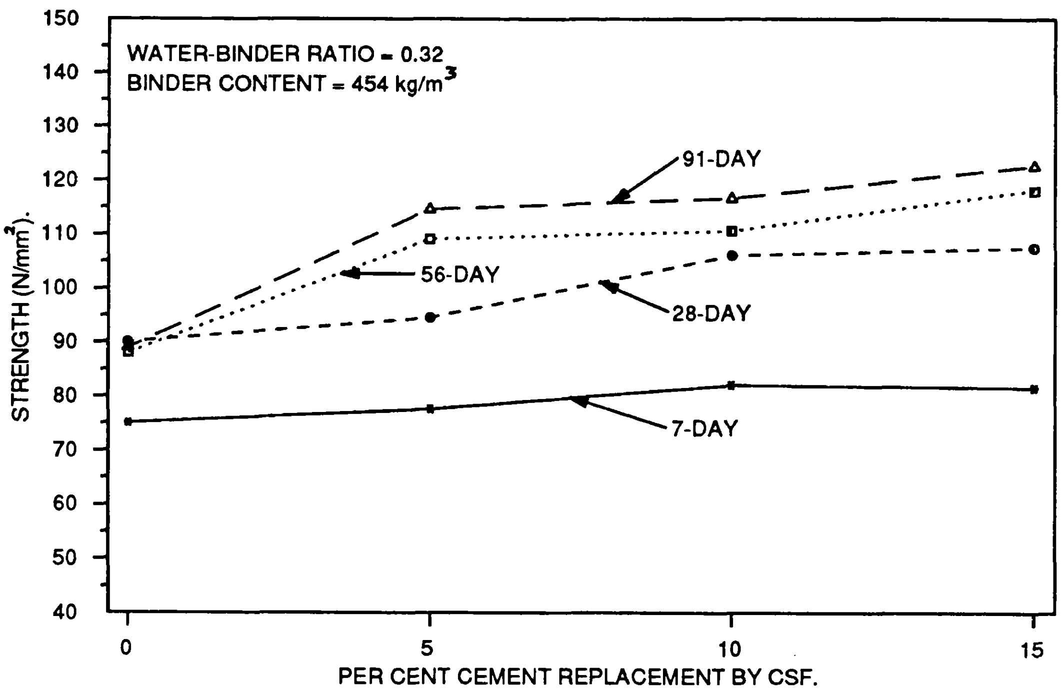


Figure 9.24: The increase in strength achieved by 5, 10 and 15 per cent cement replacement by CSF at a water-binder ratio of 0.32.

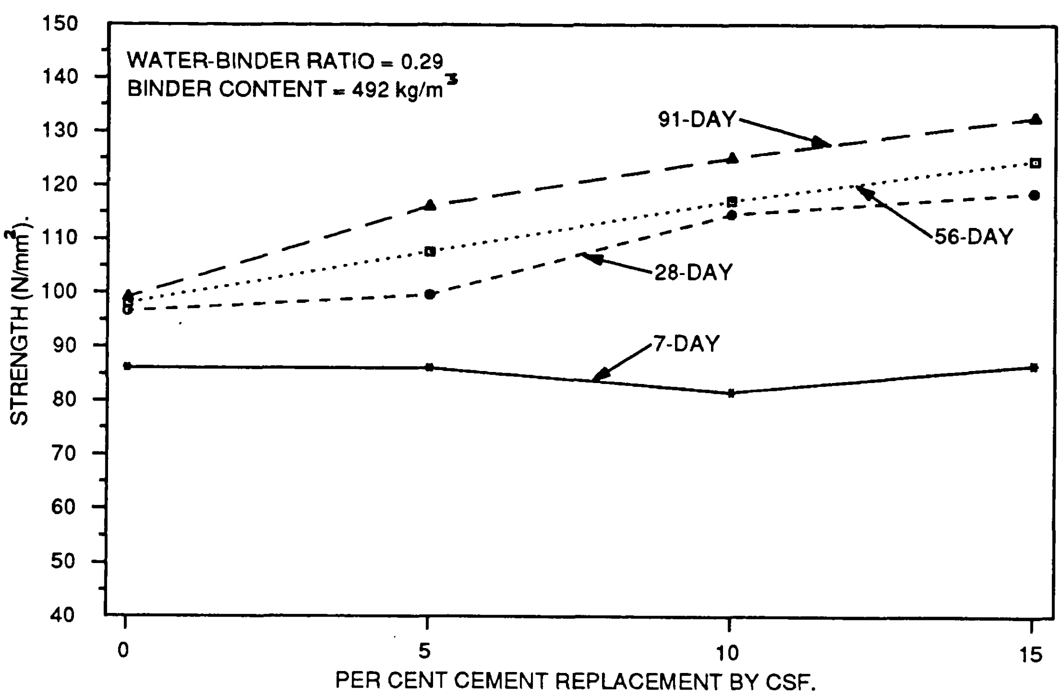


Figure 9.25: The increase in strength achieved by 5, 10 and 15 per cent cement replacement by CSF at a water-binder ratio of 0.29.

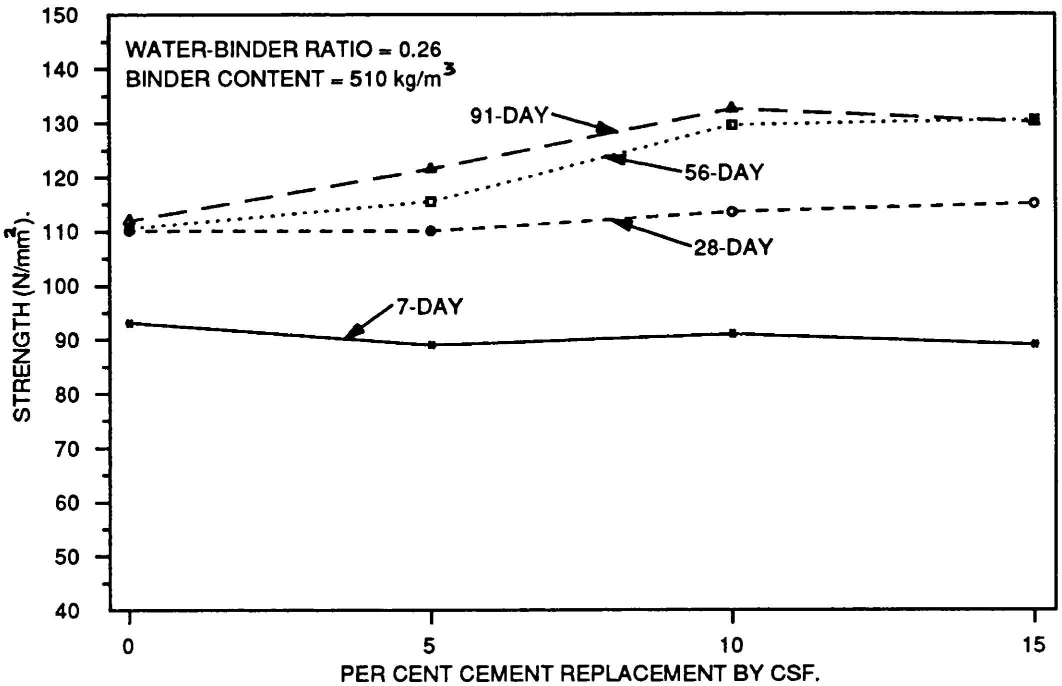


Figure 9.26: The increase in strength achieved by 5, 10 and 15 per cent cement replacement by CSF at a water-binder ratio of 0.26.

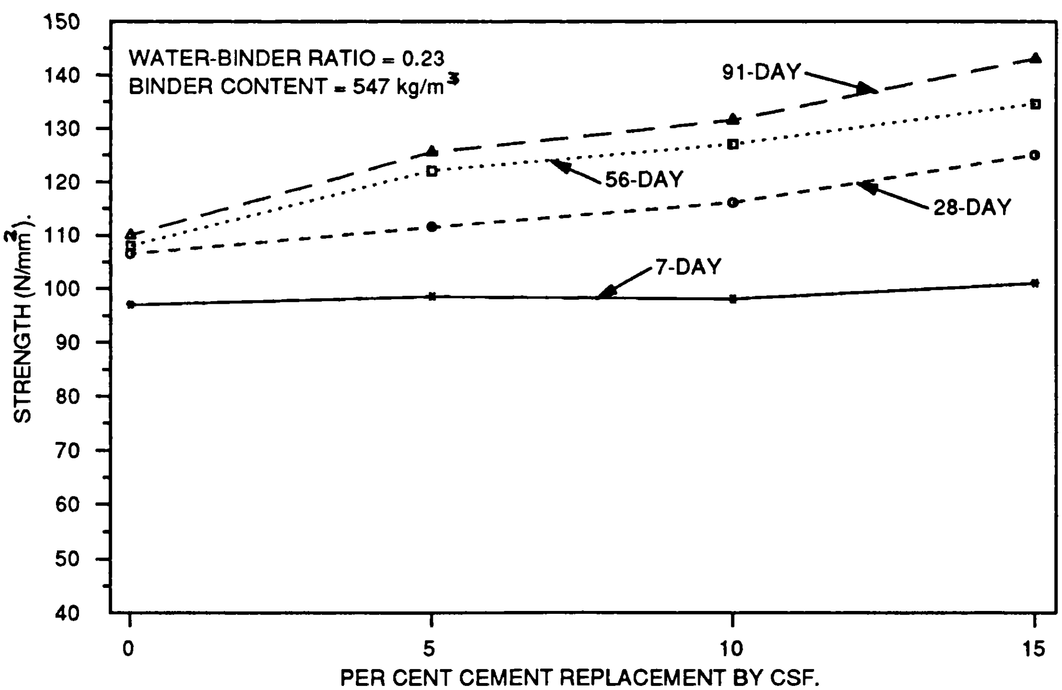


Figure 9.27: The increase in strength achieved by 5, 10 and 15 per cent cement replacement by CSF at a water-binder ratio of 0.23.

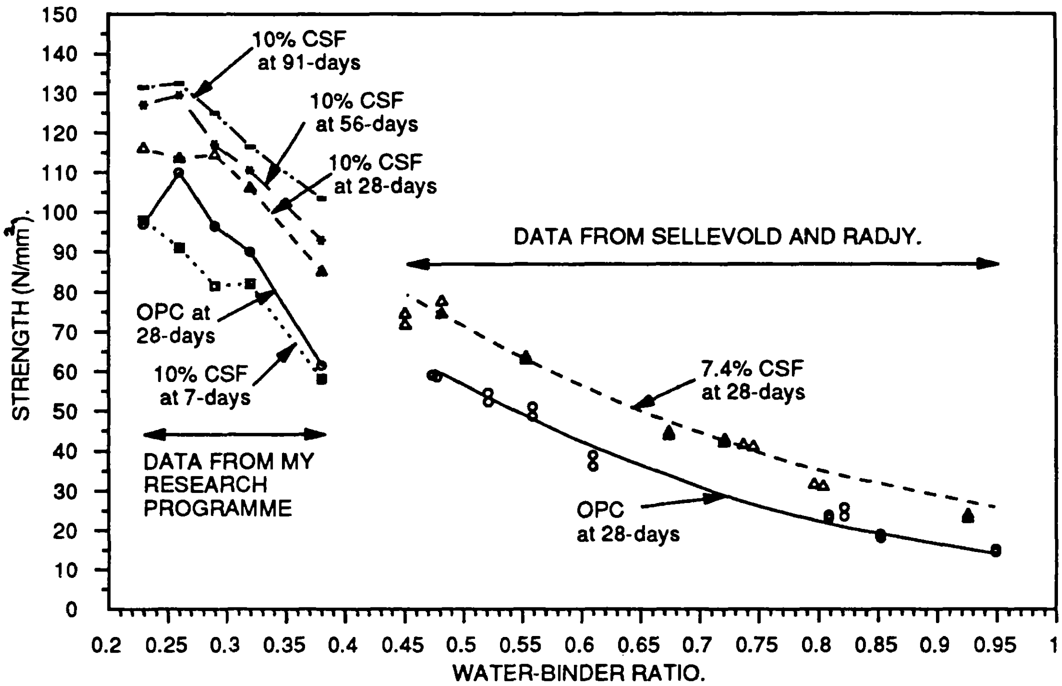


Figure 9.28: Compressive strength versus water-binder ratio for OPC and CSF concretes.

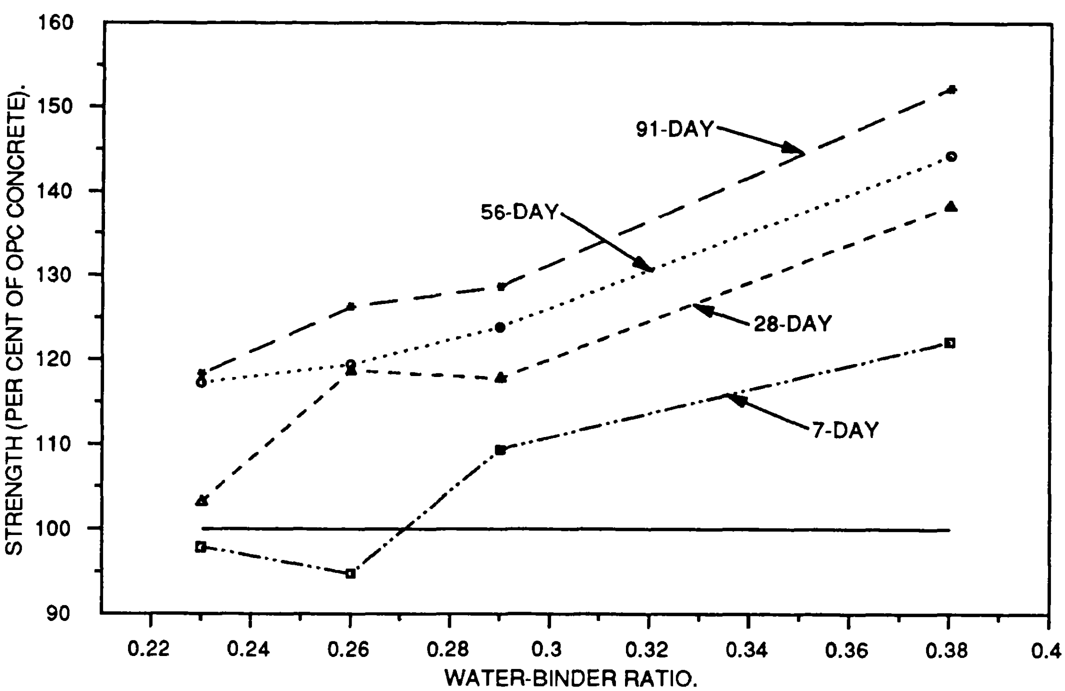


Figure 9.29: The effect of water-binder ratio on the compressive strength of CSF concretes.

level. My data, however, show that the 28-day strength difference between OPC concrete and CSF concrete is reduced at lower water-binder ratios. The 28-day compressive strength improvement obtained by incorporating 10 per cent CSF at a water-binder ratio of 0.26 is only 3.2 per cent.

Yogendran et al[169] have also reported strength increases of concrete incorporating CSF at a water-binder ratio of 0.34, while it was not possible to increase the strength of the concrete by incorporating CSF, even at the age of 91-days, at the 0.28 water-binder ratio, (see page 145). In contradiction to their results significant strength increases have been obtained after 28-days, as shown in Figure 9.28. This is also surprising as ACI Committee 226[167], Sellevold and Radjy[172], and Detwilder and Mehta[180] have reported that the main contribution of CSF to concrete strength development takes place from about 3 to 28-days and 7 to 28-days, respectively, (see pages 137-145).

Despite the significant increases in strength obtained after 28-days, the performance of CSF in concrete is, as shown in Figure 9.29, affected by the water-binder ratio of the concrete. This is in agreement with Malhotra and Carette[168] who stated that CSF performs more efficiently in superplasticised concretes having very high water-binder ratios. They advocated, however, that more supporting data were needed.

Possible reasons for the effect of water-binder ratio on the performance of CSF in concrete are:

1. Sarkar and Aitcin[191], as mentioned in Section 3.2.6, Part III, stated that the formation of $Ca(OH)_2$ in concrete of a low water-binder ratio is highly restricted due to the reduced amount of water available for it to form. This was substantiated by Cheng-yi and Feldman[186] who observed higher $Ca(OH)_2$ content per gramme of ignited cement at higher water-solids ratios for plain OPC mixes. This probably means there is more $Ca(OH)_2$ available for the pozzolanic reaction at higher water-solids ratio than lower ones. Indeed, Sarkar and Aitcin[191] using analytical and scanning electron microscopy have detected some CSF in a nearly intact state even after 28-days of curing and despite a low cement replacement level by CSF of 6

per cent. This was attributed to a delay in the dissolution of CSF in a concrete with a low water-binder ratio of 0.24, which has low $Ca(OH)_2$ content. This may also explain why the 28-day compressive strength improvement obtained by incorporating 10 per cent CSF at a water-binder ratio of 0.26 was only 3.2 per cent, while a significant strength increase was obtained at 56-days, as has been mentioned earlier.

2. The interfacial zone between the mortar and the coarse aggregate is an especially critical region in high strength concrete. It can be improved by:

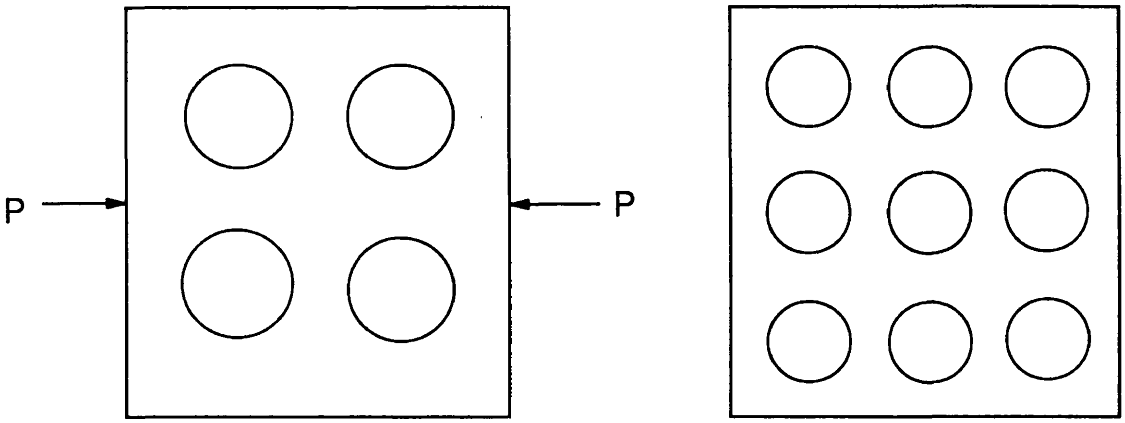
   a. lowering the water-binder ratio,

   b. increasing the surface roughness of the coarse aggregate, (see Section 3.2.4, Part II),

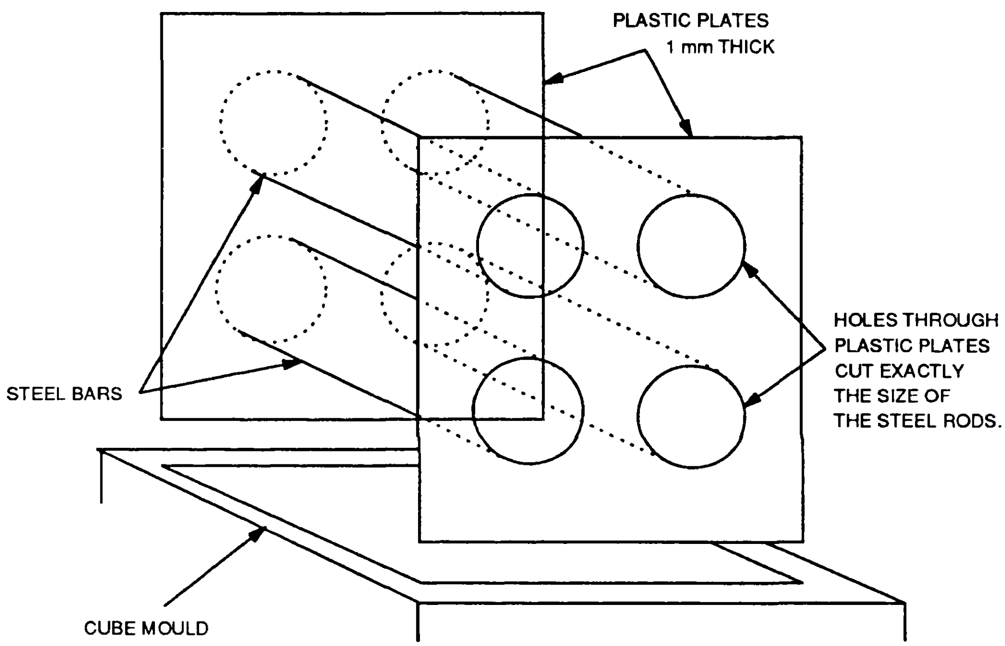
   and, c. the use of CSF to partially replace cement, (see Section 3.2.6, Part III).

Indeed, the use of CSF in concrete mixes at the 0.26 water-binder ratio and with gravel aggregate, have shown more significant strength improvements as compared to granite mixes, (see Figure 9.7).

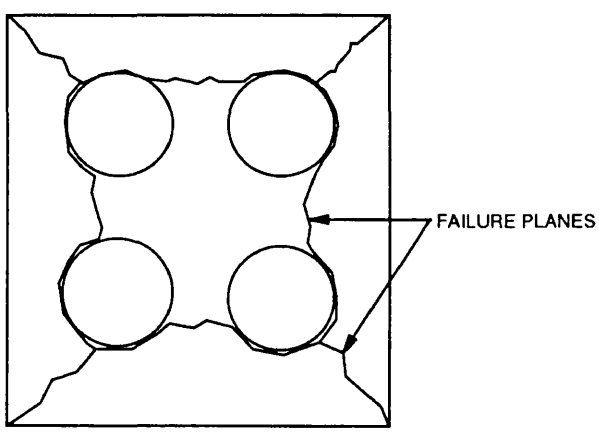
In order to investigate further the effect of surface roughness of the coarse aggregate on the contribution of CSF to the strength improvement, mortar cubes with steel bars embedded in the mortar were tested. The arrangements of the bars are as shown in Figure 9.30a. In order to investigate the effect of the size and spacing of the bars, two sizes, 25 and 16 mm, were used in arrangements of 4 and 9, respectively. Two plastic plates, 1 mm thick were used to hold the steel bars in position inside the mould as shown in Figure 9.30b. This arrangement meant that the edges of the steel bars were seen even after casting the mortar round the bars. The testing position of the cubes, with forces applied in the directions shown in Figure 9.30a, enabled the failure planes, if initiated round the steel bars, to be seen. Three mortars with water-binder ratio of 0.26 were used, i.e. an OPC mortar and mortars with 5 and 10 per cent cement replacement by CSF. Their mix proportions are shown in Table 9.1.



(a) Arrangement of the bars.



(b) Positioning the steel bars in the cube mould.



(c) Typical failure planes.

Figure 9.30: Setup of the steel bars embedded in mortar cubes.

Mix No. :	1.	2.	3.
Cement (kg/m³)	808	767.6	727.2
CSF (kg/m³)		40.4	80.8
% cement replacement		5	10
Total binder (kg/m³)		808	808
Sand (kg/m³)	1298	1298	1298
Free water (kg/m³)	210	210	210
Total water (kg/m³)	232	232	232
Free water-binder ratio	0.26	0.26	0.26
Superplasticiser dosage (%)	2.0	1.5	1.0
Flow (%)	105	140	120

Table 9.1: Mix proportions of the mortars tested.

	Mortar cube (N/mm²	Mortar with 9 bars (N/mm²)	Mortar with 4 bars (N/mm²)	% increase in strength for mortar with 9 bars	% increase in strength for mortar with 4 bars
OPC	78.9	85.4	82.1	8.24	4.06
5 % CSF	80.0	98.0	103.1	22.50	28.88
10 % CSF	89.9	94.2	97.5	4.78	8.45

Table 9.2: 28-day compressive strengths of mortar cubes with and without steel bars.

The failure plane of the mortar cubes was initiated at the interface of the steel bars and then propagated towards the other bars, or towards the edges of the cubes, as shown in Figure 9.30c. The failures were explosive and most of the steel bars usually became loose. The corresponding strengths of the mortar cubes with and without steel bars are shown in Table 9.2. It can be seen that although the use of CSF has improved the strength of the mortar cubes, it has, particularly at 5 per cent cement replacement, improved much more the strength of the mortar cubes with steel bars, and has resulted in 28-day strengths of around 100 N/mm^2.

These tests therefore confirm the findings of Cheng-yi and Feldman[186], Scrivener et al[187,188], and Goldman and Bentur[189], who all showed that the major influence of CSF in improving the strength of concretes stems from its effect on the transition zone, leading to improved aggregate-matrix bond, (see Section 3.2.6, Part III). It appears however that by lowering the water-binder ratio and increasing the surface roughness of the coarse aggregate by using crushed granite, the aggregate-matrix bond is improved so that the relative contribution of CSF in improving the strength of the concrete is reduced.

9.3.5 Combination of mineral admixtures.

It has been shown in the previous sections that high levels of cement replacement by GGBFS or PFA result in much lower strengths than OPC concrete even after 180-days. Yet, these high levels of cement replacement are beneficial in:

1) increasing the workability of the fresh concrete without the requirement of an excessive dosage of superplasticiser, as has been shown in Chapter 7,

and, 2) in reducing the adiabatic temperature rise of concrete, as has been shown in Chapter 8.

This section reports on the contribution of CSF to the improvement of the compressive strength of mixes with high levels of cement replacement by PFA or GGBFS.

Two water-binder ratios, i.e. 0.38 and 0.26, have been chosen to correspond to mixes described in the previous sections. Also, the improvement of workability resulting from the use of CSF in combination with PFA or GGBFS, (see Section 7.3.2), has made it possible to produce flowing concrete with a very low water-binder ratio of 0.20, which was also studied. The variables studied are shown in Table 9.3.

The strength development characteristics of the mixes are shown in Figures 9.31 to 9.35. As has been noted for OPC-CSF mixes the strength contribution of CSF is again higher at the higher water-binder ratio of 0.38, e.g., Figure 9.33 shows that the OPC-CSF-GGBFS mix has a higher strength than the OPC mix at 56-days and comparable strength to the CSF concrete at 91-days. Incorporation of CSF in mixes with water-binder ratio of 0.26 has improved the strength of mixes with high levels of PFA and GGBFS producing comparable strengths to OPC concrete at the age of 56-days, (Figures 9.32 and 9.34).

Figure 9.35 shows the strength development of mixes with a 0.20 water-binder ratio. Mixes with as high as 50 per cent GGBFS and 10 per cent CSF produced comparable strengths to CSF concrete (118.0 N/mm^2) from the age of 28-days and later. The OPC-PFA-CSF mix exhibited lower strengths than the mix with GGBFS but otherwise similar proportions of OPC and CSF.

9.3.6 Maximum size of aggregate.

The strength improvements achieved by redesigning the mixes according to the modified version of the "Maximum Density Theory" led to the re-investigation of the effect that the maximum size of aggregate has on strength. 20 mm granite aggregate mixes with three water-binder ratios, i.e. 0.38, 0.32 and 0.26, with and without 10 per cent cement replacement by CSF, were studied. The OPC and CSF concrete strength results of the 20 mm mixes are compared to those of the 10 mm mixes in Figures 9.36 and 9.37, respectively.

Surprisingly, at the water-binder ratio of 0.38, mixes with 20 mm maximum size of aggregate produced higher strengths than the mixes with 10 mm aggregate for both OPC

Mix No.	Water-binder ratio.	Pozzolan Type.	Percentage Substitution.
1.	0.38	GGBFS/CSF	57/5
2.	0.38	PFA/CSF	38/5
3.	0.26	GGBFS/CSF	58/5
4.	0.26	GGBFS/CSF	54/10
5.	0.26	PFA/CSF	36/10
6.	0.20	CSF	10
7.	0.20	GGBFS/CSF	50/10
8.	0.20	GGBFS/CSF	30/10
9.	0.20	PFA/CSF	30/10

Table 9.3: Mixes with CSF in combination with PFA or GGBFS.

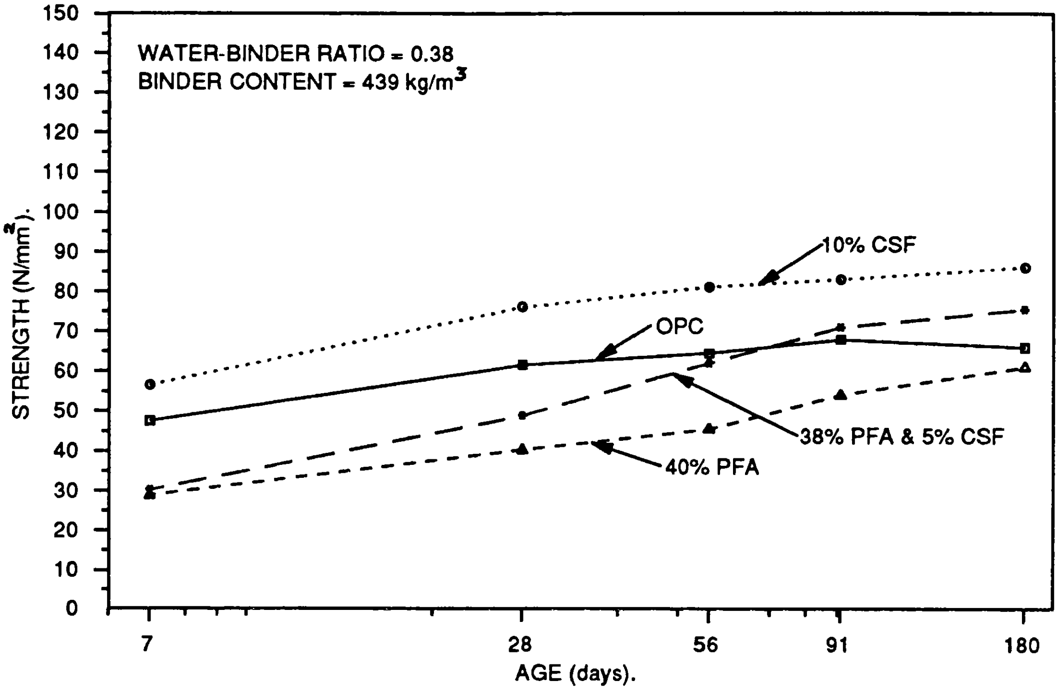


Figure 9.31: The contribution of CSF to the improvement of the compressive strength of a PFA mix with water-binder ratio of 0.38.

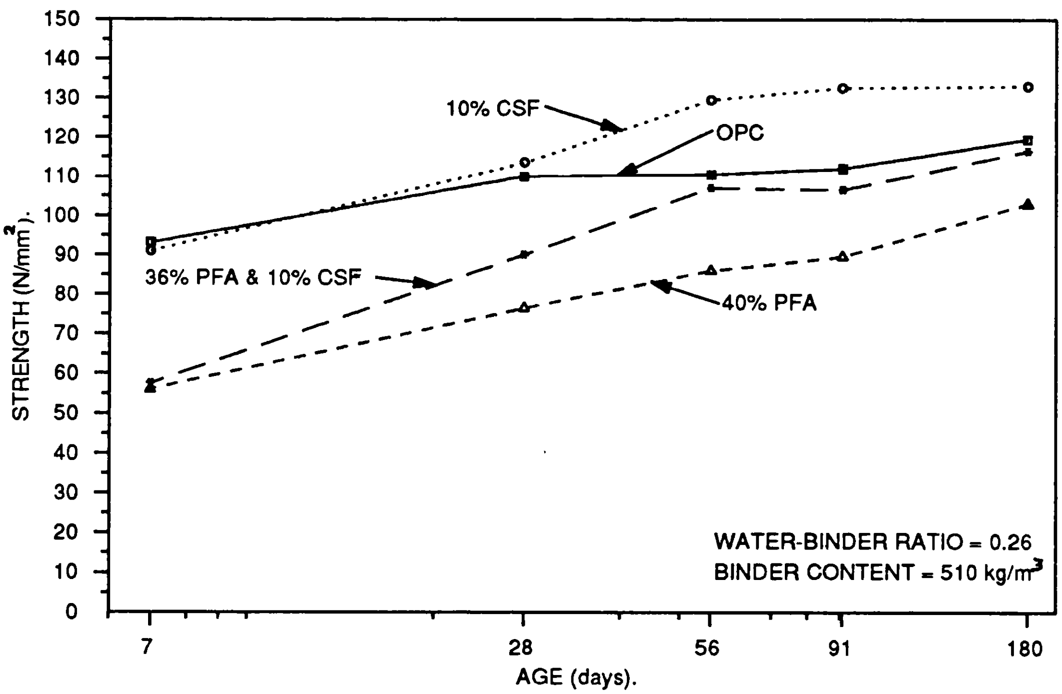


Figure 9.32: The contribution of CSF to the improvement of the compressive strength of a PFA mix with water-binder ratio of 0.26.

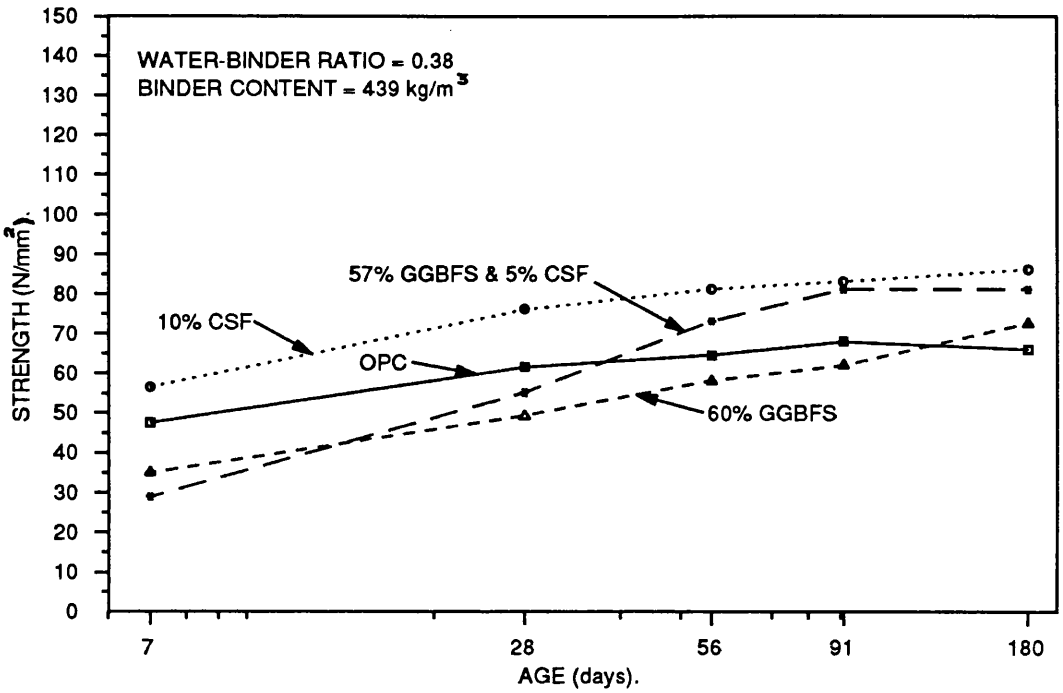


Figure 9.33: The contribution of CSF to the improvement of the compressive strength of a GGBFS mix with water-binder ratio of 0.38.

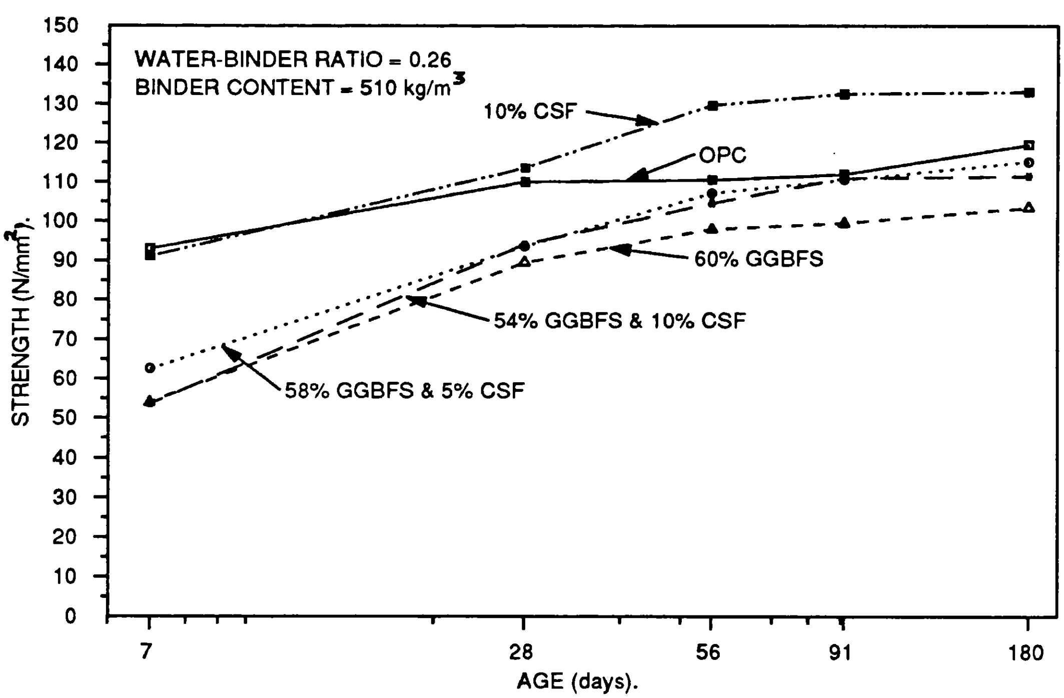


Figure 9.34: The contribution of CSF to the improvement of the compressive strength of GGBFS mixes with water-binder ratio of 0.26.

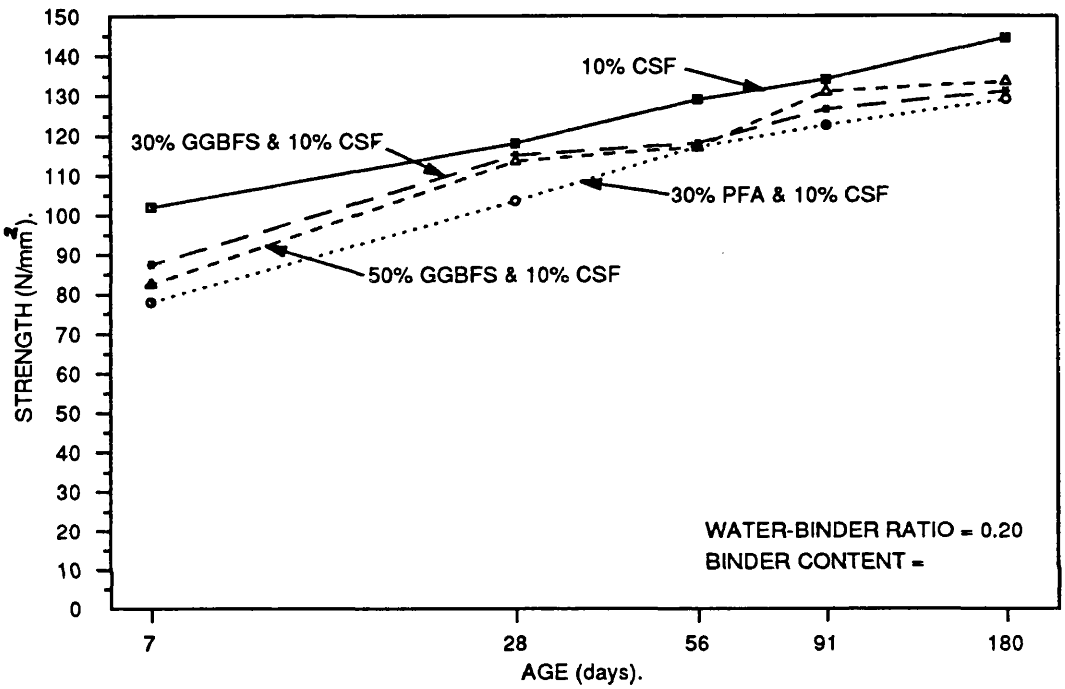


Figure 9.35: Strength development of mixes with a 0.20 water-binder ratio.

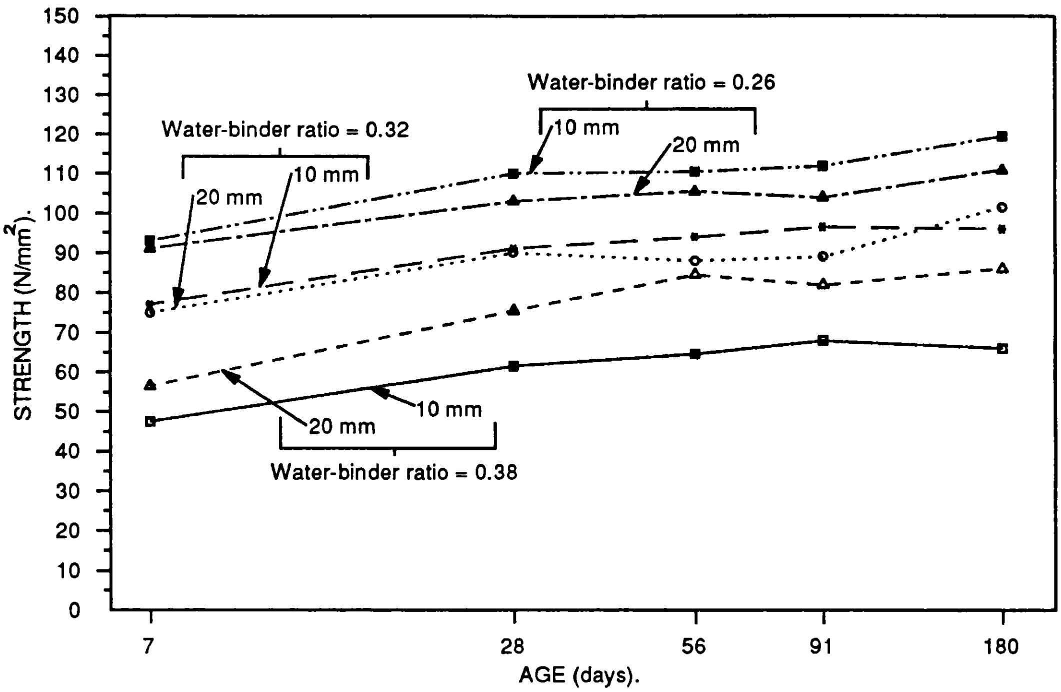


Figure 9.36: Strength development characteristics of OPC mixes with 20 and 10 mm maximum size of granite aggregate.

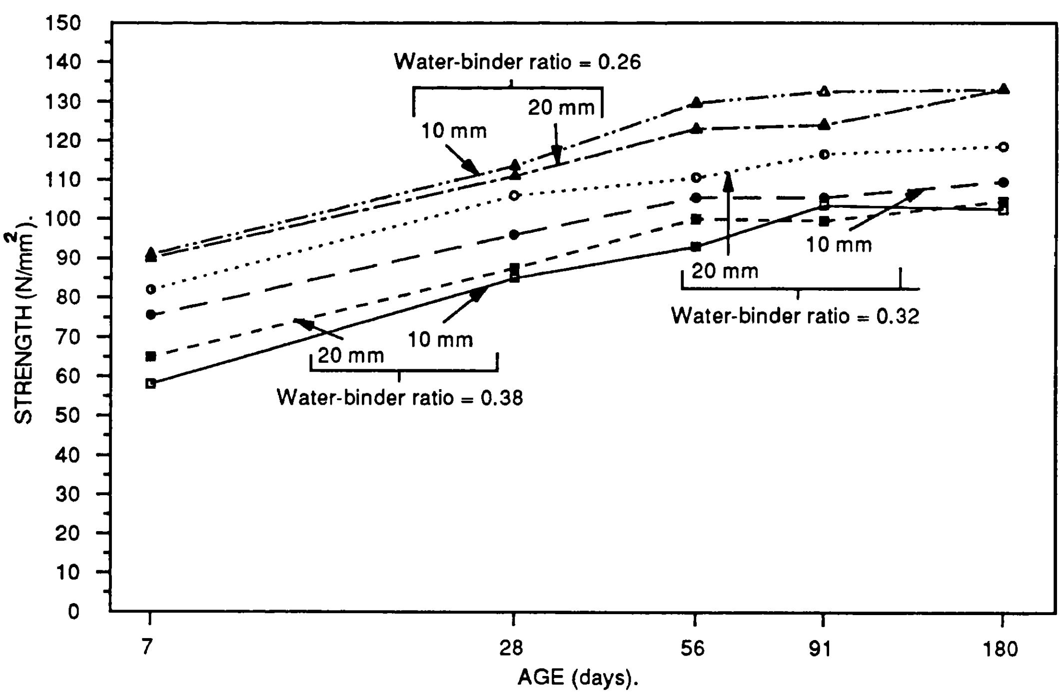


Figure 9.37: Strength development characteristics of CSF mixes with 20 and 10 mm maximum size of granite aggregate.

and CSF concretes. At the lower water-binder ratios of 0.32 and 0.26, mixes with 20 mm aggregate produced slightly lower strengths than the corresponding mixes with 10 mm aggregate. The reduction in the 28-day strength for a water-binder ratio of 0.26 is 6.4 per cent for OPC concrete and only 2.2 per cent for CSF concrete. The maximum size of the coarse aggregate has, therefore, only a small influence on the compressive strength of concrete. This is in agreement with the findings of Bedard and Aitcin(105) but contrary to the findings of other authors(140,141,142), (see Section 3.2.4, Part III), who claimed that the compressive strength of concrete increased gradually as the maximum size of aggregate decreased. It may, therefore, be concluded that optimisation of mix proportions according to the "Modified Maximum Density Theory", the quality of the aggregates used and the incorporation of CSF, led to the reduction of this effect.

9.3.7 Fineness modulus of sand.

Use of a coarse sand with a fineness modulus of about 3.0 has generally been recommended for use in the production of high strength concrete. This has been recommended from a workability and not a strength point of view. As has been mentioned in Section 3.2.5, a sand with a fineness modulus below 2.5 results in a "stickier", less workable fresh concrete with a greater water demand. The greater water demand of finer sands can, however, be counteracted in the production of high strength concrete by increasing the dosage of superplasticiser.

A coarser sand, with fineness modulus of 2.73 as compared to the 2.1 of the sand previously used, was used for mixes with three water-binder ratios, i.e. 0.38, 0.32 and 0.26, with and without 10 per cent cement replacement by CSF. The OPC and CSF concrete strength results of the coarse sand mixes are compared with those of the fine sand (FM = 2.1) mixes in Figures 9.38 and 9.39, respectively.

The use of the coarser sand resulted in considerably higher strengths for the OPC concrete with a 0.38 water-cement ratio. Surprisingly, the corresponding CSF concrete showed comparable strengths to that with finer sand. An improved paste-mortar or mortar-

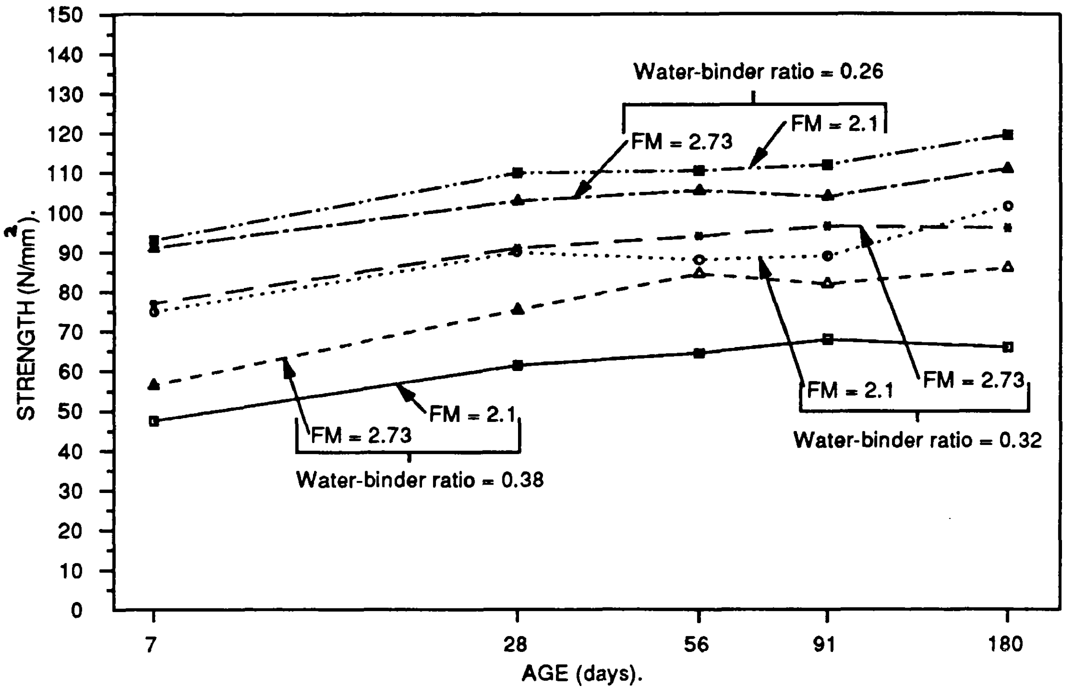


Figure 9.38: Strength development characteristics of OPC mixes with coarse and fine sands, FM = 2.73 and 2.1, respectively.

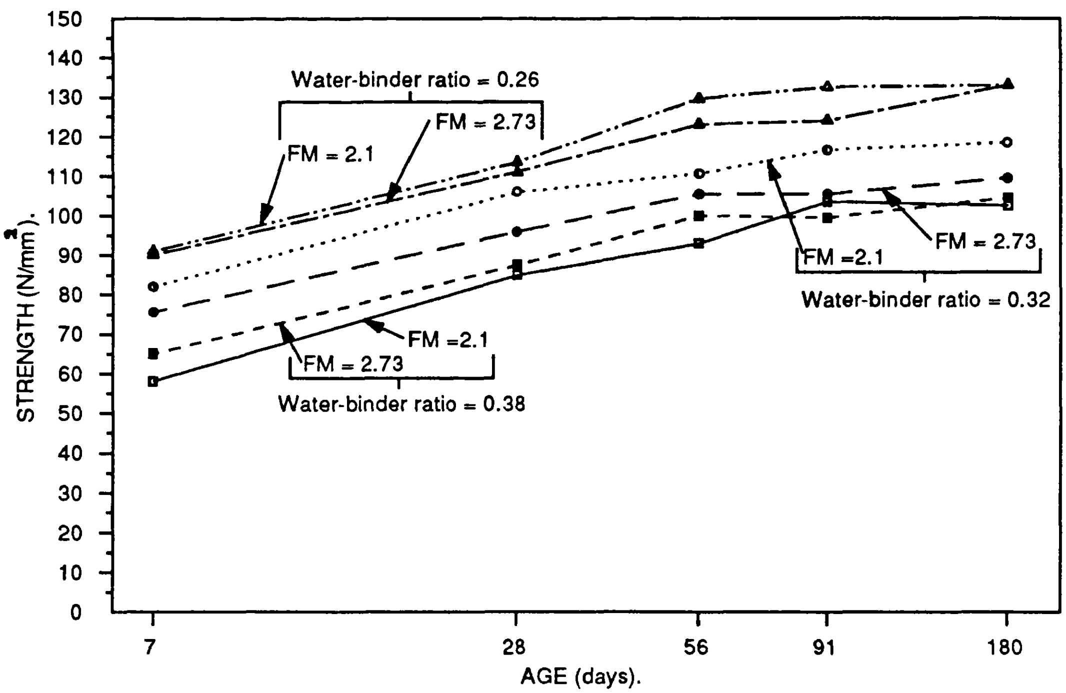


Figure 9.39: Strength development characteristics of CSF mixes with coarse and fine sands, FM = 2.73 and 2.1, respectively.

aggregate bond is a possible explanation for the higher strength of the OPC mix with the coarser sand. The improvement is not apparent in CSF mixes as the use of CSF also improves the paste-mortar and mortar-aggregate bond. The bond can also be improved by decreasing the water-binder ratio. Comparable strengths between the coarse and fine sand were therefore obtained for a 0.32 water-binder ratio with and without CSF.

Although the coarse and fine sand mixes, with a 0.26 water-binder ratio, had comparable strengths at 7-days, the coarser sand mixes had lower strengths thereafter. The failure planes of these concretes, with compressive strengths above 100 N/mm^2, passed through the coarser particles of sand.

9.4 CONCLUSIONS.

The high levels of cement replacement by PFA and GGBFS, e.g., 40 and 60 per cent, respectively, that were shown in Chapter 8 to be required for significant temperature reduction also cause a significant reduction in the compressive strength. Combinations of these with 10 per cent CSF were found to offset this; they reduced the temperature rise by more than 10^0C while the reduction in the 28-day compressive strength was less than 15 per cent.

A more complete list of the principle results from this as well as from the previous experimental chapters is included in the next chapter.

CHAPTER 10.

CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER WORK.

10.1 INTRODUCTION.

The experimental programme consisted of four main series of tests as follows:

(i) the development of an improved proportioning methodology,
(ii) a rheological study of high strength concrete,
(iii) measurement of adiabatic temperature during hydration, and
(iv) measurement of compressive strength up to 180-days.

The conclusions from these investigations are first presented followed by recommendations for further work.

10.2 CONCLUSIONS.

The objectives stated in Chapter 4 have generally been achieved and the principle results from each of the four main series of tests are listed below:

(i) Mix design of high strength concrete.

1. The "Maximum Density Theory", although it provides proportions of aggregates with minimum void content, produces a cohesive concrete requiring either a high dosage of superplasticiser, or a high percentage of overfill by paste to make a concrete with a high slump, (Figure 6.16). This arises from the fact that this theory considers only the void content of the combination of aggregates but not the effect their surface area has on the requirement of excess paste to lubricate their surfaces.

2. The void content graphs, used to determine the combination of aggregates with minimum void content, (Figures 6.4 to 6.7), can be used to investigate the combined effect of void content and surface area of the aggregate on the required quantity of paste to produce a concrete with a specified slump. This investigation leads to the optimisation of concrete mix proportions, i.e. determination of the relative proportions of the aggregates for a minimum required volume of cement paste. This procedure, since it is an extension of the "Maximum Density Theory", has been called the "Modified Maximum Density Theory". The necessary steps required for the optimisation of the concrete mix proportions can be outlined as follows:

   (i) The measurement of the aggregate void content with different proportions of sand, (page 248).

   (ii) The investigation of the combined effect of void content and surface area of the aggregate on the amount of excess paste required to "fluidify" the concrete. This requires trial mixes with lower sand proportions than that required for minimum void content. These are tested for slump and a graph of slump versus overfill may be plotted, (e.g., Figure 6.17).

   (iii) From the above graph, the sand proportion that produces the required slump with the minimum cement content can then be determined, (e.g., Figure 6.18).

Using this procedure, the optimum mix proportions of coarse and fine aggregate have been found not to be affected by a change in the water-cement ratio, (Figures 6.18, 6.20 and 6.22). Factors that may affect the optimum proportion of sand are:

(i) *Surface area of binder.* The optimum sand proportion has been found to increase from 37.5 per cent for OPC mixes to 40 per cent for mixes where 10 per cent CSF has been used to partially replace cement, (Figure 6.25). This is due to the increased surface area of the binder, for a constant amount of paste, resulting from the use of CSF.

(ii) *Fineness modulus of sand.* The use of a sand with fineness modulus of 2.73 instead of 2.1 resulted in the optimum sand proportion to be very close to the proportion required for minimum void content, e.g., with the materials used in my research programme the optimum sand proportion

was 42.5 per cent, (Figure 6.30), as compared to 45 per cent for minimum void content, (Figure 6.7).

(ii) Workability of high strength concrete.

1. Partial cement replacement by PFA or GGBFS causes a reduction in the amount of superplasticiser dosage required for a given slump from that required for an equivalent OPC concrete, (Figures 7.3 and 7.4). The reduction resulting from 10, 20 and 40 per cent cement replacement levels by PFA correspond to 15, 30, and 60 per cent replacement levels by GGBFS. Advantage can be taken of the improved workability to either reduce the superplasticiser dosage or reduce the water-binder ratio used in concrete and yet to maintain the same workability as the OPC concrete.

2. The use of CSF in concrete reduces the workability at low superplasticiser dosages and therefore requires a further addition of superplasticiser to retain the same slump. However, above a certain superplasticiser dosage, believed to be that required to reduce the cohesion of CSF particles sufficiently to disperse them, CSF has a water-reducing effect, (Figure 7.5).

3. The decrease of workability, as defined by the slump test, resulting from the use of CSF at low superplasticiser dosages can be counteracted by the use of GGBFS which improves the workability. At higher superplasticiser dosages, when CSF particles are well dispersed and act as fillers, CSF has a water-reducing effect even for the GGBFS mixes, (Figure 7.6). Concrete with 50 per cent GGBFS in combination with 10 per cent CSF, water-binder ratio of 0.20 and a slump of 150 mm can be achieved with 1.3 per cent superplasticiser (Conplast 430) dosage.

4. Workability of high strength concrete declines rapidly with time after mixing, e.g., the 200 mm slump of an OPC concrete with a water-cement ratio of 0.26 was reduced to 65 mm in only 30 minutes. Partial cement replacement by PFA,

GGBFS or CSF does not significantly contribute to decreasing the workability loss, (Figure 7.7).

5. The ratio of superplasticiser adsorbed by OPC by weight, as determined spectrophotometrically, corresponds to the dosage beyond which the efficiency of the superplasticiser, i.e. in reducing the water-cement ratio while retaining the same slump, decreases significantly. Above the superplasticiser-OPC ratio required for the saturation point to be reached, no further reductions of the water-cement ratio can be achieved even by doubling the superplasticiser dosage, (page 308).

6. The yield (g) and plastic viscosity (h) values, as determined by Tattersall's two point test apparatus, increase by decreasing the water-binder ratio, despite the concrete having an equal slump of 150 mm.

7. Coarse sand mixes with mix proportions optimised according to the "Modified Maximum Density Theory", (the fineness modulus of the sand was 2.73 and the optimum proportion was 42.5 per cent), showed only slightly lower yield values (g) and almost identical viscosity (h) values to those of the fine sand mixes (the fineness modulus of the sand was 2.1 and the optimum proportion was 37.5 per cent), (Figure 7.11). This similarity of yield (g) and plastic viscosity (h) values, is a further indication that the "Modified Maximum Density Theory" takes into account the quantitative effect that the surface area of all the aggregate particles has on paste requirement and, therefore, proportions determined using this theory are the optimum.

8. Mixes with a lower fine sand content, i.e. 30 per cent compared to 37.5 per cent, showed a great reduction in the yield value (g) but again showed slightly higher plastic viscosity (h) values, (Figure 7.11).

9. The relative efficiencies of two superplasticisers, a naphthalene and a melamine based formaldehyde, in producing concrete with a slump of 150 mm appear to vary at different dosages. At superplasticiser dosages of 0.5 and 2.3 per cent solids

by weight of cement the two superplasticisers are equally efficient. At dosages between 0.5 and 2.3 per cent the melamine based superplasticiser appears to be slightly more efficient, (Figure 7.12). However, despite equal slumps obtained using the two types of superplasticisers, two point test results have shown that mixes with the melamine based superplasticiser have higher yield (g) and plastic viscosity (h) values, (Figure 7.13), i.e. it seems to be less efficient.

10. Partial cement replacement by PFA has the effect of reducing the yield (g) value of 150 mm slump concrete, the effect being more pronounced at the lower water-binder ratios, (Figure 7.14a). Its effect on the plastic viscosity (h) value is, however, to increase it, the effect being more pronounced at the higher water-binder ratios, (Figure 7.14b).

11. It appears that GGBFS has only a very small effect on the yield (g) and plastic viscosity (h) values of 150 mm slump concrete, (Figure 7.16).

12. Partial cement replacement by CSF reduces the yield (g) and plastic viscosity (h) values of 150 mm slump concrete.

13. The yield (g) and plastic viscosity (h) values of 150 mm slump concrete are influenced by, (page 330):
    (i) the shape, texture and total surface area of the aggregate,
    (ii) the richness of the mixture, i.e. not only the cementitious content but also its specific area.
    (iii) the water-cement or water-binder ratio, and
    (iv) the type of binder used, i.e. the rheological properties of the paste.

14. It appears that the yield (g) and plastic viscosity (h) values of 150 mm slump concrete may be influenced more by the richness of the mixture while compactibility, i.e. the response of fresh concrete to vibration, may be influenced more by the rheological properties of the paste, (pages 328 to 330).

(iii) Adiabatic temperature rise during hydration.

1. The adiabatic temperature rise for 100 kg of cement decreases at lower water-cement ratios, (page 337).

2. The effect of a higher casting temperature is to increase the initial rate of heat evolution but decrease the total heat evolution, (Figure 8.4).

3. PFA and GGBFS, used at low cement replacement levels, can have a potentially higher level of hydration than OPC, (Figures 8.9 and 8.13). High levels of cement replacement by PFA and GGBFS, e.g., 40 and 60 per cent, respectively, are required to reduce significantly the adiabatic temperature rise.

4. Partial cement replacement by CSF at the level of 10 per cent does not significantly affect the adiabatic temperature of concrete, (Figure 8.18).

5. (36% PFA - 10% CSF) and (54% GGBFS - 10% CSF) mixes, despite their lower cement contents as compared to 40 and 60 per cent PFA and GGBFS mixes, respectively, have a higher initial rate of hydration. However, they have lower peak temperature rise, (Figures 8.25 and 8.26).

(iv) Strength development of high strength concrete.

1. The compressive strength of OPC mixes may reach a ceiling value that cannot be increased by lowering the water-cement ratio. This is due to having reached the limit of the bonding potential of that cement-aggregate combination, i.e. no matter how much stronger the paste is made, failure is initiated in the transition zone. The potential of the cement-aggregate combination also depends on the mix proportions. The potential increased when the mixes were re-designed according to the modified version of the "Maximum Density Theory", (Figure 9.4).

2. The relation between the compressive strength and the crushing value, i.e. a decrease in the compressive strength with an increase in the crushing value of the aggregate, is affected by:
   (i) the mix design method used to optimise the mix proportions,
   (ii) the surface texture of the aggregates, and
   (iii) the use of CSF.

3. The performance of mineral admixtures (PFA, GGBFS and CSF) in concrete is affected by the water-binder ratio of the concrete. The percentage of strength, as compared to an OPC concrete, achieved with these mineral admixtures is greater in concrete mixtures which have high water-binder ratio. For example,
   (i) The strength of concretes with water-binder ratios of 0.38 and 0.32 and with levels of cement replacement by PFA of 10 and 20 per cent was equal or even higher than that of OPC concrete at 56-days, (Figures 9.8 and 9.9). However, for the low water-binder ratio of 0.26, PFA concrete did not equal the compressive strength of OPC concrete even after 180-days, (Figure 9.10).
   (ii) Concretes with water-binder ratios higher than 0.32 and with levels of cement replacement by GGBFS of up to 30 per cent exhibited 28-day compressive strengths equal to the OPC concrete and greater strengths beyond 28-days, (Figures 9.14 and 9.15). However, for the low water-binder ratio of 0.26, GGBFS concrete, even at the low cement replacement level by GGBFS of 10 per cent, did not equal the compressive strength of OPC concrete even after 180-days, (Figure 9.16).
   (iii) The compressive strength of CSF concrete was higher than the OPC concrete for all water-binder ratios. However, the 28-day compressive strength improvement obtained by the use of 10 per cent CSF in mixes with 10 mm granite was 38.2 per cent for the water-binder ratio of 0.38 while only 3.2 per cent for the water-binder ratio of 0.26, (Figure 9.29). The strength contribution of CSF is also dependent on the bonding potential of the cement-aggregate combination used, e.g., the contribution of 10 per cent CSF to the strength of gravel mixes with water-binder ratio

of 0.26 was 17.2 per cent as compared to the granite mixes which was only 3.2 per cent, (page 388 and Figure 9.7).

4. The high levels of cement replacement by PFA and GGBFS required for significant temperature reduction, e.g., 40 and 60 per cent, respectively, also cause a significant reduction in the compressive strength. Combinations of these with 10 per cent CSF were found to offset this; they reduced the temperature rise by more than 10^{0}C while the reduction in the 28-day compressive strength was less than 15 per cent, (Figures 9.31 to 9.34).

5. The improvement of workability resulting from the use of 10 per cent CSF in combination with 50 per cent GGBFS or 30 per cent PFA has made it possible to produce flowing concrete with a 0.20 water-binder ratio and 28-day compressive strengths of around 110 N/mm^2, (Figure 9.35).

6. Optimisation of mix proportions according to the modified version of the "Maximum Density Theory" in combination with the use of a good quality aggregate and the use of CSF resulted in comparable strengths between mixes with 20 and 10 mm maximum size of aggregate, i.e. the effect of the maximum size of aggregate on strength has been reduced, (page 392).

7. Contrary to the production of normal strength concrete, use of the finer sands generally resulted in higher strengths when producing high strength concrete with compressive strength above 110 N/mm^2, (page 398).

Results from my research programme as well as from the literature review have been used to formulate guidelines as to the selection and proportioning of materials for producing high strength concrete. These are presented in Appendix 6, in a similar format as the BRE report: "Design of Normal Concrete Mixes".

10.3 RECOMMENDATIONS FOR FURTHER WORK.

The following recommendations for further work are suggested:

1. An experimental procedure has been developed for the optimisation of mix proportions that considers both the void content and the surface area of the aggregate on the requirement of excess paste for lubrication. It may be of interest, using this procedure, to develop mix design charts relating the void content of the coarse aggregate and the fineness moduli of sands to the optimum mix proportions.

2. The suitability of equipment and practices for mixing, transportation, placement, and consolidation depends on the rheology of the fresh concrete. The slump test is the current standard test for evaluating the rheological characteristics of fresh concrete. However, this test provides little direct information on the behaviour of concrete during consolidation by vibration or during pumping. There is a need for practical, field test methods, based on technically sound principles, to evaluate rheology. Tattersall's two point test apparatus appears to be suitable for defining the rheological properties of high strength concrete. However, the factors that influence mobility, as measured by this test, may not influence compactibility to the same extend. The possibility of combinations of the factors affecting the mobility of concrete will require large populations of test data before definitive relationships between mobility and compactibility are established.

3. The performance of a trial mixture is usually evaluated under standard temperature conditions. This approach does not provide a true indication of the strength potential of the concrete under field conditions. High strength concretes are complex chemical systems, and the rates of chemical reactions are temperature dependent. In addition, there can be temperature dependent interactions among admixtures, or between the cementitious materials and admixtures. The situation is exacerbated by the late ages for acceptance testing, such as 56, 91 and even 180-days. Thus new practices are needed to evaluate the performance of trial mixtures under the range of conditions expected in the field. The adiabatic

temperature rises of concretes with various binders that have been determined will provide the basis for the prediction of the temperature cycling the concrete will undergo in the field. The relation between the compressive strengths of test specimens subjected to the predicted temperature cycling with those cured under standard temperature conditions should be determined.

4. Testing mortar cubes with steel bars embedded in them so that the failure is initiated in the transition zone appears promising in quantifying the influence of various parameters on the mortar-aggregate bond strength. The use of steel bars instead of coarse aggregates, whose physical properties vary from source to source, will enable results of various workers to be compared. Suggested parameters for investigation are:
   (i) water-cement ratio,
   (ii) partial cement replacement by PFA, GGBFS and CSF,
   (iii) sand-cement ratio, and
   (iv) fineness modulus of sand.

   An investigation, however, is first needed to determine the best size and number of bars to be used.

5. The contribution of PFA, GGBFS and CSF to the compressive strength of concrete with low water-binder ratios has been investigated using mostly 10 mm crushed granite. It would be of interest to compare this to mixes with a strong but smooth and round aggregate.

6. The above compressive strength tests should, if possible, be carried out in conjunction with measurements of the pozzolanic reactivity. This can be evaluated by measuring the calcium hydroxide content at different ages of pastes using Differential Thermal Analysis (DTA), Thermogravimetric Analysis (TGA), and X-ray Diffraction (XRD) methods.

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APPENDIX 1.

EFFECT OF SIZE AND SHAPE OF TEST SPECIMENS ON THE COMPRESSIVE STRENGTH.

EFFECT OF SIZE AND SHAPE OF TEST SPECIMENS ON THE COMPRESSIVE STRENGTH.

The size and shape of specimens for testing concrete in the laboratory differ from country to country. According to the FIP/CEB state-of-the-art report on high strength concrete[1], the characteristic compressive strength is measured on:

(a) 100 mm cubes, 100 x 200 mm or 100 x 300 mm cylinders in Norway,

(b) 160 x 320 mm cylinders in France,

(c) 6 x 12 in. (152.4 x 305.8 mm) or 4 x 8 in. (101.6 x 203.2 mm) cylinders in USA, and,

(d) 200 mm cubes, and more frequently 150 mm cubes, in Germany.

It is a general requirement of standards, e.g., the ASTM C 192[2], that the diameter of the cylinder, or the edge of the cube, should at least be three times that of the maximum nominal aggregate. Therefore, wherever possible there is a trend towards using 100 mm cylinders or cubes to limit the load in the testing machine.

The need to compare or convert the strength of different types of concrete specimens does not arise only from the fact that the size and shape of specimens differ from country to country, but, furthermore, it arises from the use of cores whose height to diameter ratio may not always be controlled.

The investigations that have been carried out to establish conversion factors between different specimens may be divided into two groups; size and shape.

(i) SIZE.

Neville[3] attempted a generalised treatment for any size. He suggests the following empirical formula for converting the strengths between specimen sizes A and B:

$$\log (P_A/P_B \times d_A/d_B) = \log a + b.\log (A_A/A_B)$$

where:

P is the strength of the specimen, and arises from considerations of the probability of occurrence of an element containing a weakest link of a given level of strength.

d is the maximum lateral dimension,

A is the cross-sectional area,

a = 0.8878

b = 0.4525

He does, however, realise that the strength of a concrete specimen is influenced by many factors that were not considered for the derivation of this formula. These include the modulus of elasticity of the aggregate, its Poisson's ratio, and the aggregate-cement ratio (for which factors there were no experimental data available at the time).

At present, there seems to be a lot of research in developing and using 4 by 8 in. (101.6 x 203.2 mm) and even 3 by 6 in. (76.2 x 152.4 mm) cylinders, instead of the 6 by 12 in. (152.4 x 305.8 mm) cylinders for compression tests. Nasser[4], reporting data from normal strength concrete, found that the 3 by 6 in. (76.2 x 152.4 mm) cylinders broke slightly before the 6 by 12 in. (152.4 x 305.8 mm) cylinders, as shown in Figure 1. Their findings differ from those of Forstie et al[5], and Malhotra[6], shown in Figure 2, who showed that the compressive strengths of 6 by 12 in. (152.4 by 305.8 mm) cylinders are generally lower than those of 4 by 8 in. (101.6 by 203.2 mm) cylinders. This is what might be expected from general size effect theory, i.e. the statistical probability of a "weak element" occurring in a cylinder is a function of its size. Their results also show that the differences in strength between the two sizes of cylinders increase with increasing strength level of concrete.

As for normal strength concrete, while Cook[7] reported that 4 by 8 in. (101.6 by 203.2 mm) cylinders exhibited approximately 5 per cent higher strengths than 6 by 12 in. (152.4 x 305.8 mm) cylinders, Carrasquillo et al's test results[8] from 4 by 8 in. (101.6 x 203.2 mm) concrete cylinders were, on average, approximately 93 per cent of those from 6 by 12 in. (152.4 x 305.8 mm) concrete cylinders. Cook's and Carrasquillo et al's results are shown in Figures 3 and 4, respectively.

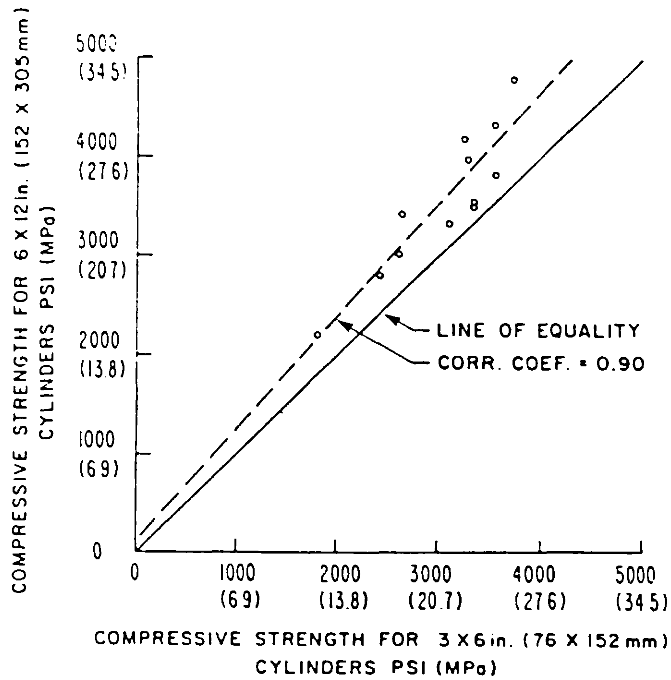

Figure 1: Compressive strength of 6 x 12 in. (152.4 x 305.8 mm) versus 3 x 6 in. (76.2 x 152.4 mm) cylinders, (after Nasser[4]).

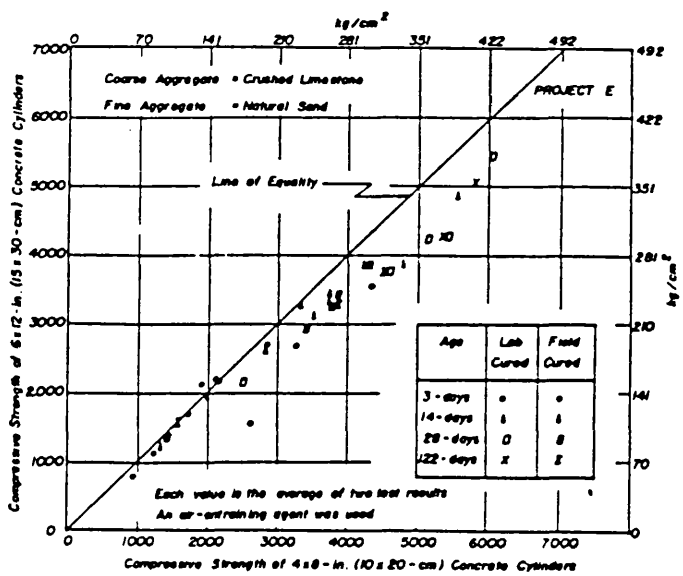

Figure 2: Relationship between strengths of 4 x 8 in. (101.6 x 203.2 mm) and 6 x 12 in. (152.4 x 305.8 mm) cylinders (after Malhotra[6]).

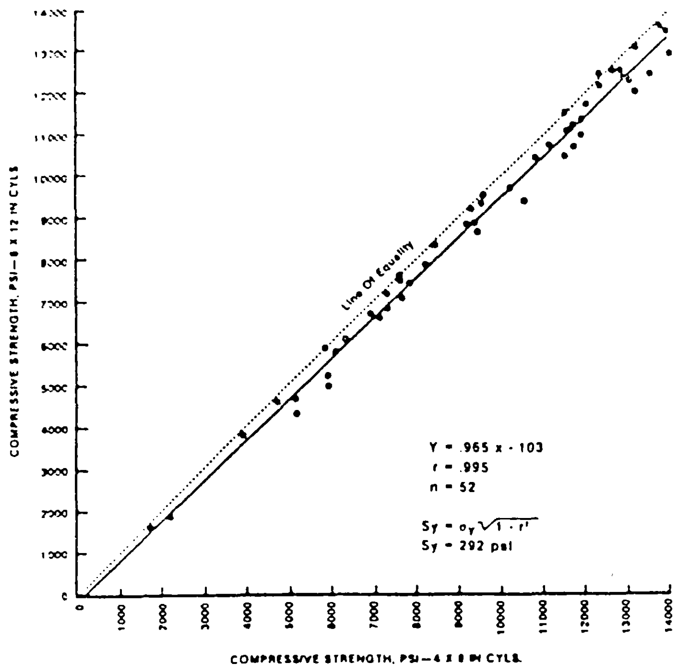

Figure 3: Relationship between the compressive strength of 4 x 8 in. (101.6 x 203.2 mm) and 6 x 12 in. (152.4 x 305.8 mm) cylinders (after Cook[7]).

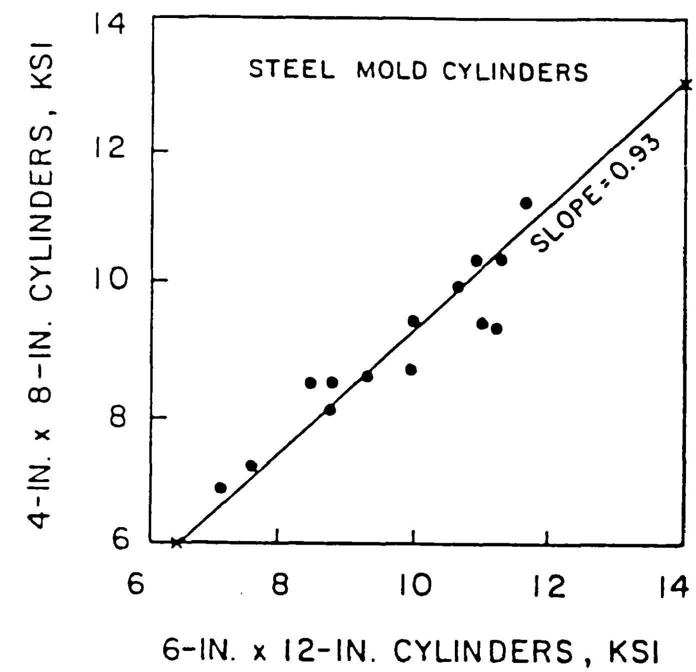

Figure 4: Compressive strength of 4 x 8 in. (101.6 x 203.2 mm) versus 6 x 12 in. (152.4 x 305.8 mm) cylinders, (after Carrasquillo et al[8]).

(ii) SHAPE.

Different conversion factors between compressive cube strength and compressive cylinder strength (height/diameter = 2) of normal strength concretes have been recommended by various standards, e.g. in the British Standard BS 1881: 1952[9] the conversion factor is 0.75, in the revised version, (BS 1881: Part 4: 1970)[10], it has been increased to 0.8, in the CEB/FIP Model Code 90[11], it is 0.8, and in the ASTM Standard C42-64[12] it is 0.91. The problem for normal strength concrete has been investigated by several workers.

Evans[13] suggested that the cylinder-strength/cube-strength ratio depends primarily on the level of strength of the concrete, and increases with strength. L'Hermite[14] suggested that the ratio be taken as:

$$0.76 + 0.2 \log_{10}(f_{cu}/2840)$$

where f_{cu} is the strength of the cube in pounds per square inch.

For comparison between compressive cube strength and compressive cylinder strength (height/diameter = 2) of high strength concrete, the Norwegian Standard NS 3473[15] recommends a reduction of 10 N/mm^2 to the cube strength, whereas the CEB-FIP Model Code 1990[11] recommends a reduction of 15 N/mm^2. However, there exists no universal conversion factor for the different specimens.

Held[16] has obtained the results in Table 1. Fifty four specimens out of each type were tested. The cylinder strengths were in the range of 50 to 95 N/mm^2, the 100 mm cube strengths were in the range of 60 to 110 N/mm^2. The maximum aggregate size was 16 mm, all specimens were tested under the same stressing rate of 0.5 N/mm^2 per second, and the surfaces of the cylinders were ground to a permissible planeness of 0.05 mm. The relationship between 100 mm cube and 150 x 300 mm cylinder results is given in Figure 5. The conversion factor between cube and cylinder strength is approximately 0.8.

In another study on high strength concrete, Smeplass[17] compared 100 mm cubes with 100 x 300 mm cylinders by basically changing the water-cement ratio. The conversion factors he obtained are given in Table 2. The maximum aggregate size was 16 mm. However, in

	Cube			*Cylinder*
Specimens	100	150	200	150/300
Cube 100	1	0.99	0.95	0.82
Cube 150	-	1	0.96	0.83
Cube 200	-	-	1	0.87

Table 1: Conversion factors between different cube and cylinder specimens, (after Held[16]).

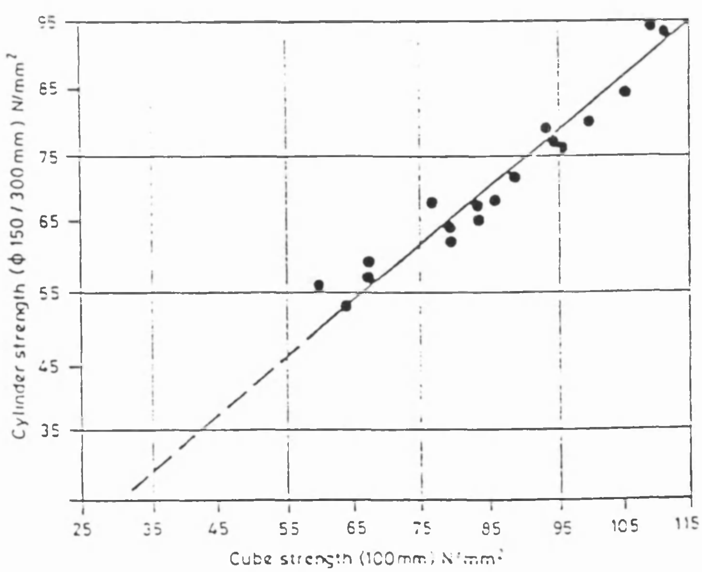

Figure 5: Ratio between 100 mm cube strength and 150 x 300 mm cylinder strength, (after Held[16]).

100 mm Cube N/mm²	*100 x 300 mm Cylinder Strength cyl./cube*	*150 x 300 mm Cylinder Strength cyl./cube*
66.3	0.73	0.75
79.7	0.73	-
97.0	0.77	0.77
115.4	0.82	0.83

Table 2: Cylinder strength/cube strength conversion factors for various concrete strengths and cylinder sizes, (after Smeplass[(17)]).

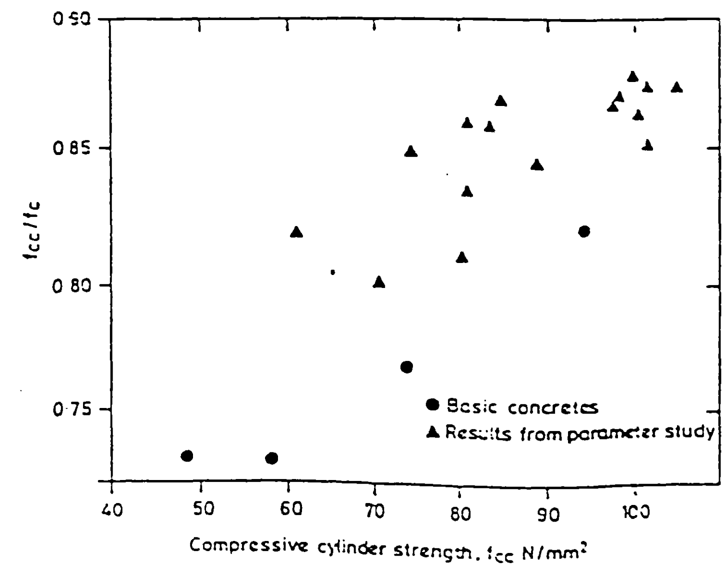

Figure 6: The cylinder/cube strength as a function of cylinder strength, (after Smeplass[(17)]).

a following up study he varied the paste-aggregate ratio, the CSF content, the fine-coarse aggregate ratio and the maximum size of aggregate. The results are given in Figure 6 and demonstrate that the cylinder-cube ratio must not only be a function of the strength grade, but also of the mix design parameters. This may also be the reason that there is still no generally accepted ratio for the strength of 4 by 8 in. (101.6 x 203.2 mm) and 6 by 12 in. (152.4 x 305.8 mm) concrete test cylinders.

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APPENDIX 2.

VOID CONTENT MEASUREMENTS.

% of sand	Test No.	10 mm Granite & Sand (FM = 2.1)		10 mm Limestone & Sand (FM = 2.1)		10 mm Gravel & Sand (FM = 2.1)		10 mm Granite & Sand of FM = 2.73	
		Per cent void content	Average void content	Per cent void content	Average void content	Per cent void content	Average void content	Per cent void content	Average void content
0	1	44.0	44.0	42.0	42.0	41.0	41.0	44.0	44.0
20	1 2	33.0 31.8	32.4	33.0 33.5	33.25	29.8 29.8	29.8	36.5 37.5	37.0
25	1 2	30.8 30.0	30.4	30.0 30.5	30.25	28.0 28.0	28.0	34.0 33.0	33.5
30	1 2	28.5 29.0	28.75	29.5 29.5	29.5	26.2 27.8	27.0	32.5 33.5	33.0
35	1 2	28.0 27.5	27.75	26.8 26.5	26.65	26.5 25.5	26.0	30.0 31.0	30.5
40	1 2	25.5 25.8	25.65	24.5 25.0	24.75	25.0 25.5	25.25	28.5 29.5	29.0
45	1 2	23.5 25.0	24.25	23.8 24.5	24.15	25.0 24.0	24.5	28.0 27.0	27.5
50	1 2	24.8 24.8	24.8	24.0 25.0	24.5	24.5 24.5	24.5	27.5 28.0	27.75
55	1 2	25.0 25.5	25.25	24.5 25.0	24.75	25.0 24.5	24.75	28.0 28.0	28.0
60	1 2					25.3 25.8	25.55	29.0	29.0
100	1	35.0	35.0	35.0	35.0	35.0	35.0	38.0	38.0

(a) Void content of mixtures of 10 mm granite, limestone and gravel with sand.

% of 10 mm	Test No.	20 & 10 mm Granite		20 & 10 mm Limestone	
0	1	44.0	44.0	43.5	43.5
20	1 2	42.0 42.0	42.0	41.0	41.0
25	1 2	41.0 41.5	41.25	40.0	40.0
30	1 2	41.0 40.8	40.9	40.0 40.5	40.25
35	1 2	40.5 40.5	40.5	40.0 40.0	40.0
40	1 2	41.0 41.0	41.0	40.0 40.0	40.0
45	1 2 3 4	41.5 41.5	41.5	40.0 39.0 40.0 39.5	39.625
50	1 2 3 4	41.5 41.5	41.5	39.5 39.0 41.0 40.0	39.875
55	1 2 3 4	41.8 42.0	41.9	39.0 39.0 40.0 39.5	39.375
60	1 2 3 4			39.5 40.0 40.0 40.0	39.875
65	1 2 3 4			40.5 40.5 40.2 40.8	40.5
70	1 2			40.2 40.8	40.5
100	1	44.0	44.0	42.0	42

(b) Void content of mixtures of 20 and 10 mm granite and limestone.

% of 10 mm	Test No.	20 and 10 mm Gravel	
		Per cent void content	Average void content
0	1 2	40 39.8	39.9
7.6	1 2	39.8 39.8	39.8
12.6	1 2	39.0 39.0	39.0
17.6	1 2	39.5 39.0	39.25
22.6	1 2	39.0 38.8	38.9
27.6	1 2	38.5 38.5	38.5
32.6	1 2	39.0 39.0	39.0
37.6	1 2	39.0 39.5	39.25
42.6	1 2	39.5 39.5	39.5
47.6	1 2	39.0 39.5	39.25
52.6	1 2	39.8 39.5	39.65
100	1	41.0	41.0

(c) Void content of mixtures of 20 and 10 mm gravel.

% of sand	Test No.	20-5 mm Granite[1] & Sand (FM = 2.1)		20-5 mm Limestone[2] & Sand (FM = 2.1)		20-5 mm Gravel[3] & Sand (FM = 2.1)	
		Per cent void content	Average void content	Per cent void content	Average void content	Per cent void content	Average void content
0	1 2	40.5 40.5	40.5	38.5 40.0	39.25	38.0 39.0	38.5
20	1 2	30.5 30.0	30.25	28.5 29.0	28.75	28.0 28.5	28.25
25	1 2	27.0 28.0	27.5	27.5 27.5	27.5	26.5 27.3	26.9
30	1 2 3 4	26.2 24.5 25.5 25.5	25.425	27.0 26.8	26.9	26.0 24.8	25.4
35	1 2 3 4	23.0 24.0 24.2 23.5	23.675	24.8 24.0 23.5 23.8	24.025	24.5 23.5 23.8 24.0	23.95
40	1 2 3 4	22.8 24.0 24.3 23.0	23.525	23.0 22.0 23.2 22.0	22.55	23.8 23.8 23.0 23.0	23.4
45	1 2 3 4	22.5 23.0 24.0 22.8	23.075	23.0 22.0 23.0 22.0	22.5	23.5 23.5	23.5
50	1 2 3 4	23.0 24.5 23.3 22.0	23.2	23.5 23.0	23.25	23.8 24.0	23.9
55	1 2 3	24.8 24.0 24.2	24.333	24.8	24.8		
60	1 2	25.5 24.5	25.0	25.5	25.5		
100	1	35.0	35.0	35.0	35.0	35.0	35.0

(1) 65 % of 20 mm and 35 % of 10 mm granite.
(2) 50 % of 20 mm and 50 % of 10 mm limestone.
(3) 72.4 % of 20 mm and 27.6 % of 10 mm gravel.

(d) Void content of mixtures of 20-5 mm granite, limestone and gravel.

APPENDIX 3.

ADSORPTION TESTS.

(a) Calibration procedure.

(b) Adsorption tests on OPC.

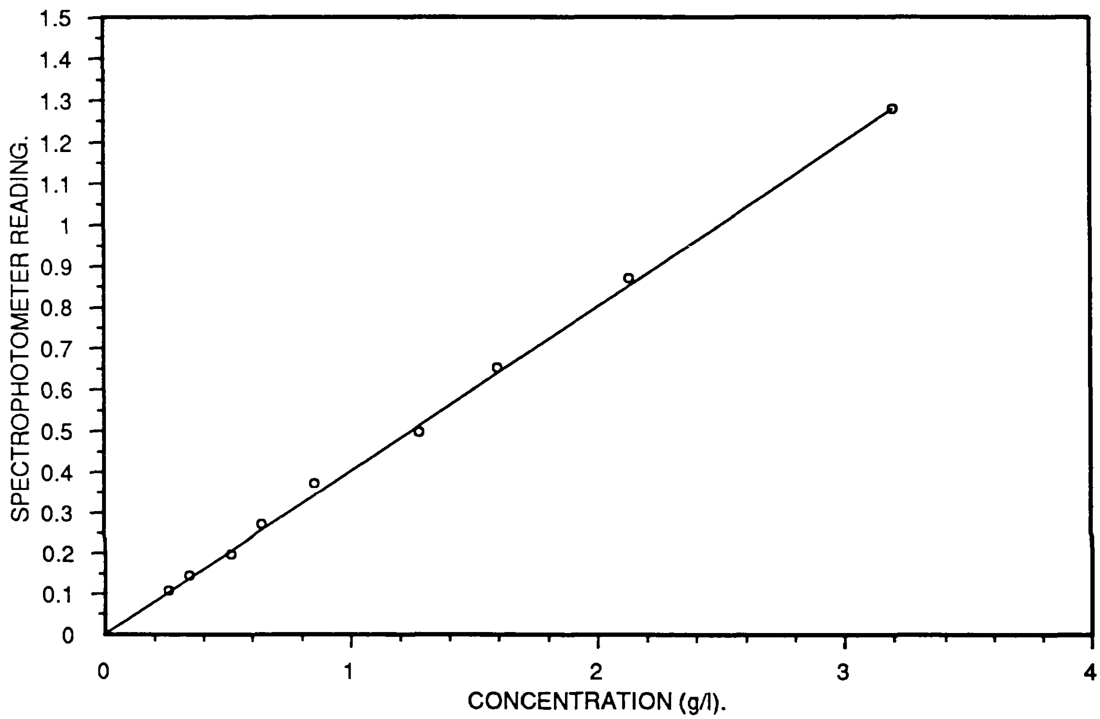
(c) Adsorption tests on CSF.

CALIBRATION PROCEDURE.

Concentration of solutions used for the calibration of the CAMSPEC Spectrophotometer. 375 nm wavelength was used.

Superplasticiser Concentration (g/l)	Spectrophotometer Reading
3.200	1.280
2.132	0.871
1.600	0.652
1.280	0.496
0.853	0.372
0.640	0.272
0.512	0.196
0.341	0.145
0.256	0.107

The calibration coefficient, as determined from the figure on the next page is 2.5.



CALIBRATION OF THE CAMSPEC SPECTROPHOTOMETER.

Solution No.	Weight of Conplast 430 slurry[1] (g/l)	Weight of Conplast 430 solids (g)	Ratio of superplasticiser to OPC solids[2] (% by weight)	Spectrophotometer reading	Remaining superplasticiser (g/l)	Adsorbed superplasticiser (g)	Ratio of superplasticiser adsorbed to OPC solids (% by weight)
1.	0.250	0.10	0.50	0.016	0.040	0.060	0.300
2.	0.500	0.20	1.00	0.010	0.025	0.175	0.875
3.	0.625	0.25	1.25	0.016	0.040	0.210	1.050
4.	0.750	0.30	1.50	0.026	0.065	0.235	1.175
5.	0.875	0.35	1.75	0.026	0.065	0.285	1.425
6.	1.000	0.40	2.00	0.043	0.108	0.293	1.463
7.	1.125	0.45	2.25	0.051	0.128	0.323	1.613
8.	1.250	0.50	2.50	0.081	0.203	0.298	1.488
9.	1.500	0.60	3.00	0.150	0.375	0.225	1.125

(1) 100 g of Conplast 430 slurry contains 40 g of Conplast 430 solids.
(2) 1 litre of solution contained 20 g of OPC solids.

ADSORPTION TESTS ON ORDINARY PORTLAND CEMENT.

Solution No.	Weight of Conplast 430 slurry[1] (g/l)	Weight of Conplast 430 solids (g)	Ratio of superplasticiser to CSF solids[2] (% by weight)	Spectrophotometer reading	Remaining superplasticiser (g/l)	Adsorbed superplasticiser (g)	Ratio of superplasticiser adsorbed to CSF solids (% by weight)
1.	0.500	0.20	1.00	0.006	0.002	0.185	0.925
2.	1.000	0.40	2.00	0.010	0.003	0.375	1.875
3.	1.500	0.60	3.00	0.024	0.060	0.540	2.700
4.	2.000	0.80	4.00	0.062	0.155	0.645	3.225
5.	2.500	1.00	5.00	0.095	0.238	0.763	3.813
6.	2.750	1.10	5.50	0.132	0.330	0.770	3.850
7.	3.000	1.20	6.00	0.166	0.415	0.785	3.925
8.	3.250	1.30	6.50	0.189	0.473	0.828	4.138
9.	3.500	1.40	7.00	0.227	0.568	0.833	4.163
10.	4.000	1.60	8.00	0.378	0.945	0.655	3.275
11.	5.000	2.00	10.00	0.568	1.420	0.580	2.900
12.	6.000	2.40	12.00	0.716	1.790	0.610	3.050

(1) 100 g of Conplast 430 slurry contains 40 g of Conplast 430 solids.
(2) 1 litre contained 20 g of CSF solids.

ADSORPTION TESTS ON CONDENSED SILICA FUME.

APPENDIX 4.

TWO POINT WORKABILITY TESTS.

(a) Torque/pressure calibration and calibration of speed settings.

(b) Detailed results of two point workability tests,
   (i) in tabular form, and,
   (ii) in graphical form.

PART A.

Torque/pressure calibration and calibration of speed settings.

(I) Torque/pressure calibration.

The torque/pressure calibration graph may be obtained by using an ordinary plummer block lined with brass shells as shown in Figure A.1. The plummer block is fixed so that the impeller drive shaft passes through it and, by varying the tightness of the bolts, it is easy to obtain any desired level of frictional force. A simple lever attached to the base of the plummer block then permits the application of a retarding torque which can be measured by the dead weight and pulley method, as shown in Figure A.1.

Method of measurement:

1. The calibration unit is clamped to the machine as shown in Figure A.2, and the apparatus allowed to warm up for 30 minutes.
2. The spring balance is set to zero and oil is applied to the shaft and bearing to provide lubrication and prevent binding.
3. The hydraulic unit speed control is set to 6.
4. Torque is applied to the shaft by tightening evenly the cap securing nuts of the plumber bearing till a pressure of around 450 psi is attained.
5. Readings of spring balance and pressure are noted, (see Table A.1).
6. Without altering the speed control setting, the cap securing nuts are slightly released to give a pressure drop of around 50 psi and again readings of spring balance and pressure are obtained.
7. This procedure is repeated till 4 pairs of results are derived at each particular speed setting, (5, 4, 3, 2,), as shown in Table A.1.
8. The bearing cap nuts are then fully released, oil applied to the shaft and bearing and the procedure repeated.
9. The spring balance and pressure are then recorded at speeds 5, 4, 3, and 2.
10. The bearing assembly is then completely removed from the impeller shaft and idling pressures obtained at speeds 6, 5, 4, 3, and 2.

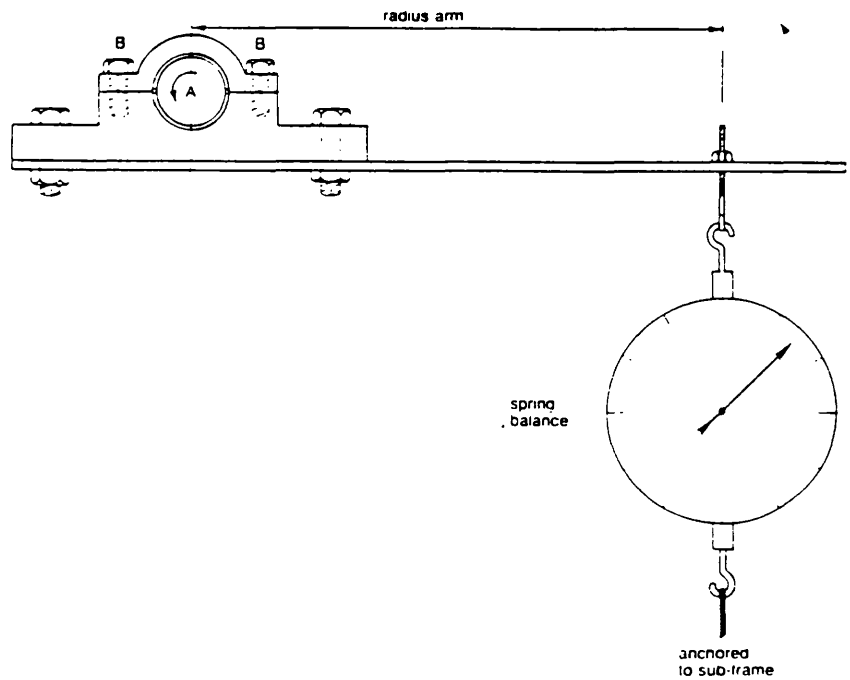


Figure A.1: Torque calibration arrangement using plummer block, lever, and spring balance.

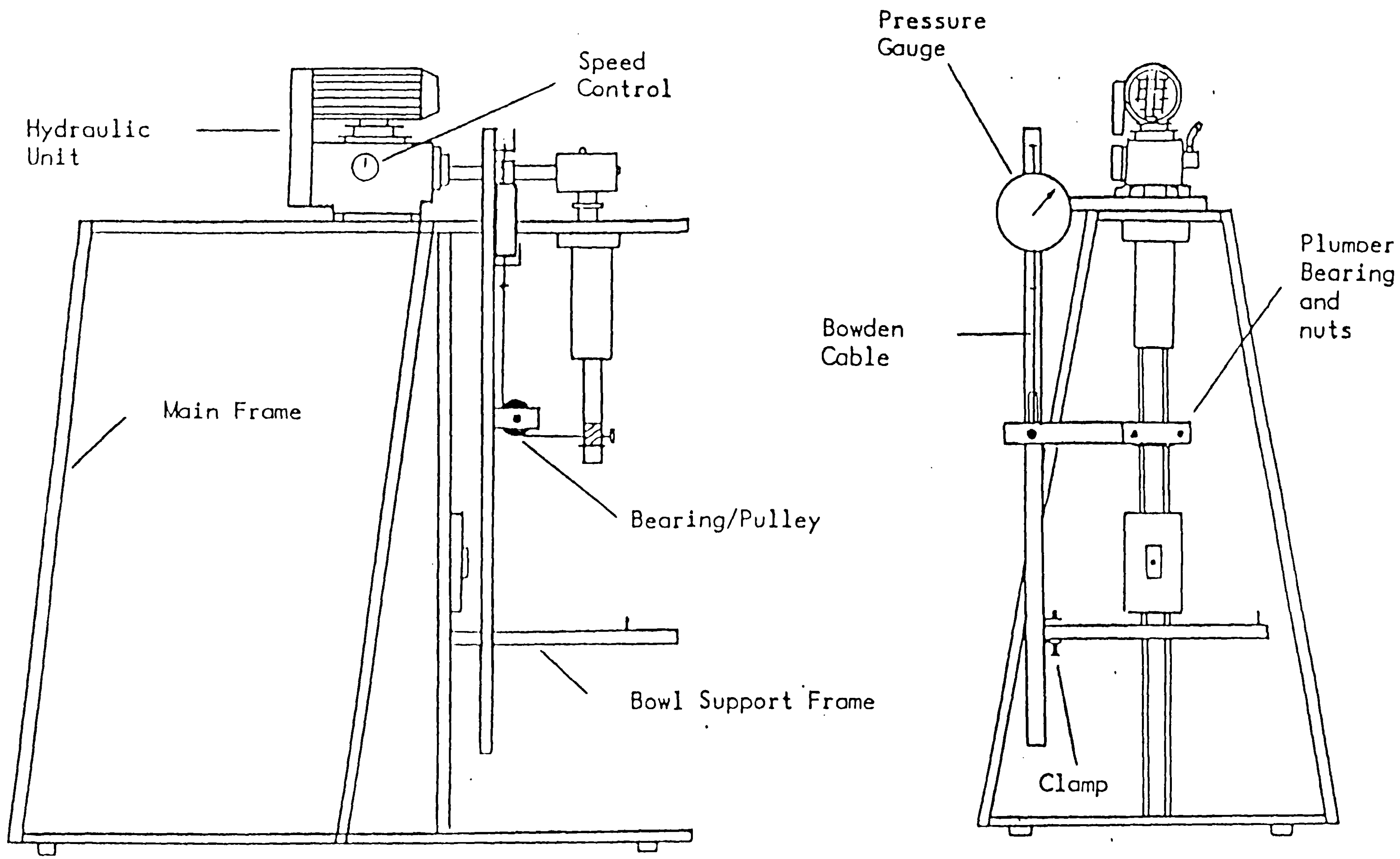


Figure A.2: Calibration assembly for two point test apparatus.

Speed setting	Spring balance (N)	Oil pressure (psi)	Idling pressure (psi)	Net pressure (psi)
6.	37.5	450	170	280
	29	390		220
	18.5	310		140
	15.5	280		110
	8	220		50
	5	200		30
	1.75	175		5
5.	40	470	155	315
	30.5	390		235
	25	340		185
	21	310		155
	13.5	255		100
	9.75	220		65
	4.25	180		25
	1	160		5
4.	38	430	145	285
	27	350		205
	20	290		145
	10	220		75
	4	175		30
3.	42	460	135	315
	31	370		225
	23	310		175
	16	260		125
	7.5	190		55
	2	150		15
2.	43	470	125	345
	30	370		245
	22.5	300		175
	16.5	250		125
	9.5	200		75
	4	160		35
	1.25	135		10

Oil pressure after warm-up and idling at zero speed = 70 psi.

Lever arm distance = 179 mm

Table A.1: Torque/pressure calibration.

11. A plot of spring balance against net pressure can, therefore, be made and the slope of the line derived. This has been found, from Figure A.3, to be 42/320 N/psi.
12. Having determined the length of the metal arm from the centre of the plumber bearing to the centre of the Bowden Cable (in metres) the resultant figure can be multiplied by the SLOPE and "g" (acceleration due to gravity) to give the CALIBRATION COEFFICIENT in Nm per psi. This has been found to be 0.0235 Nm/psi.

(II) CALIBRATION OF SPEED SETTINGS.

The revolutions per minute of the spider gear coupling was measured using a tachometer, (see Table A.2 and Figure A.4). The gear ratio was found to be very close to that specified by Tattersall in the apparatus' manual, i.e. 4.746:1 was the measured and 4.75:1 was the suggested value.

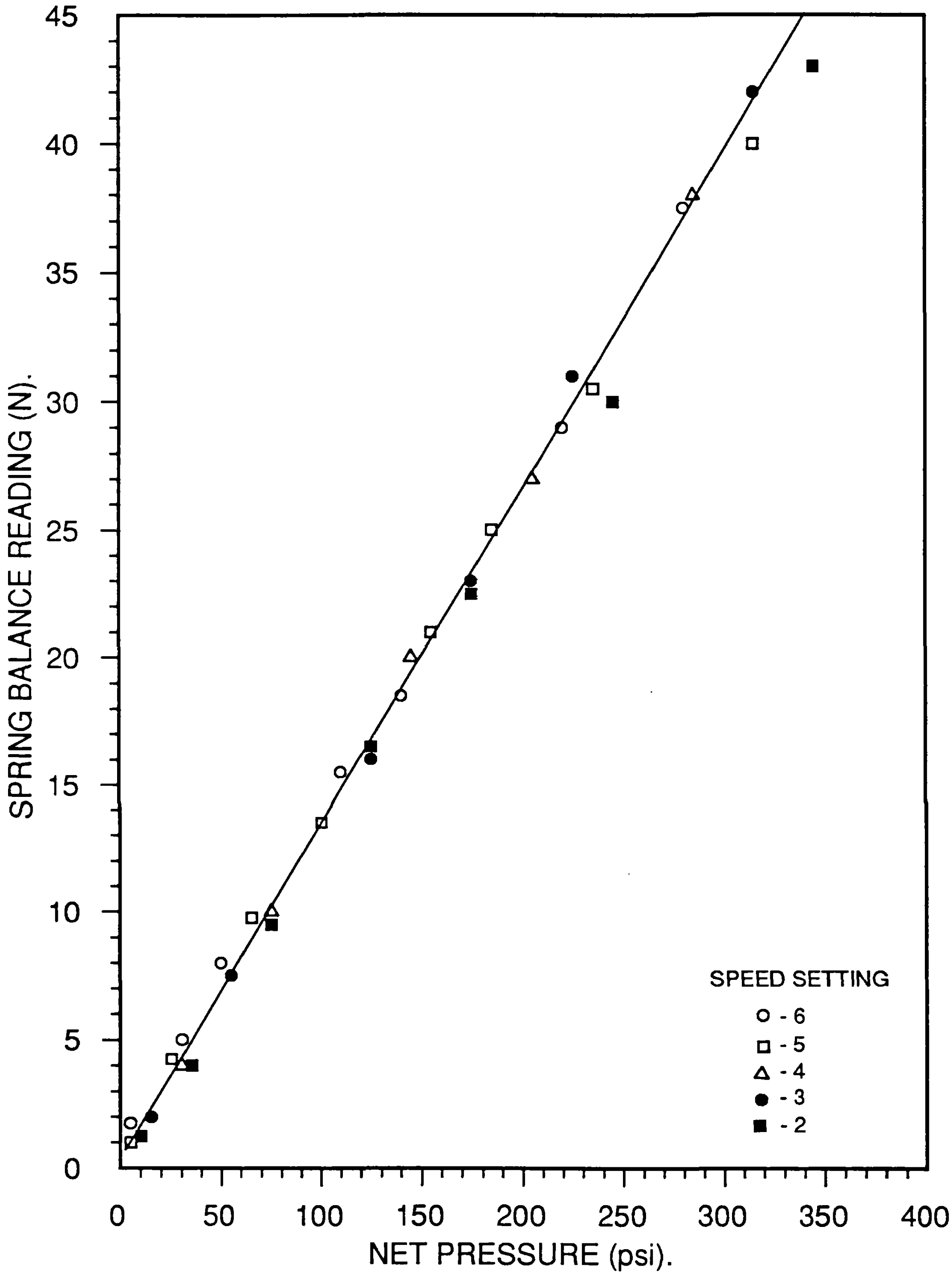


Figure A.3: Torque/pressure calibration.
(Calibration coefficient = 0.0235 Nm/psi).

Speed setting	Revolutions per minute (rpm) of:				Gear ratio (decreasing)
	Spider gear		Impeller shaft		
	Increasing	Decreasing	Increasing	Decreasing	
0.5	47				
1.0	70	70			
1.5	120				
2.0	174	174			
2.5	226		47		(4.81)
3.0	280	282	60	59	4.67
3.5	332		70		4.74
4.0	383	388	81	82	4.73
4.5	438		92		4.76
5.0	490	492	103	103	4.76
5.5	542		114		4.75
6.0	594	597	125	125	4.75
6.5	644		136		4.74
7.0	697	697	147	147	4.74
7.5	749		157		4.77
8.0	802		170		4.72
				Average:	4.746

NOTE: Mark II apparatus tested with no load.

Table A.2: Calibration of speed settings.

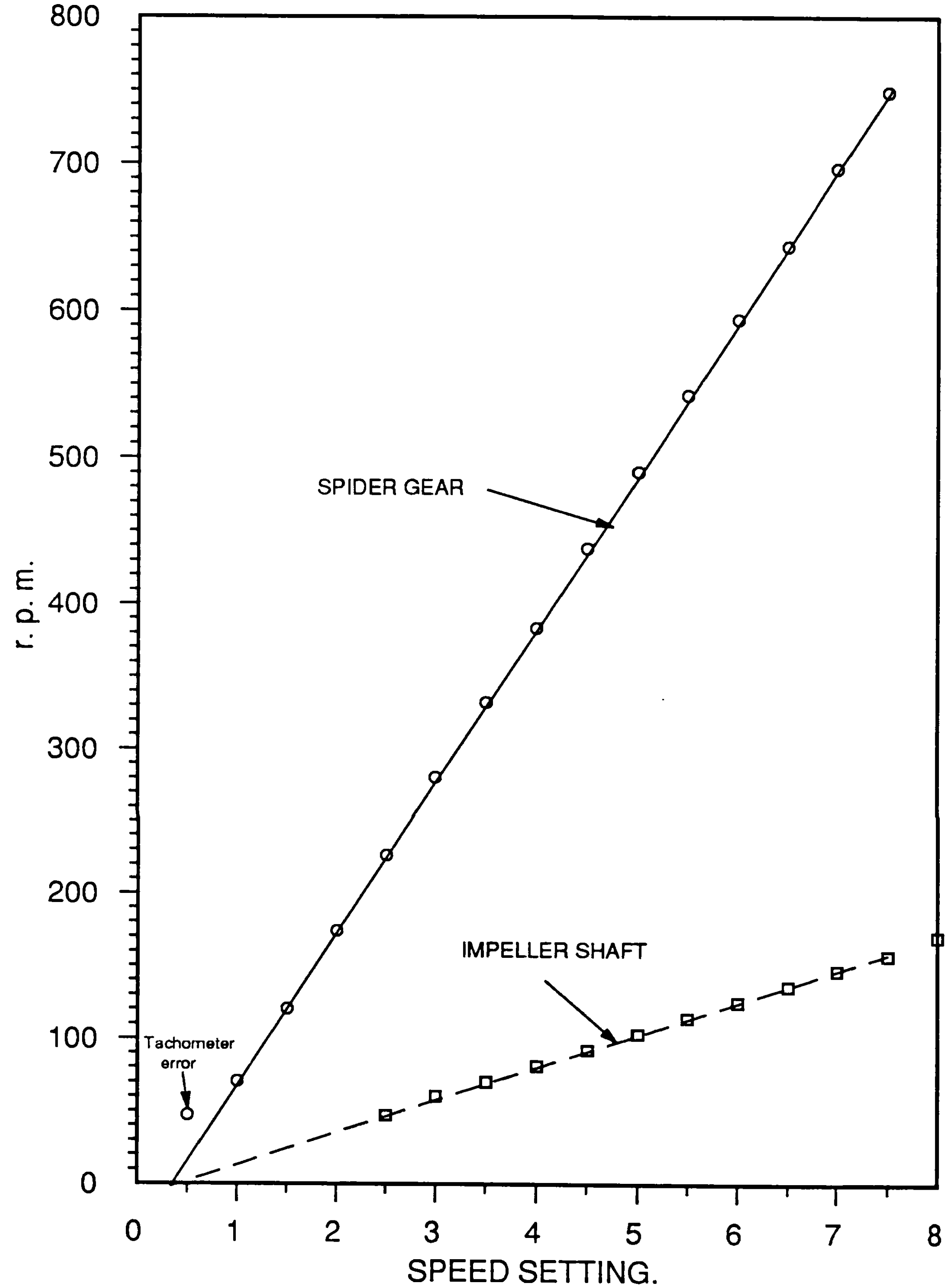


Figure A.4: Calibration of speed settings.

PART B: DETAILED RESULTS OF TWO POINT WORKABILITY TESTS (I) IN TABULAR FORM.

MIX No. : 1 — OPC mix with fine sand, water-cement ratio = 0.38

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	380	177.5	202.5	1.34	4.76
3.5	332	360	170.0	190.0	1.17	4.47
3.0	280	340	162.5	177.5	0.98	4.17
2.5	226	315	155.0	160.0	0.79	3.76
2.0	174	290	147.5	142.5	0.61	3.35
1.5	120	270	140.0	130.0	0.42	3.06
1.0	70	250	132.5	117.5	0.25	2.76

Slump before test : 170 mm
Slump after test : 140 mm
Average slump : 155 mm

Yield (g) value : 2.27 Nm
Plastic viscosity (h) value : 1.88 Nms
Correlation coefficient : 99.7 %

MIX No. : 2 — OPC mix with fine sand, water-cement ratio = 0.35

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	400.0	175.0	225.0	1.34	5.29
3.5	332	375.0	167.5	207.5	1.17	4.88
3.0	280	350.0	160.0	190.0	0.98	4.47
2.5	226	325.0	152.5	172.5	0.79	4.05
2.0	174	305.0	145.0	160.0	0.61	3.76
1.5	120	285.0	137.5	147.5	0.42	3.47
1.0	70	255.0	130.0	125.0	0.25	2.94

Slump before test : 160 mm
Slump after test : 140 mm
Average slump : 150 mm

Yield (g) value : 2.49 Nm
Plastic viscosity (h) value : 2.49 Nms
Correlation coefficient : 99.4 %

MIX No. : 3 — OPC mix with fine sand, water-cement ratio = 0.32

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	430.0	177.5	252.5	1.34	5.93
3.5	332	405.0	170.0	235.0	1.17	5.52
3.0	280	375.0	162.5	212.5	0.98	4.99
2.5	226	355.0	155.0	200.0	0.79	4.70
2.0	174	330.0	147.5	182.5	0.61	4.29
1.5	120	305.0	140.0	165.0	0.42	3.88
1.0	70	280.0	132.5	147.5	0.25	3.47

Slump before test : 155 mm
Slump after test : 140 mm
Average slump : 150 mm

Yield (g) value : 2.91 Nm
Plastic viscosity (h) value : 2.22 Nms
Correlation coefficient : 99.7 %

MIX No. : 4

OPC mix with fine sand, water-cement ratio = 0.30

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	465.0	177.5	287.5	1.34	6.76
3.5	332	445.0	172.5	272.5	1.17	6.40
3.0	280	410.0	165.0	245.0	0.98	5.76
2.5	226	390.0	157.5	232.5	0.79	5.46
2.0	174	355.0	150.0	205.0	0.61	4.82
1.5	120	330.0	142.5	187.5	0.42	4.41
1.0	70	300.0	135.0	165.0	0.25	3.88

Slump before test : 170 mm
Slump after test : 150 mm
Average slump : 160 mm

Yield (g) value : 3.26 Nm
Plastic viscosity (h) value : 2.65 Nms
Correlation coefficient : 99.5 %

MIX No. : 5

OPC mix with fine sand, water-cement ratio = 0.28

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	520.0	187.5	332.5	1.34	7.81
3.5	332	480.0	180.0	300.0	1.17	7.05
3.0	280	450.0	172.5	277.5	0.98	6.52
2.5	226	425.0	165.0	260.0	0.79	6.11
2.0	174	395.0	157.5	237.5	0.61	5.58
1.5	120	360.0	150.0	210.0	0.42	4.94
1.0	70	325.0	142.5	182.5	0.25	4.29

Slump before test : 160 mm
Slump after test : 140 mm
Average slump : 150 mm

Yield (g) value : 3.60 Nm
Plastic viscosity (h) value : 3.08 Nms
Correlation coefficient : 99.3 %

MIX No. : 6

OPC mix with fine sand, water-cement ratio = 0.26

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	540.0	165.0	375.0	1.34	8.81
3.5	332	515.0	157.5	357.5	1.17	8.40
3.0	280	480.0	152.5	327.5	0.98	7.70
2.5	226	450.0	145.0	305.0	0.79	7.17
2.0	174	410.0	140.0	270.0	0.61	6.35
1.5	120	380.0	132.5	247.5	0.42	5.82
1.0	70	340.0	127.5	212.5	0.25	4.99

Slump before test : 160 mm
Slump after test : 140 mm
Average slump : 150 mm

Yield (g) value : 4.25 Nm
Plastic viscosity (h) value : 3.52 Nms
Correlation coefficient : 99.4 %

MIX No. : 7

OPC mix with fine sand, water-cement ratio = 0.26

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	555.0	167.5	387.5	1.34	9.11
3.5	332	525.0	160.0	365.0	1.17	8.58
3.0	280	490.0	152.5	337.5	0.98	7.93
2.5	226	455.0	147.5	307.5	0.79	7.23
2.0	174	420.0	140.0	280.0	0.61	6.58
1.5	120	385.0	132.5	252.5	0.42	5.93
1.0	70	350.0	125.0	225.0	0.25	5.29

Slump before test : 150 mm
Slump after test : 130 mm
Average slump : 140 mm

Yield (g) value : 4.43 Nm
Plastic viscosity (h) value : 3.53 Nms
Correlation coefficient : 100.0 %

MIX No. : 7

OPC mix with low sand content, water-cement ratio = 0.30

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	440.0	167.5	272.5	1.34	6.40
3.5	332	415.0	160.0	255.0	1.17	5.99
3.0	280	390.0	152.5	237.5	0.98	5.58
2.5	226	360.0	145.0	215.0	0.79	5.05
2.0	174	330.0	140.0	190.0	0.61	4.47
1.5	120	300.0	132.5	167.5	0.42	3.94
1.0	70	270.0	125.0	145.0	0.25	3.41

Slump before test : 150 mm
Slump after test : 130 mm
Average slump : 140 mm

Yield (g) value : 2.79 Nm
Plastic viscosity (h) value : 2.77 Nms
Correlation coefficient : 99.6 %

MIX No. : 9

OPC mix with low sand content, water-cement ratio = 0.28

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	460.0	160.0	300.0	1.34	7.05
3.5	332	430.0	155.0	275.0	1.17	6.46
3.0	280	405.0	150.0	255.0	0.98	5.99
2.5	226	370.0	142.5	227.5	0.79	5.35
2.0	174	340.0	135.0	205.0	0.61	4.82
1.5	120	305.0	127.5	177.5	0.42	4.17
1.0	70	280.0	122.5	157.5	0.25	3.70

Slump before test : 165 mm
Slump after test : 145 mm
Average slump : 155 mm

Yield (g) value : 2.92 Nm
Plastic viscosity (h) value : 3.08 Nms
Correlation coefficient : 99.9 %

MIX No. : 10 — OPC mix with low sand content, water-cement ratio = 0.26

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	510.0	162.5	347.5	1.34	8.17
3.5	332	475.0	157.5	317.5	1.17	7.46
3.0	280	440.0	150.0	290.0	0.98	6.82
2.5	226	405.0	142.5	262.5	0.79	6.17
2.0	174	370.0	137.5	232.5	0.61	5.46
1.5	120	335.0	130.0	205.0	0.42	4.82
1.0	70	290.0	122.5	167.5	0.25	3.94

Slump before test : 155 mm
Slump after test : 145 mm
Average slump : 150 mm

Yield (g) value : 3.13 Nm
Plastic viscosity (h) value : 3.77 Nms
Correlation coefficient : 99.7 %

MIX No. : 11 — OPC mix with coarse sand, water-cement ratio = 0.38

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	350.0	165.0	185.0	1.34	4.35
3.5	332	330.0	157.5	172.5	1.17	4.05
3.0	280	310.0	150.0	160.0	0.98	3.76
2.5	226	290.0	145.0	145.0	0.79	3.41
2.0	174	270.0	137.5	132.5	0.61	3.11
1.5	120	250.0	130.0	120.0	0.42	2.82
1.0	70	225.0	122.5	102.5	0.25	2.41

Slump before test : 170 mm
Slump after test : 130 mm
Average slump : 150 mm

Yield (g) value : 2.03 Nm
Plastic viscosity (h) value : 1.74 Nms
Correlation coefficient : 99.7 %

MIX No. : 12 — OPC mix with coarse sand, water-cement ratio = 0.35

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	380.0	162.5	217.5	1.34	5.11
3.5	332	355.0	157.5	197.5	1.17	4.64
3.0	280	330.0	150.0	180.0	0.98	4.23
2.5	226	310.0	145.0	165.0	0.79	3.88
2.0	174	285.0	137.5	147.5	0.61	3.47
1.5	120	260.0	130.0	130.0	0.42	3.06
1.0	70	240.0	122.5	117.5	0.25	2.76

Slump before test : 160 mm
Slump after test : 120 mm
Average slump : 140 mm

Yield (g) value : 2.19 Nm
Plastic viscosity (h) value : 2.14 Nms
Correlation coefficient : 99.8 %

MIX No. : 13 — OPC mix with coarse sand, water-cement ratio = 0.32

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	415.0	165.0	250.0	1.34	5.88
3.5	332	390.0	157.5	232.5	1.17	5.46
3.0	280	365.0	150.0	215.0	0.98	5.05
2.5	226	340.0	145.0	195.0	0.79	4.58
2.0	174	320.0	137.5	182.5	0.61	4.29
1.5	120	290.0	130.0	160.0	0.42	3.76
1.0	70	265.0	122.5	142.5	0.25	3.35

Slump before test : 155 mm
Slump after test : 135 mm
Average slump : 145 mm

Yield (g) value : 2.80 Nm
Plastic viscosity (h) value : 2.29 Nms
Correlation coefficient : 99.8 %

MIX No. : 14 — OPC mix with coarse sand, water-cement ratio = 0.29

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	475.0	165.0	310.0	1.34	7.29
3.5	332	445.0	157.5	287.5	1.17	6.76
3.0	280	410.0	150.0	260.0	0.98	6.11
2.5	226	385.0	145.0	240.0	0.79	5.64
2.0	174	355.0	137.5	217.5	0.61	5.11
1.5	120	325.0	130.0	195.0	0.42	4.58
1.0	70	290.0	125.0	165.0	0.25	3.88

Slump before test : 160 mm
Slump after test : 140 mm
Average slump : 150 mm

Yield (g) value : 3.21 Nm
Plastic viscosity (h) value : 3.04 Nms
Correlation coefficient : 99.7 %

MIX No. : 15 — OPC mix with coarse sand, water-cement ratio = 0.26

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	540.0	162.5	377.5	1.34	8.87
3.5	332	510.0	155.0	355.0	1.17	8.34
3.0	280	485.0	150.0	335.0	0.98	7.87
2.5	226	450.0	142.5	307.5	0.79	7.23
2.0	174	410.0	135.0	275.0	0.61	6.46
1.5	120	370.0	130.0	240.0	0.42	5.64
1.0	70	340.0	125.0	215.0	0.25	5.05

Slump before test : 175 mm
Slump after test : 125 mm
Average slump : 150 mm

Yield (g) value : 4.21 Nm
Plastic viscosity (h) value : 3.58 Nms
Correlation coefficient : 99.2 %

MIX No. : 16 — OPC mix with Conplast M1, water-cement ratio = 0.38

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	440.0	170.0	270.0	1.34	6.35
3.5	332	410.0	162.5	247.5	1.17	5.82
3.0	280	380.0	157.5	222.5	0.98	5.23
2.5	226	350.0	150.0	200.0	0.79	4.70
2.0	174	320.0	142.5	177.5	0.61	4.17
1.5	120	290.0	135.0	155.0	0.42	3.64
1.0	70	260.0	127.5	132.5	0.25	3.11

Slump before test : 150 mm
Slump after test : 130 mm
Average slump : 140 mm

Yield (g) value : 2.39 Nm
Plastic viscosity (h) value : 2.95 Nms
Correlation coefficient : 100.0 %

MIX No. : 17 — OPC mix with Conplast M1, water-cement ratio = 0.35

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	460.0	160.0	300.0	1.34	7.05
3.5	332	430.0	152.5	277.5	1.17	6.52
3.0	280	400.0	147.5	252.5	0.98	5.93
2.5	226	370.0	140.0	230.0	0.79	5.41
2.0	174	340.0	135.0	205.0	0.61	4.82
1.5	120	300.0	127.5	172.5	0.42	4.05
1.0	70	270.0	122.5	147.5	0.25	3.47

Slump before test : 150 mm
Slump after test : 120 mm
Average slump : 135 mm

Yield (g) value : 2.72 Nm
Plastic viscosity (h) value : 3.28 Nms
Correlation coefficient : 99.7 %

MIX No. : 18 — OPC mix with Conplast M1, water-cement ratio = 0.32

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	520.0	162.5	357.5	1.34	8.40
3.5	332	490.0	155.0	335.0	1.17	7.87
3.0	280	460.0	150.0	310.0	0.98	7.29
2.5	226	430.0	142.5	287.5	0.79	6.76
2.0	174	395.0	137.5	257.5	0.61	6.05
1.5	120	355.0	130.0	225.0	0.42	5.29
1.0	70	320.0	125.0	195.0	0.25	4.58

Slump before test : 150 mm
Slump after test : 130 mm
Average slump : 140 mm

Yield (g) value : 3.83 Nm
Plastic viscosity (h) value : 3.49 Nms
Correlation coefficient : 99.4 %

MIX No. : 19

OPC mix with Conplast M1, water-cement ratio = 0.29

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	650.0	160.0	490.0	1.34	11.52
3.5	332	610.0	152.5	457.5	1.17	10.75
3.0	280	560.0	147.5	412.5	0.98	9.69
2.5	226	520.0	140.0	380.0	0.79	8.93
2.0	174	470.0	135.0	335.0	0.61	7.87
1.5	120	420.0	127.5	292.5	0.42	6.87
1.0	70	370.0	122.5	247.5	0.25	5.82

Slump before test : 170 mm
Slump after test : 130 mm
Average slump : 150 mm

Yield (g) value : 4.67 Nm
Plastic viscosity (h) value : 5.20 Nms
Correlation coefficient : 99.8 %

MIX No. : 20

40 per cent PFA mix, water-binder ratio = 0.38

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	410.0	170.0	240.0	1.34	5.64
3.5	332	380.0	162.5	217.5	1.17	5.11
3.0	280	350.0	155.0	195.0	0.98	4.58
2.5	226	325.0	147.5	177.5	0.79	4.17
2.0	174	300.0	140.0	160.0	0.61	3.76
1.5	120	280.0	132.5	147.5	0.42	3.47
1.0	70	250.0	125.0	125.0	0.25	2.94

Slump before test : 160 mm
Slump after test : 130 mm
Average slump : 145 mm

Yield (g) value : 2.35 Nm
Plastic viscosity (h) value : 2.39 Nms
Correlation coefficient : 99.2 %

MIX No. : 21

40 per cent PFA mix, water-binder ratio = 0.32

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	430.0	160.0	270.0	1.34	6.35
3.5	332	410.0	155.0	255.0	1.17	5.99
3.0	280	385.0	150.0	235.0	0.98	5.52
2.5	226	360.0	142.5	217.5	0.79	5.11
2.0	174	330.0	137.5	192.5	0.61	4.52
1.5	120	300.0	130.0	170.0	0.42	4.00
1.0	70	275.0	122.5	152.5	0.25	3.58

Slump before test : 160 mm
Slump after test : 135 mm
Average slump : 150 mm

Yield (g) value : 2.96 Nm
Plastic viscosity (h) value : 2.59 Nms
Correlation coefficient : 99.7 %

MIX No. : 22 — 20 per cent PFA mix, water-binder ratio = 0.26

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	560.0	160.0	400.0	1.34	9.40
3.5	332	525.0	155.0	370.0	1.17	8.70
3.0	280	490.0	147.5	342.5	0.98	8.05
2.5	226	460.0	142.5	317.5	0.79	7.46
2.0	174	415.0	135.0	280.0	0.61	6.58
1.5	120	375.0	130.0	245.0	0.42	5.76
1.0	70	340.0	125.0	215.0	0.25	5.05

Slump before test : 155 mm
Slump after test : 135 mm
Average slump : 145 mm

Yield (g) value : 4.12 Nm
Plastic viscosity (h) value : 3.98 Nms
Correlation coefficient : 99.6 %

MIX No. : 23 — 30 per cent PFA mix, water-binder ratio = 0.26

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	540.0	160.0	380.0	1.34	8.93
3.5	332	510.0	155.0	355.0	1.17	8.34
3.0	280	475.0	150.0	325.0	0.98	7.64
2.5	226	445.0	142.5	302.5	0.79	7.11
2.0	174	410.0	137.5	272.5	0.61	6.40
1.5	120	360.0	130.0	230.0	0.42	5.41
1.0	70	320.0	125.0	195.0	0.25	4.58

Slump before test : 165 mm
Slump after test : 145 mm
Average slump : 155 mm

Yield (g) value : 3.79 Nm
Plastic viscosity (h) value : 3.96 Nms
Correlation coefficient : 99.0 %

MIX No. : 24 — 40 per cent PFA mix, water-binder ratio = 0.26

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	510.0	160.0	350.0	1.34	8.23
3.5	332	485.0	155.0	330.0	1.17	7.76
3.0	280	455.0	147.5	307.5	0.98	7.23
2.5	226	415.0	142.5	272.5	0.79	6.40
2.0	174	380.0	135.0	245.0	0.61	5.76
1.5	120	340.0	130.0	210.0	0.42	4.94
1.0	70	300.0	125.0	175.0	0.25	4.11

Slump before test : 170 mm
Slump after test : 140 mm
Average slump : 155 mm

Yield (g) value : 3.31 Nm
Plastic viscosity (h) value : 3.82 Nms
Correlation coefficient : 99.2 %

MIX No. : 25 — 40 per cent PFA mix, water-binder ratio = 0.23

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	610.0	170.0	440.0	1.34	10.34
3.5	332	570.0	162.5	407.5	1.17	9.58
3.0	280	540.0	155.0	385.0	0.98	9.05
2.5	226	505.0	147.5	357.5	0.79	8.40
2.0	174	460.0	140.0	320.0	0.61	7.52
1.5	120	415.0	132.5	282.5	0.42	6.64
1.0	70	375.0	125.0	250.0	0.25	5.88

Slump before test : 170 mm
Slump after test : 150 mm
Average slump : 160 mm

Yield (g) value : 4.99 Nm
Plastic viscosity (h) value : 4.07 Nms
Correlation coefficient : 99.3 %

MIX No. : 26 — 15 per cent GGBFS mix, water-binder ratio = 0.26

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	580.0	165.0	415.0	1.34	9.75
3.5	332	545.0	162.5	382.5	1.17	8.99
3.0	280	505.0	155.0	350.0	0.98	8.23
2.5	226	470.0	147.5	322.5	0.79	7.58
2.0	174	430.0	140.0	290.0	0.61	6.82
1.5	120	395.0	132.5	262.5	0.42	6.17
1.0	70	350.0	125.0	225.0	0.25	5.29

Slump before test : 170 mm
Slump after test : 140 mm
Average slump : 155 mm

Yield (g) value : 4.40 Nm
Plastic viscosity (h) value : 3.98 Nms
Correlation coefficient : 99.8 %

MIX No. : 27 — 30 per cent GGBFS mix, water-binder ratio = 0.26

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	585.0	165.0	420.0	1.34	9.87
3.5	332	555.0	162.5	392.5	1.17	9.22
3.0	280	515.0	155.0	360.0	0.98	8.46
2.5	226	480.0	147.5	332.5	0.79	7.81
2.0	174	445.0	140.0	305.0	0.61	7.17
1.5	120	405.0	132.5	272.5	0.42	6.40
1.0	70	365.0	125.0	240.0	0.25	5.64

Slump before test : 160 mm
Slump after test : 140 mm
Average slump : 150 mm

Yield (g) value : 4.77 Nm
Plastic viscosity (h) value : 3.82 Nms
Correlation coefficient : 99.9 %

MIX No. : 28

45 per cent GGBFS mix, water-binder ratio = 0.26

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	590.0	185.0	405.0	1.34	9.52
3.5	332	555.0	177.5	377.5	1.17	8.87
3.0	280	510.0	167.5	342.5	0.98	8.05
2.5	226	470.0	157.5	312.5	0.79	7.34
2.0	174	430.0	147.5	282.5	0.61	6.64
1.5	120	380.0	137.5	242.5	0.42	5.70
1.0	70	345.0	130.0	215.0	0.25	5.05

Slump before test : 155 mm
Slump after test : 130 mm
Average slump : 145 mm

Yield (g) value : 4.03 Nm
Plastic viscosity (h) value : 4.12 Nms
Correlation coefficient : 99.9 %

MIX No. : 29

60 per cent GGBFS mix, water-binder ratio = 0.26

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	555.0	162.5	392.5	1.34	9.22
3.5	332	515.0	155.0	360.0	1.17	8.46
3.0	280	475.0	150.0	325.0	0.98	7.64
2.5	226	440.0	142.5	297.5	0.79	6.99
2.0	174	400.0	137.5	262.5	0.61	6.17
1.5	120	360.0	132.5	227.5	0.42	5.35
1.0	70	320.0	125.0	195.0	0.25	4.58

Slump before test : 170 mm
Slump after test : 150 mm
Average slump : 160 mm

Yield (g) value : 3.59 Nm
Plastic viscosity (h) value : 4.21 Nms
Correlation coefficient : 99.9 %

MIX No. : 30

10 per cent CSF mix, water-binder ratio = 0.38

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	330.0	160.0	170.0	1.34	4.00
3.5	332	310.0	155.0	155.0	1.17	3.64
3.0	280	290.0	147.5	142.5	0.98	3.35
2.5	226	270.0	142.5	127.5	0.79	3.00
2.0	174	250.0	135.0	115.0	0.61	2.70
1.5	120	230.0	127.5	102.5	0.42	2.41
1.0	70	215.0	122.5	92.5	0.25	2.17

Slump before test : 170 mm
Slump after test : 140 mm
Average slump : 155 mm

Yield (g) value : 1.70 Nm
Plastic viscosity (h) value : 1.68 Nms
Correlation coefficient : 99.7 %

MIX No. : 31 — 10 per cent CSF mix, water-binder ratio = 0.35

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	340.0	162.5	177.5	1.34	4.17
3.5	332	320.0	155.0	165.0	1.17	3.88
3.0	280	300.0	150.0	150.0	0.98	3.53
2.5	226	280.0	145.0	135.0	0.79	3.17
2.0	174	260.0	137.5	122.5	0.61	2.88
1.5	120	240.0	130.0	110.0	0.42	2.59
1.0	70	220.0	125.0	95.0	0.25	2.23

Slump before test : 160 mm
Slump after test : 150 mm
Average slump : 155 mm

Yield (g) value : 1.81 Nm
Plastic viscosity (h) value : 1.76 Nms
Correlation coefficient : 99.9 %

MIX No. : 32 — 10 per cent CSF mix, water-binder ratio = 0.32

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	360.0	160.0	200.0	1.34	4.70
3.5	332	335.0	155.0	180.0	1.17	4.23
3.0	280	315.0	147.5	167.5	0.98	3.94
2.5	226	290.0	142.5	147.5	0.79	3.47
2.0	174	270.0	137.5	132.5	0.61	3.11
1.5	120	250.0	130.0	120.0	0.42	2.82
1.0	70	230.0	122.5	107.5	0.25	2.53

Slump before test : 145 mm
Slump after test : 135 mm
Average slump : 140 mm

Yield (g) value : 1.96 Nm
Plastic viscosity (h) value : 1.99 Nms
Correlation coefficient : 99.4 %

MIX No. : 33 — 10 per cent CSF mix, water-binder ratio = 0.29

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	385.0	160.0	225.0	1.34	5.29
3.5	332	360.0	155.0	205.0	1.17	4.82
3.0	280	335.0	147.5	187.5	0.98	4.41
2.5	226	310.0	142.5	167.5	0.79	3.94
2.0	174	285.0	135.0	150.0	0.61	3.53
1.5	120	260.0	127.5	132.5	0.42	3.11
1.0	70	235.0	122.5	112.5	0.25	2.64

Slump before test : 170 mm
Slump after test : 150 mm
Average slump : 160 mm

Yield (g) value : 2.08 Nm
Plastic viscosity (h) value : 2.38 Nms
Correlation coefficient : 99.9 %

MIX No. : 34

5 per cent CSF mix, water-binder ratio = 0.26

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	470.0	157.5	312.5	1.34	7.34
3.5	332	440.0	152.5	287.5	1.17	6.76
3.0	280	420.0	145.0	275.0	0.98	6.46
2.5	226	390.0	140.0	250.0	0.79	5.88
2.0	174	360.0	135.0	225.0	0.61	5.29
1.5	120	330.0	130.0	200.0	0.42	4.70
1.0	70	300.0	125.0	175.0	0.25	4.11

Slump before test : 160 mm
Slump after test : 140 mm
Average slump : 150 mm

Yield (g) value : 3.48 Nm
Plastic viscosity (h) value : 2.93 Nms
Correlation coefficient : 99.3 %

MIX No. : 35

10 per cent CSF mix, water-binder ratio = 0.26

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	460.0	160.0	300.0	1.34	7.05
3.5	332	430.0	155.0	275.0	1.17	6.46
3.0	280	400.0	147.5	252.5	0.98	5.93
2.5	226	370.0	142.5	227.5	0.79	5.35
2.0	174	340.0	137.5	202.5	0.61	4.76
1.5	120	310.0	130.0	180.0	0.42	4.23
1.0	70	280.0	125.0	155.0	0.25	3.64

Slump before test : 150 mm
Slump after test : 120 mm
Average slump : 135 mm

Yield (g) value : 2.90 Nm
Plastic viscosity (h) value : 3.09 Nms
Correlation coefficient : 99.9 %

MIX No. : 36

15 per cent CSF mix, water-binder ratio = 0.26

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	425.0	160.0	265.0	1.34	6.23
3.5	332	400.0	155.0	245.0	1.17	5.76
3.0	280	375.0	150.0	225.0	0.98	5.29
2.5	226	350.0	142.5	207.5	0.79	4.88
2.0	174	320.0	137.5	182.5	0.61	4.29
1.5	120	290.0	130.0	160.0	0.42	3.76
1.0	70	260.0	125.0	135.0	0.25	3.17

Slump before test : 175 mm
Slump after test : 140 mm
Average slump : 160 mm

Yield (g) value : 2.59 Nm
Plastic viscosity (h) value : 2.77 Nms
Correlation coefficient :

MIX No. : 37 GGBFS-CSF mix, water-binder ratio = 0.26

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	460.0	160.0	300.0	1.34	7.05
3.5	332	430.0	155.0	275.0	1.17	6.46
3.0	280	400.0	150.0	250.0	0.98	5.88
2.5	226	375.0	142.5	232.5	0.79	5.46
2.0	174	345.0	135.0	210.0	0.61	4.94
1.5	120	315.0	127.5	187.5	0.42	4.41
1.0	70	285.0	122.5	162.5	0.25	3.82

Slump before test : 160 mm
Slump after test : 140 mm
Average slump : 150 mm

Yield (g) value : 3.16 Nm
Plastic viscosity (h) value : 2.87 Nms
Correlation coefficient : 99.7 %

MIX No. : 38 GGBFS/CSF mix, water-binder ratio = 0.23

Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	550.0	165.0	385.0	1.34	9.05
3.5	332	510.0	157.5	352.5	1.17	8.28
3.0	280	465.0	150.0	315.0	0.98	7.40
2.5	226	435.0	145.0	290.0	0.79	6.82
2.0	174	395.0	137.5	257.5	0.61	6.05
1.5	120	355.0	130.0	225.0	0.42	5.29
1.0	70	315.0	122.5	192.5	0.25	4.52

Slump before test : 170 mm
Slump after test : 140 mm
Average slump : 155 mm

Yield (g) value : 3.53 Nm
Plastic viscosity (h) value : 4.08 Nms
Correlation coefficient : 99.8 %

MIX No. : 39 GGBFS-CSF mix, water-binder ratio = 0.20

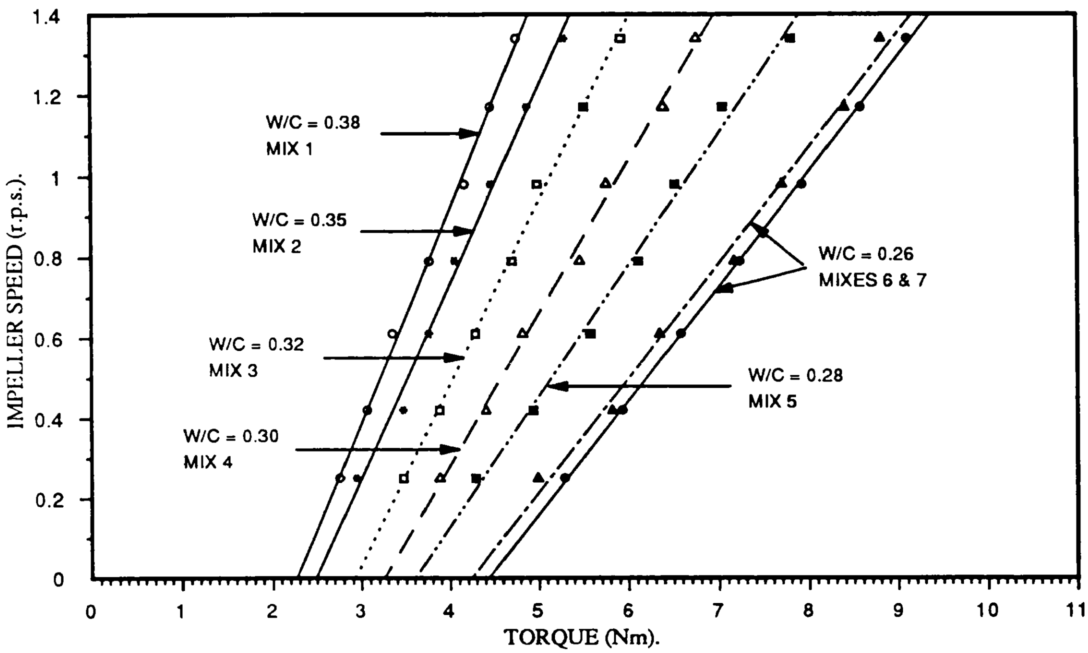
Speed setting	Speed (r.p.m.)	Total pressure (p.s.i.)	Idling pressure (p.s.i.)	Net pressure (p.s.i.)	Impeller speed (r.p.s.)	Torque (Nm)
4.0	383	625.5	155.0	470.0	1.34	11.05
3.5	332	590.0	150.0	440.0	1.17	10.34
3.0	280	550.0	145.0	405.0	0.98	9.52
2.5	226	510.0	140.0	370.0	0.79	8.70
2.0	174	470.0	135.0	335.0	0.61	7.87
1.5	120	425.0	127.5	297.5	0.42	6.99
1.0	70	370.0	122.5	247.5	0.25	5.82

Slump before test : 170 mm
Slump after test : 150 mm
Average slump : 160 mm

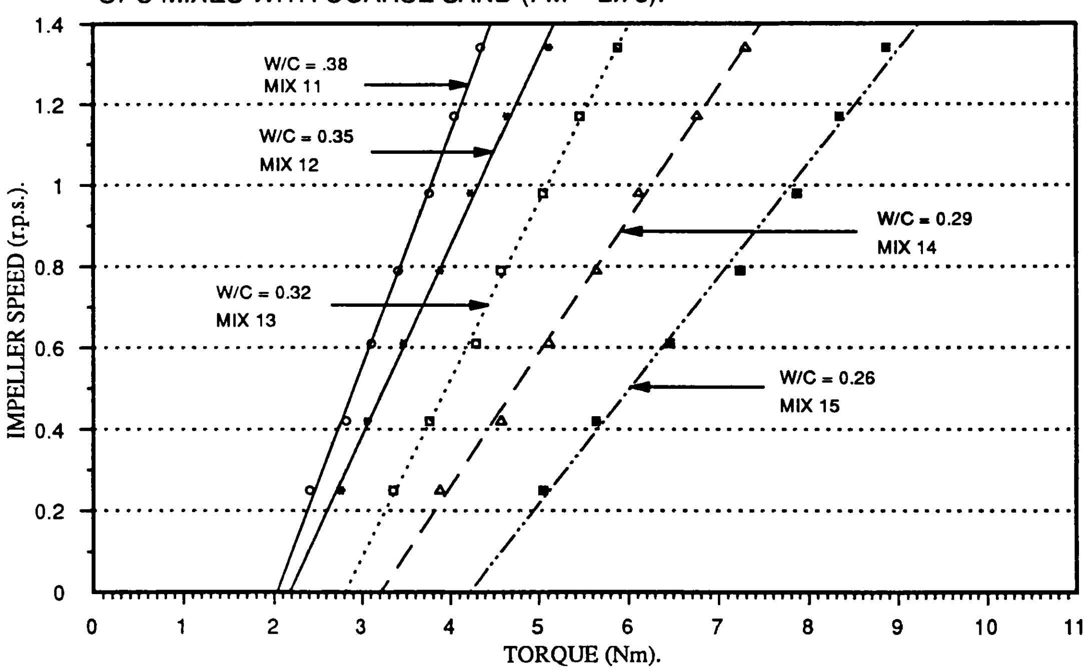
Yield (g) value : 4.89 Nm
Plastic viscosity (h) value : 4.70 Nms
Correlation coefficient : 99.4 %

PART B: DETAILED RESULTS OF TWO POINT WORKABILITY TESTS. (II) IN GRAPHICAL FORM.

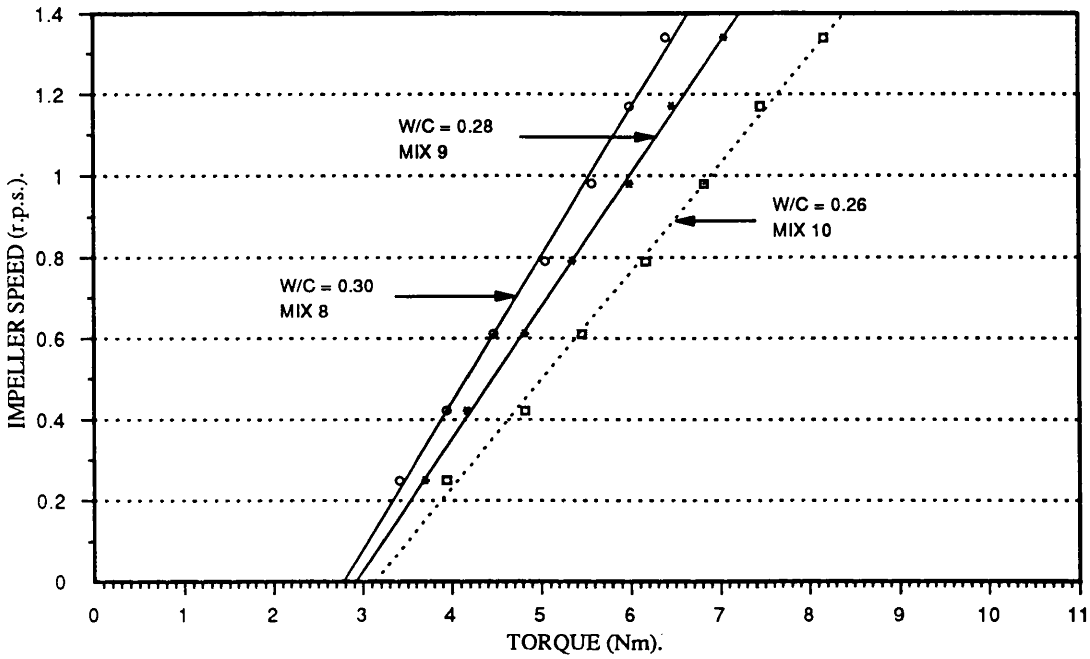
OPC MIXES WITH FINE SAND (FM = 2.1).



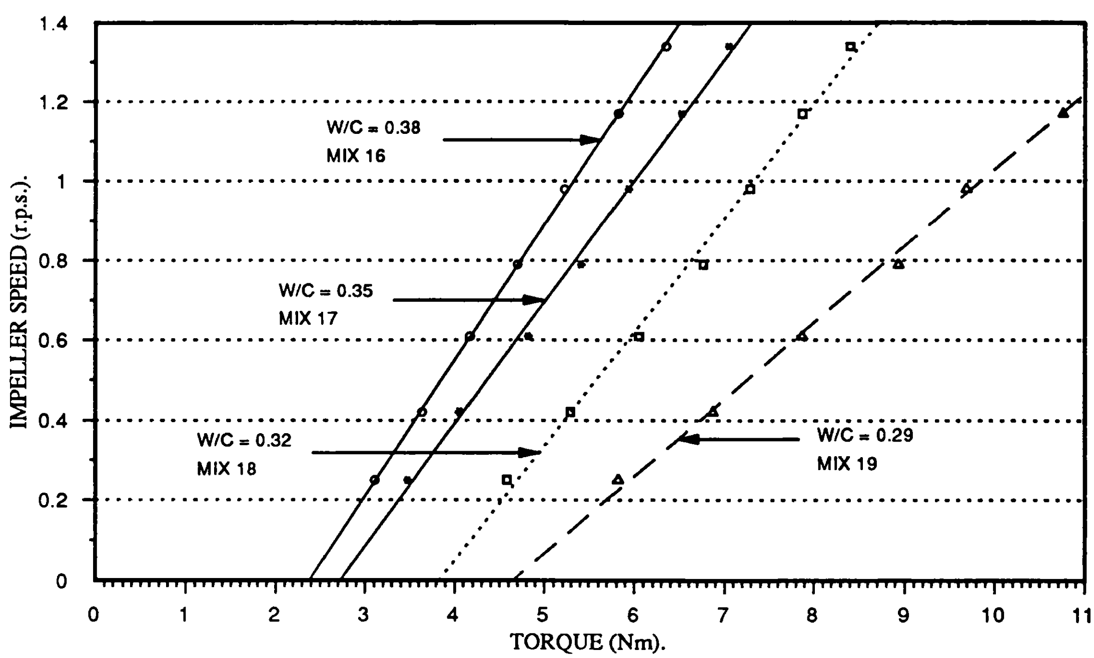
OPC MIXES WITH COARSE SAND (FM = 2.73).

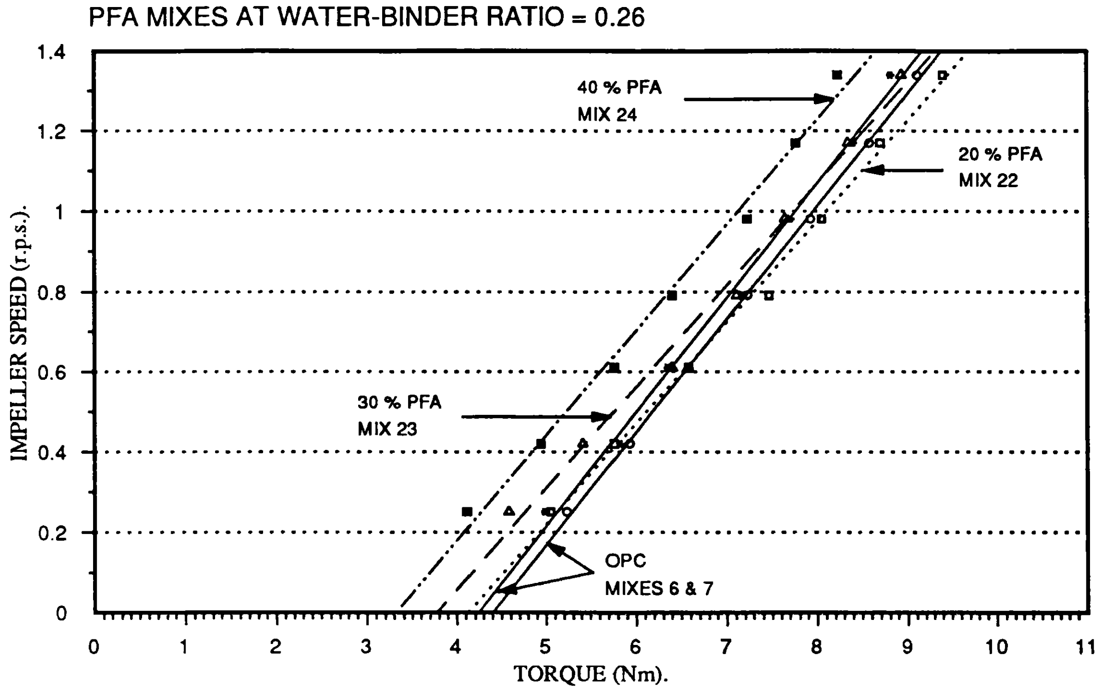


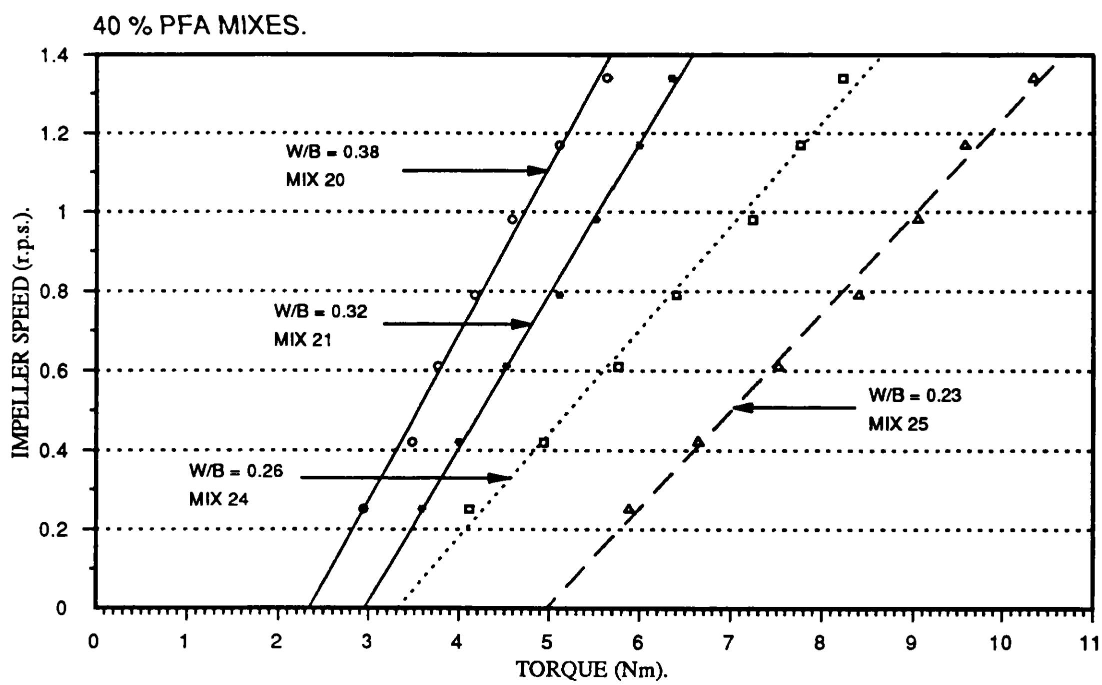
OPC MIXES WITH LOW SAND CONTENT.



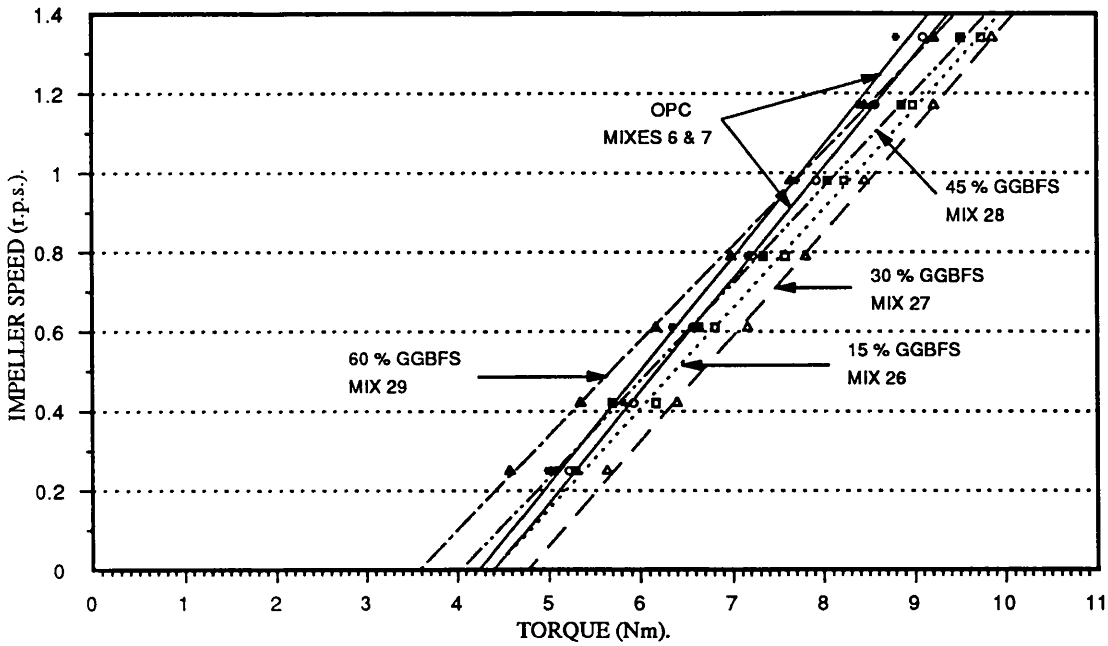
OPC MIXES WITH CONPLAST M1.



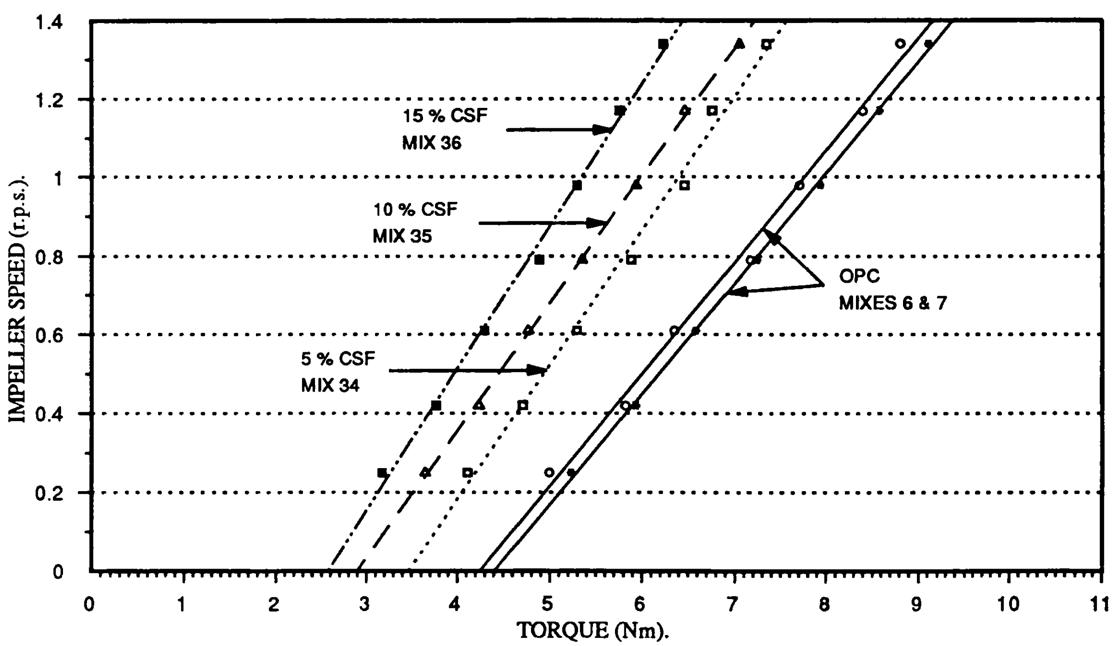

PFA MIXES AT WATER-BINDER RATIO = 0.26
40 % PFA
MIX 24
20 % PFA
MIX 22
30 % PFA
MIX 23
OPC
MIXES 6 & 7
IMPELLER SPEED (r.p.s.).
TORQUE (Nm).

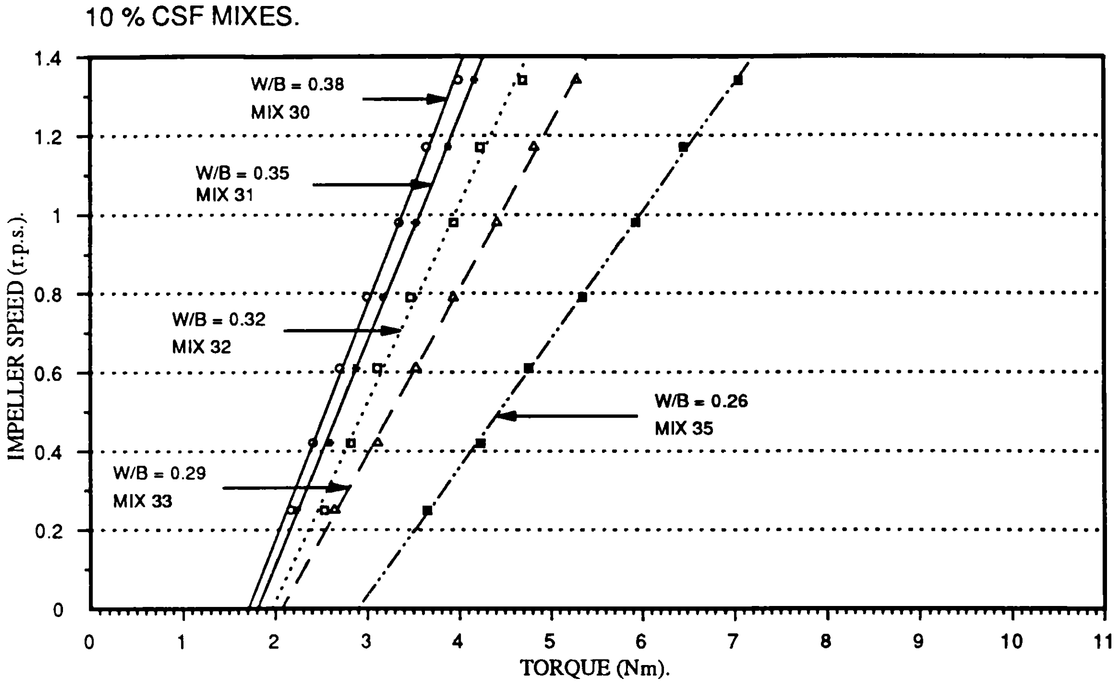

40 % PFA MIXES.
W/B = 0.38
MIX 20
W/B = 0.32
MIX 21
W/B = 0.26
MIX 24
W/B = 0.23
MIX 25
IMPELLER SPEED (r.p.s.).
TORQUE (Nm).

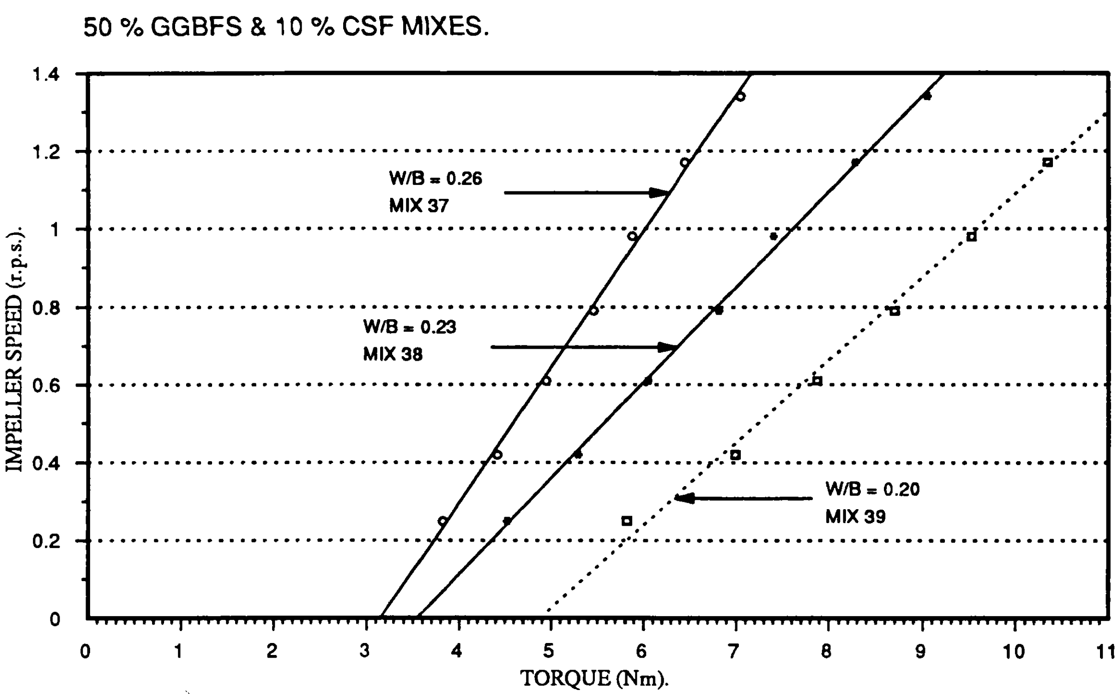
GGBFS MIXES AT WATER-BINDER RATIO = 0.26.



CSF MIXES AT WATER-BINDER RATIO = 0.26




10 % CSF MIXES.
W/B = 0.38
MIX 30
W/B = 0.35
MIX 31
W/B = 0.32
MIX 32
W/B = 0.29
MIX 33
W/B = 0.26
MIX 35
IMPELLER SPEED (r.p.s.).
TORQUE (Nm).


50 % GGBFS & 10 % CSF MIXES.
W/B = 0.26
MIX 37
W/B = 0.23
MIX 38
W/B = 0.20
MIX 39
IMPELLER SPEED (r.p.s.).
TORQUE (Nm).

APPENDIX 5.

COMPRESSIVE STRENGTH RESULTS.

MIX No.	A1.		A2.		A3.		A4.		A5.		A6.	
Cement content (kg/m^3)	470		456		441		429		415		400	
20mm coarse aggregate (kg/m^3)												
10mm coarse aggregate (kg/m^3)	955		955		955		955		955		955	
Fine aggregate (kg/m^3)	860		860		860		860		860		860	
Total water (kg/m^3)	165		170		174		179		183		186	
Superplasticiser dosage[(1)] (%)	2.5		1.5		1.1		0.8		0.5		0.5	
Free water-cement ratio	0.28		0.30		0.32		0.34		0.36		0.38	
Slump (mm)	20		120		130		100		60		75	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
28-days	2381	65.5	2408	68.0	2343	54.0	2370	57.0	2350	50.5	2352	48.0
	2413	71.0	2379	60.5	2354	57.0	2349	52.0	2349	52.2	2335	51.8
	2416	73.0	2405	68.0	2387	57.0	2341	57.5	2333	54.2	2353	50.5
	2396	67.0	2393	67.5	2350	56.0	2369	56.0	2377	55.3	2358	55.4
	2406	71.0	2398	67.5	2352	53.5	2369	53.5	2351	51.0	2346	52.8
Average:	2400	69.5	2395	66.5	2355	55.5	2360	55.0	2350	52.6	2350	51.7

(1) Superplasticiser dosage is given as the percentage of active solids by weight of cement.

Preliminary mixes tested to determine the effect of the aggregate type and size on the compressive strength of concrete. Mixes have been designed according to the "Maximum Density Theory" at 5 per cent overfill of voids by paste.

PART A: 10 mm GRAVEL.

MIX No.	B1.		B2.		B3.		B4.		B5.		B6.	
Cement content (kg/m^3)	450		440		425		410		400		390	
20mm coarse aggregate (kg/m^3)	750		750		750		750		750		750	
10mm coarse aggregate (kg/m^3)	270		270		270		270		270		270	
Fine aggregate (kg/m^3)	700		700		700		700		700		700	
Total water (kg/m^3)	154		160		163		168		171		175	
Superplasticiser dosage[(1)] (%)	2.2		1.0		0.6		0.5		0.2		0.2	
Free water-cement ratio	0.28		0.30		0.32		0.34		0.36		0.38	
Slump (mm)	85		40		75		220		60		90	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
28-days	2405	71.0	2356	63.0	2386	67.0	2357	55.5	2408	61.0	2353	54.0
	2430	77.5	2398	70.5	2410	70.0	2395	60.0	2369	57.0	2344	53.5
	2405	68.0	2367	63.0	2379	58.5	2370	58.0	2377	56.5	2361	53.0
	2413	67.0	2374	64.0	2384	61.5	2377	60.0	2383	57.5	2368	51.0
	2409	62.5	2380	64.0	2395	55.0	2380	55.0	2369	52.5	2398	54.5
Average:	2410	69.0	2375	65.0	2390	62.5	2375	57.5	2380	57.0	2365	53.0

(1) Superplasticiser dosage is given as the percentage of active solids by weight of cement.

Preliminary mixes tested to determine the effect of the aggregate type and size on the compressive strength of concrete. Mixes have been designed according to the "Maximum Density Theory" at 5 per cent overfill of voids by paste.

PART B: 20 mm GRAVEL.

MIX No.	C1.		C2.		C3.		C4.		C5.		C6.	
Cement content (kg/m^3)	462		447		437		423		408		398	
20mm coarse aggregate (kg/m^3)												
10mm coarse aggregate (kg/m^3)	995		995		995		995		995		995	
Fine aggregate (kg/m^3)	815		815		815		815		815		815	
Total water (kg/m^3)	151		156		162		166		169		173	
Superplasticiser dosage[1] (%)	2.7		1.3		1.1		0.9		0.8		0.7	
Free water-cement ratio	0.28		0.30		0.32		0.34		0.36		0.38	
Slump (mm)	60		55		60		55		65		85	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
28-days	2468	82.0	2473	79.5	2415	63.5	2411	66.5	2437	68.0	2347	52.0
	2446	81.5	2453	82.5	2414	68.0	2375	61.5	2410	62.0	2342	54.0
	2503	81.5	2459	77.0	2473	80.0	2400	57.5	2416	66.5	2337	52.5
	2474	84.0	2485	82.0	2410	70.0	2384	65.0	2425	54.5	2337	54.5
	2455	78.5	2460	81.0	2418	69.5	2448	75.5	2437	66.5	2341	55.5
Average:	2470	81.5	2465	80.5	2425	70.0	2405	65.0	2425	63.5	2340	53.5

(1) Superplasticiser dosage is given as the percentage of active solids by weight of cement.

Preliminary mixes tested to determine the effect of the aggregate type and size on the compressive strength of concrete. Mixes have been designed according to the "Maximum Density Theory" at 5 per cent overfill of voids by paste.

PART C: 10 mm LIMESTONE.

MIX No.	D1.		D2.		D3.		D4.		D5.		D6.	
Cement content (kg/m^3)	438		429		410		400		387		376	
20mm coarse aggregate (kg/m^3)	505		505		505		505		505		505	
10mm coarse aggregate (kg/m^3)	515		515		515		515		515		515	
Fine aggregate (kg/m^3)	835		835		835		835		835		835	
Total water (kg/m^3)	144		150		152		156		160		164	
Superplasticiser dosage(1) (%)	2.5		2.0		1.7		1.3		1.2		1.0	
Free water-cement ratio	0.28		0.30		0.32		0.34		0.36		0.38	
Slump (mm)	20		80		95		65		100		85	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
28-days	2469	76.5	2431	75.0	2449	74.5	2394	64.0	2436	66.0	2420	64.0
	2482	78.0	2452	80.0	2427	70.5	2390	64.0	2389	60.0	2403	60.5
	2570	73.0	2448	74.0	2451	69.5	2408	63.0	2438	66.5	2394	58.0
	2439	78.0	2445	75.0	2439	67.0	2429	71.0	2408	65.5	2427	64.5
	2487	79.0	2479	78.0	2430	75.0	2383	65.5	2426	56.5	2456	61.0
Average:	2470	77.0	2450	76.5	2440	71.5	2400	65.5	2420	63.0	2420	61.5

(1) Superplasticiser dosage is given as the percentage of active solids by weight of cement.

Preliminary mixes tested to determine the effect of the aggregate type and size on the compressive strength of concrete. Mixes have been designed according to the "Maximum Density Theory" at 5 per cent overfill of voids by paste.

PART D: 20 mm LIMESTONE.

MIX No.	E1.		E2.		E3.		E4.		E5.		E6.		E7.	
Cement content (kg/m^3)	482		467		452		437		425		412		400	
20mm coarse aggregate (kg/m^3)														
10mm coarse aggregate (kg/m^3)	1015		1015		1015		1015		1015		1015		1015	
Fine aggregate (kg/m^3)	830		830		830		830		830		830		830	
Total water (kg/m^3)	151		156		161		166		170		174		173	
Superplasticiser dosage(1) (%)	2.0		2.0		2.0		1.5		1.2		1.0		0.8	
Free water-cement ratio	0.26		0.28		0.30		0.32		0.34		0.36		0.38	
Slump (mm)	35		60		185		125		95		155		85	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
28-days	2442	76.0	2450	80.5	2464	80.0	2444	81.0	2455	77.0	2406	64.0	2352	56.5
	2423	84.5	2442	83.0	2448	78.5	2458	84.0	2411	70.0	2408	67.0	2367	53.5
	2518	76.0	2445	79.5	2432	74.0	2451	80.0	2399	73.5	2418	66.5	2388	57.0
	2445	82.0	2462	79.5	2436	70.5	2442	77.5	2392	69.0	2420	67.0	2367	59.0
	2416	83.5	2468	86.5	2473	83.0	2446	82.0	2412	73.0	2449	70.0	2347	58.0
Average:	2430	80.5	2455	82.0	2450	77.0	2450	81.0	2415	72.5	2420	67.0	2365	57.0

(1) Superplasticiser dosage is given as the percentage of active solids by weight of cement.

Preliminary mixes tested to determine the effect of the aggregate type and size on the compressive strength of concrete. Mixes have been designed according to the "Maximum Density Theory" at 5 per cent overfill of voids by paste.

PART E: 10 mm GRANITE.

MIX No.	F1.		F2.		F3.		F4.		F5.		F6.	
Cement content (kg/m^3)	448		434		419		407		394		384	
20mm coarse aggregate (kg/m^3)	670		670		670		670		670		670	
10mm coarse aggregate (kg/m^3)	360		360		360		360		360		360	
Fine aggregate (kg/m^3)	850		850		850		850		850		850	
Total water (kg/m^3)	149		154		158		162		166		170	
Superplasticiser dosage[(1)] (%)	2.5		2.0		1.5		1.1		0.9		0.8	
Free water-cement ratio	0.28		0.30		0.32		0.34		0.36		0.38	
Slump (mm)	165		160		155		85		65		75	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
28-days	2469	76.5	2481	85.0	2442	74.0	2401	58.5	2361	61.0	2334	51.5
	2456	79.5	2466	82.0	2405	64.5	2394	63.0	2340	58.5	2367	54.0
	2445	76.0	2464	77.0	2398	66.0	2381	64.0	2353	59.5	2357	52.0
	2439	68.0	2479	80.0	2373	61.0	2404	63.0	2399	57.0	2347	51.0
	2416	75.0	2464	77.5	2398	68.0	2380	62.0	2362	58.0	2411	60.5
Average:	2445	75.0	2470	80.5	2405	66.5	2390	62.0	2365	59.0	2365	54.0

(1) Superplasticiser dosage is given as the percentage of active solids by weight of cement.

Peliminary mixes tested to study the effect of the aggregate type and size on the compressive strength of concrete. Mixes have been designed according to the "Maximum Density Theory" at 5 per cent overfill of voids by paste.

PART F: 20 mm GRANITE.

MIX No.	G1.		G2.		G3.	
Cement content (kg/m³)	457		434		411	
CSF content (kg/m³)			23		46	
Level of cement replacement (%)			5		10	
Total binder content (kg/m³)	457		457		457	
20mm coarse aggregate(1) (kg/m³)	785		785		785	
10mm coarse aggregate(1) (kg/m³)	285		285		285	
Fine aggregate(2) (kg/m³)	735		735		735	
Total water (kg/m³)	148		148		148	
Superplasticiser dosage(3) (%)	5.0		5.0		5.0	
Free water-binder ratio	0.26		0.26		0.26	
Slump (mm):	145		170		190	
	Density kg/m³	Strength N/mm²	Density kg/m³	Strength N/mm²	Density kg/m³	Strength N/mm²
3 days	2397	46.0	2428	52.4	2392	47.7
	2410	46.2	2436	51.8	2375	51.0
	2395	45.8	2406	60.5	2363	51.7
	2398	48.0	2414	58.5	2378	53.0
	2405	45.5	2426	53.5	2390	49.5
Average:	2400	46.3	2420	55.5	2380	50.4
7 days	2394	56.0	2397	70.5	2375	61.0
	2422	53.0	2422	68.0	2379	61.5
	2393	53.2	2408	63.0	2370	59.0
	2410	53.6	2412	67.5	2378	63.0
	2405	53.0	2432	65.0	2367	62.5
Average:	2405	53.8	2415	67.0	2375	61.5
28 days	2415	71.0	2417	82.5	2423	78.0
	2418	65.0	2427	75.0	2396	79.0
	2440	67.0	2429	84.0	2394	78.0
	2408	69.0	2422	79.0	2385	76.0
	2425	63.0	2418	83.5	2379	81.0
Average:	2420	67.0	2425	81.0	2395	78.5
56 days	2420	69.0	2420	80.5	2390	82.0
	2425	67.0	2402	86.0	2394	79.0
	2415	73.0	2449	87.5	2427	87.0
	2440	71.0	2428	89.0	2432	85.0
	2418	70.0	2412	82.5	2398	83.0
Average:	2425	70.0	2420	85.0	2410	83.0

(1) Gravel aggregate was used for these mixes.
(2) The fineness modulus of sand is 2.1
(3) Superplasticiser dosage is given as the percentage of active solids by weight of total binder.

Preliminary mixes tested to determine the effect of CSF on the compressive strength of concrete.

MIX No.	H1.		H2.		H3.	
Cement content (kg/m^3)	439		454		492	
Percentage overfill	6		5		5	
Coarse aggregate[1] (kg/m^3)	1095		1105		1105	
Fine aggregate[2] (kg/m^3)	655		660		660	
Total water (kg/m^3)	190		169		167	
Superplasticiser dosage[3] (%)	0.5		0.8		1.2	
Free water-cement ratio	0.38		0.32		0.29	
Slump (mm):	200		190		165	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
7 days	2424	50.0	2460	73.5	2501	92.0
	2431	50.0	2465	75.5	2497	87.5
	2416	50.0	2464	78.5	2475	88.0
	2318	40.0	2457	76.0	2456	77.5
	2423	47.5	2473	72.0	2495	86.0
Average:	2400	47.5	2465	75.0	2485	86.0
28 days	2346	53.0	2453	87.0	2484	99.0
	2506	63.0	2457	93.0	2509	111.0
	2521	67.0	2454	86.0	2463	90.0
	2400	61.0	2455	93.0	2468	92.5
	2507	64.0	2458	92.0	2475	90.5
Average:	2455	61.5	2455	90.0	2480	96.5
56 days	2427	69.0	2460	89.0	2465	90.0
	2335	56.0	2449	85.0	2489	104.0
	2419	70.0	2445	84.0	2465	100.0
	2327	58.0	2459	86.0	2470	96.0
	2411	69.5	2492	95.0	2471	100.0
Average:	2385	64.5	2460	88.0	2470	98.0
91 days	2427	68.0	2449	87.5	2471	89.0
	2425	67.5	2463	90.0	2474	89.5
	2334	57.5	2472	92.0	2483	97.0
	2453	73.0	2461	90.5	2480	108.0
	2451	73.5	2459	85.0	2480	112.0
Average:	2420	68.0	2460	89.0	2480	99.0
180 days	2383	61.0	2483	102.0		
	2424	72.0	2471	102.0		
	2328	61.0	2500	105.0		
	2335	63.0	2452	87.0		
	2428	74.0	2471	84.0		
Average:	2380	66.0	2475	96.0		

(1) 10 mm granite aggregate was used for these mixes.
(2) The fineness modulus of sand is 2.1
(3) Superplasticiser dosage is given as the percentage of active solids (Conplast 430) by weight of cement.

Preliminary mixes tested to determine the effect of mix design procedure on the compressive strength of OPC concrete. Mixes have been designed according to the "Modified Maximum Density Theory".

MIX No.	H4.		H5.	
Cement content (kg/m³)	510		547	
Percentage overfill	4		5	
Coarse aggregate[1] (kg/m³)	1115		1105	
Fine aggregate[2] (kg/m³)	670		660	
Total water (kg/m³)	157		150	
Superplasticiser/retarder dosage[3] (%)	1.5		2.3/0.14	
Free water-cement ratio	0.26		0.23	
Slump (mm):	190		115	
	Density kg/m³	Strength N/mm²	Density kg/m³	Strength N/mm²
7 days	2488	96.0	2524	96.0
	2508	95.0	2499	97.0
	2500	91.0	2527	97.0
	2503	93.0	2513	98.5
	2498	91.0	2498	96.0
Average:	2500	93.0	2515	97.0
28 days	2513	115.0	2520	112.0
	2504	107.0	2507	105.5
	2493	109.0	2500	112.0
	2499	111.5	2487	102.0
	2516	108.0	2491	102.0
Average:	2505	110.0	2500	106.5
56 days	2510	109.5	2516	103.0
	2500	117.0	2496	108.0
	2505	113.0	2512	102.0
	2517	105.0	2511	118.0
	2498	107.5	2501	110.0
Average:	2505	110.5	2505	108.0
91 days	2504	112.0	2502	110.0
	2507	104.0	2524	108.0
	2521	115.5	2508	106.5
	2509	114.0	2495	112.5
	2526	114.0	2495	112.5
Average:	2515	112.0	2505	110.0
180 days	2512	124.5	2505	107.0
	2524	121.0	2501	109.5
	2500	113.0	2502	114.0
	2505	116.0	2501	97.0
	2500	124.0	2503	108.0
Average:	2510	119.5	2500	107.0

(1) 10 mm granite aggregate was used for these mixes.
(2) The fineness modulus of sand is 2.1
(3) Superplasticiser/retarder dosage is given as the percentage of active solids (Conplast 430/Conplast R) by weight of cement.

Preliminary mixes tested to determine the effect of mix design procedure on the compressive strength of OPC concrete. Mixes have been designed according to the "Modified Maximum Density Theory".

PULVERISED FUEL ASH MIXES : Series 1.

DETAILS OF MIXES:

Free water-binder ratio	= 0.38	
Percentage overfill	= 6	
Total binder content	= 439	kg/m^3
Coarse aggregate content(1)	= 1095	kg/m^3
Fine aggregate content(2)	= 655	kg/m^3
Free water content	= 167	kg/m^3
Total water content	= 190	kg/m^3

MIX No.	H1.		I1.		I2.		I3.	
Level of cement replacement (%):			10		20		40	
Superplasticiser dosage(3) (%)	0.5		0.5		0.5		0.4	
Slump (mm):	200		215		225		225	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
7 days	2424	50.0	2402	39.5	2444	28.7	2419	26.4
	2431	50.0	2425	43.6	2419	42.7	2414	29.6
	2416	50.0	2358	46.6	2425	39.8	2421	29.0
	2318	40.0	2436	45.0	2396	41.5	2423	29.4
	2423	47.5	2390	46.6	2399	41.4	2416	29.6
Average:	2400	47.5	2400	44.3	2415	38.8	2420	28.8
28 days	2346	53.0	2477	61.0	2440	55.5	2414	39.5
	2506	63.0	2506	56.0	2427	53.5	2405	40.6
	2521	67.0	2519	60.0	2428	56.0	2405	37.3
	2400	61.0	2513	57.5	2431	56.0	2399	41.4
	2507	64.0	2503	56.5	2423	54.5	2393	42.2
Average:	2455	61.5	2505	58.0	2430	55.0	2405	40.2
56 days	2427	69.0	2394	67.0	2436	59.5	2404	44.5
	2335	56.0	2400	66.5	2441	63.0	2403	46.0
	2419	70.0	2441	64.0	2452	65.5	2407	45.0
	2327	58.0	2449	65.0	2428	60.0	2421	45.5
	2411	69.5	2398	64.0	2415	63.0	2432	46.3
Average:	2385	64.5	2415	65.5	2435	62.0	2415	45.5
91 days	2427	68.0	2426	67.0	2444	68.0	2429	56.0
	2425	67.5	2449	70.0	2396	70.0	2418	51.5
	2334	57.5	2426	70.0	2394	67.0	2435	56.0
	2453	73.0	2408	64.0	2435	63.0	2411	52.0
	2451	73.5	2430	75.0	2422	69.0	2399	53.0
Average:	2420	68.0	2430	69.0	2420	67.5	2420	54.0
180 days	2383	61.0	2420	75.5	2426	76.0	2412	59.0
	2424	72.0	2421	78.0	2429	75.0	2409	62.0
	2328	61.0	2429	80.0	2432	76.0	2404	62.0
	2335	63.0	2413	71.5	2434	78.0	2421	61.0
	2428	74.0	2441	78.0	2397	78.0	2411	60.0
Average:	2380	66.0	2425	76.5	2425	76.5	2410	61.0

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1
(3) Superplasticiser dosage is given as the percentage of active solids (Conplast 430) by weight of total binder.

PULVERISED FUEL ASH MIXES : Series 2.

DETAILS OF MIXES: Free water-binder ratio = 0.32
Percentage overfill = 5
Total binder content = 454 kg/m³
Coarse aggregate content(1) = 1105 kg/m³
Fine aggregate content(2) = 660 kg/m³
Free water content = 145 kg/m³
Total water content = 169 kg/m³

MIX No.	H2.		I4.		I5.		I6.	
Level of cement replacement (%):			10		20		40	
Superplasticiser dosage(3) (%)	0.8		0.8		0.7		0.6	
Slump (mm):	190		210		220		200	
	Density kg/m³	Strength N/mm²	Density kg/m³	Strength N/mm²	Density kg/m³	Strength N/mm²	Density kg/m³	Strength N/mm²
7 days	2460	73.5	2450	70.5	2460	61.0	2443	40.0
	2465	75.5	2449	67.0	2448	64.0	2443	39.2
	2464	78.5	2450	71.5	2470	71.0	2433	40.0
	2457	76.0	2451	69.0	2454	65.0	2460	44.7
	2473	72.0	2482	80.0	2458	63.0	2437	39.6
Average:	2465	75.0	2455	71.5	2460	65.0	2445	40.7
28 days	2453	87.0	2460	85.0	2453	78.5	2443	60.0
	2457	93.0	2452	80.0	2470	75.0	2456	65.0
	2454	86.0	2468	91.0	2460	81.0	2450	58.5
	2455	93.0	2468	89.5	2453	77.0	2453	65.0
	2458	92.0	2457	84.0	2477	85.0	2454	60.0
Average:	2455	90.0	2460	86.0	2465	79.5	2450	61.5
56 days	2460	89.0	2464	97.0	2474	99.0	2463	69.5
	2449	85.0	2455	91.0	2464	81.0	2445	70.5
	2445	84.0	2458	90.0	2469	93.0	2447	79.0
	2459	86.0	2450	91.0	2454	86.0	2446	78.5
	2492	95.0	2483	95.0	2467	85.5	2445	79.0
Average:	2460	88.0	2460	93.0	2465	89.0	2450	75.5
91 days	2449	87.5	2493	109.0	2469	98.0	2443	78.0
	2463	90.0	2470	104.5	2466	96.0	2450	86.0
	2472	92.0	2460	96.0	2459	92.0	2448	80.5
	2461	90.5	2467	105.0	2465	101.0	2458	88.5
	2459	85.0	2476	103.0	2479	97.0	2446	85.0
Average:	2460	89.0	2475	103.5	2470	97.0	2450	83.5
180 days	2483	102.0	2477	106.0	2476	106.0	2465	90.5
	2471	102.0	2479	110.0	2468	105.0	2464	98.0
	2500	105.0	2476	108.5	2474	108.0	2449	93.0
	2452	87.0	2472	104.0	2462	103.0	2467	98.5
	2471	84.0	2471	110.0	2492	120.0	2467	100.5
Average:	2475	96.0	2475	107.5	2475	108.5	2460	96.0

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1
(3) Superplasticiser dosage is given as the percentage of active solids (Conplast 430) by weight of total binder.

PULVERISED FUEL ASH MIXES : Series 3.

DETAILS OF MIXES: Free water-binder ratio = 0.26
Percentage overfill = 4
Total binder content = 510 kg/m^3
Coarse aggregate content(1) = 1115 kg/m^3
Fine aggregate content(2) = 670 kg/m^3
Free water content = 133 kg/m^3
Total water content = 157 kg/m^3

MIX No.	H4.		I7.		I8.		I9.	
Level of cement replacement (%):			10		20		40	
Superplasticiser dosage(3) (%)	1.5		1.5		1.4		1.2	
Slump (mm):	190		210		220		FLOWING	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
7 days	2488	96.0	2471	82.5	2564	73.5	2437	56.0
	2508	95.0	2491	89.0	2578	77.0	2452	60.0
	2500	91.0	2485	84.0	2477	81.0	2442	56.0
	2503	93.0	2473	85.0	2456	80.0	2439	55.0
	2498	91.0	2415	74.0	2490	81.5	2431	52.0
Average:	2500	93.0	2465	83.0	2515	78.5	2440	56.0
28 days	2513	115.0	2421	90.0	2472	92.5	2448	74.5
	2504	107.0	2417	89.5	2459	92.0	2467	80.0
	2493	109.0	2420	84.5	2462	97.5	2446	78.5
	2499	111.5	2416	88.0	2468	91.0	2448	75.0
	2516	108.0	2421	90.0	2466	96.0	2444	74.0
Average:	2505	110.0	2420	88.5	2465	94.0	2450	76.5
56 days	2510	109.5	2431	97.0	2458	105.0	2455	88.0
	2500	117.0	2430	96.5	2460	102.0	2442	85.5
	2505	113.0	2419	88.0	2464	105.5	2444	84.0
	2517	105.0	2460	98.0	2485	104.0	2453	89.0
	2498	107.5	2423	92.0	2474	96.0	2443	83.0
Average:	2505	110.5	2435	94.5	2470	102.5	2445	86.0
91 days	2504	112.0	2420	103.0	2480	109.0	2447	86.0
	2507	104.0	2484	102.0	2468	101.5	2447	87.0
	2521	115.5	2482	106.0	2478	107.5	2448	96.0
	2509	114.0	2484	105.5	2491	105.5	2440	87.0
	2526	114.0	2447	99.0	2468	104.0	2452	90.5
Average:	2515	112.0	2465	103.0	2475	105.5	2445	89.5
180 days	2512	124.5	2474	115.0	2479	120.5	2456	111.0
	2524	121.0	2470	121.5	2462	105.5	2452	106.0
	2500	113.0	2418	104.0	2467	111.5	2452	92.0
	2505	116.0	2481	111.0	2499	110.5	2455	101.0
	2500	124.0	2479	117.0	2463	115.0	2454	104.0
Average:	2510	119.5	2465	113.5	2475	112.5	2455	103.0

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1
(3) Superplasticiser dosage is given as the percentage of active solids (Conplast 430) by weight of total binder.

GROUND GRANULATED BLAST FURNACE SLAG : Series 1.

DETAILS OF MIXES:

Free water-binder ratio	= 0.38	
Percentage overfill	= 6	
Total binder content	= 439	kg/m^3
Coarse aggregate content(1)	= 1095	kg/m^3
Fine aggregate content(2)	= 655	kg/m^3
Free water content	= 167	kg/m^3
Total water content	= 190	kg/m^3

MIX No.	H1.		J1.		J2.		J3.	
Level of cement replacement (%):			10		30		60	
Superplasticiser dosage(3) (%)	0.5		0.5		0.4		0.3	
Slump (mm):	200		220		195		150	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
7 days	2424	50.0	2417	44.3	2373	34.0	2432	34.4
	2431	50.0	2432	50.0	2373	33.6	2424	31.3
	2416	50.0	2432	51.0	2386	35.4	2439	37.0
	2318	40.0	2435	49.0	2393	37.0	2446	36.2
	2423	47.5	2416	43.6	2399	35.7	2428	35.9
Average:	2400	47.5	2425	47.6	2385	35.1	2435	35.0
28 days	2346	53.0	2416	59.0	2368	48.5	2430	48.6
	2506	63.0	2526	62.0	2370	51.0	2451	49.8
	2521	67.0	2427	56.0	2411	48.9	2447	50.6
	2400	61.0	2422	56.0	2409	48.1	2438	49.2
	2507	64.0	2444	59.5	2410	52.0	2428	48.0
Average:	2455	61.5	2425	58.5	2395	49.7	2440	49.2
56 days	2427	69.0	2434	63.0	2423	53.0	2430	54.0
	2335	56.0	2445	75.5	2367	54.0	2425	55.0
	2419	70.0	2430	71.0	2378	58.0	2438	61.0
	2327	58.0	2419	60.5	2407	56.0	2435	55.0
	2411	69.5	2422	63.0	2412	57.0	2435	66.0
Average:	2385	64.5	2430	66.5	2400	55.5	2435	58.0
91 days	2427	68.0	2441	76.5	2420	57.0	2431	66.0
	2425	67.5	2412	64.0	2423	60.5	2421	61.0
	2334	57.5	2419	67.0	2454	63.0	2429	60.0
	2453	73.0	2435	71.5	2423	60.5	2428	64.0
	2451	73.5	2429	66.0	2443	63.5	2429	59.0
Average:	2420	68.0	2425	69.0	2435	61.0	2430	62.0
180 days	2383	61.0	2428	69.5	2411	67.5	2432	77.5
	2424	72.0	2435	79.0	2412	65.5	2440	68.0
	2328	61.0	2425	72.0	2440	68.0	2447	77.0
	2335	63.0	2435	72.5	2418	63.0	2433	69.0
	2428	74.0	2433	70.0	2424	66.5	2450	70.0
Average:	2380	66.0	2430	72.5	2420	66.0	2440	72.5

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1
(3) Superplasticiser dosage is given as the percentage of active solids (Conplast 430) by weight of total binder.

GROUND GRANULATED BLAST FURNACE SLAG MIXES : Series 2.

DETAILS OF MIXES: Free water-binder ratio = 0.32
Percentage overfill = 5
Total binder content = 454 kg/m^3
Coarse aggregate content(1) = 1105 kg/m^3
Fine aggregate content(2) = 660 kg/m^3
Free water content = 145 kg/m^3
Total water content = 169 kg/m^3

MIX No.	H2.		J4.		J5.		J6.	
Level of cement replacement (%):			10		30		60	
Superplasticiser dosage(3) (%)	0.8		0.8		0.7		0.6	
Slump (mm):	190		200		160		130	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
7 days	2460	73.5	2455	69.5	2486	67.0	2458	52.8
	2465	75.5	2459	70.5	2475	66.0	2463	47.0
	2464	78.5	2460	65.0	2481	73.0	2460	52.0
	2457	76.0	2458	73.0	2466	65.5	2454	47.0
	2473	72.0	2460	74.0	2474	72.0	2455	45.1
Average:	2465	75.0	2460	70.5	2475	69.0	2460	48.8
28 days	2453	87.0	2469	91.0	2487	87.5	2481	86.0
	2457	93.0	2463	87.5	2476	86.0	2469	74.0
	2454	86.0	2481	96.0	2497	93.0	2458	73.0
	2455	93.0	2473	89.0	2499	93.0	2465	76.0
	2458	92.0	2463	86.5	2483	83.0	2457	74.0
Average:	2455	90.0	2470	90.0	2490	88.5	2465	76.5
56 days	2460	89.0	2468	97.0	2485	100.5	2473	91.0
	2449	85.0	2463	93.0	2469	95.5	2486	92.0
	2445	84.0	2485	97.0	2476	90.0	2470	90.0
	2459	86.0	2468	105.5	2473	92.0	2471	86.0
	2492	95.0	2466	97.0	2475	102.5	2467	95.0
Average:	2460	88.0	2470	98.0	2475	96.0	2475	91.0
91 days	2449	87.5	2462	97.0	2477	102.0	2467	96.0
	2463	90.0	2454	91.0	2487	98.0	2466	91.0
	2472	92.0	2465	97.0	2489	109.0	2463	87.0
	2461	90.5	2490	103.0	2479	104.0	2469	100.0
	2459	85.0	2483	107.0	2487	105.0	2472	92.0
Average:	2460	89.0	2470	99.0	2485	103.5	2465	93.0
180 days	2483	102.0	2465	104.0	2492	96.0	2474	89.0
	2471	102.0	2471	99.5	2487	105.0	2470	99.0
	2500	105.0	2471	91.0	2485	97.0	2465	94.0
	2452	87.0	2473	95.0	2485	104.0	2472	97.0
	2471	84.0	2496	109.5	2484	101.5	2487	90.0
Average:	2475	96.0	2475	100.0	2485	100.5	2475	94.0

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1
(3) Superplasticiser dosage is given as the percentage of active solids (Conplast 430) by weight of total binder.

GROUND GRANULATED BLAST FURNACE SLAG MIXES : Series 3.

DETAILS OF MIXES:

Free water-binder ratio	= 0.26	
Percentage overfill	= 4	
Total binder content	= 510	kg/m^3
Coarse aggregate content(1)	= 1115	kg/m^3
Fine aggregate content(2)	= 670	kg/m^3
Free water content	= 133	kg/m^3
Total water content	= 157	kg/m^3

MIX No.	H4.		J7.		J8.		J9.	
Level of cement replacement (%):			10		30		60	
Superplasticiser dosage(3) (%)	1.5		1.5		1.5		1.5	
Slump (mm):	190		135		205		220	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
7 days	2488	96.0	2410	79.0	2460	81.0	2470	51.5
	2508	95.0	2429	80.5	2498	87.0	2459	57.5
	2500	91.0	2403	77.0	2485	85.0	2475	50.5
	2503	93.0	2412	77.5	2487	83.0	2467	58.0
	2498	91.0	2481	88.5	2497	85.0	2484	53.0
Average:	2500	93.0	2425	80.5	2485	84.0	2470	54.0
28 days	2513	115.0	2492	102.0	2473	104.5	2480	97.0
	2504	107.0	2475	101.0	2481	109.0	2492	85.5
	2493	109.0	2475	105.0	2462	96.0	2468	93.0
	2499	111.5	2476	106.5	2488	105.0	2497	86.5
	2516	108.0	2507	110.0	2496	112.0	2482	85.0
Average:	2505	110.0	2485	105.0	2480	105.0	2485	89.5
56 days	2510	109.5	2406	97.0	2496	99.0	2498	106.0
	2500	117.0	2417	95.0	2466	104.0	2488	96.0
	2505	113.0	2480	108.0	2488	117.0	2487	95.0
	2517	105.0	2482	106.0	2475	106.5	2492	97.0
	2498	107.5	2485	107.0	2463	105.0	2477	96.5
Average:	2505	110.5	2455	102.5	2480	106.5	2490	98.0
91 days	2504	112.0	2480	106.5	2462	113.5	2489	92.0
	2507	104.0	2399	97.0	2467	103.5	2490	106.0
	2521	115.5	2411	92.5	2491	111.0	2485	95.5
	2509	114.0	2411	93.0	2504	110.5	2467	99.0
	2526	114.0	2487	102.0	2483	113.0	2490	105.0
Average:	2515	112.0	2440	98.0	2480	110.5	2485	99.5
180 days	2512	124.5	2475	109.5	2484	120.0	2499	96.5
	2524	121.0	2420	99.0	2494	111.0	2483	114.0
	2500	113.0	2406	100.0	2462	113.0	2485	103.0
	2505	116.0	2473	107.0	2460	118.0	2487	95.0
	2500	124.0	2475	108.5	2481	114.0	2461	108.5
Average:	2510	119.5	2450	105.0	2475	115.0	2485	103.5

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1
(3) Superplasticiser dosage is given as the percentage of active solids (Conplast 430) by weight of total binder.

CONDENSED SILICA FUME MIXES : Series 1.

DETAILS OF MIXES:

Free water-binder ratio	= 0.38	
Percentage overfill	= 6	
Total binder content	= 439	kg/m^3
Coarse aggregate content(1)	= 1095	kg/m^3
Fine aggregate content(2)	= 655	kg/m^3
Free water content	= 167	kg/m^3
Total water content	= 190	kg/m^3

MIX No.	H1.		K1.		K2.		K3.	
Level of cement replacement (%):			5		10		15	
Superplasticiser dosage(3) (%)	0.5		0.5		0.7		0.9	
Slump (mm):	200		110		185		170	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
7 days	2424	50.0	2420	56.0	2453	60.5	2439	55.0
	2431	50.0	2409	58.0	2422	58.0	2434	56.0
	2416	50.0	2412	56.0	2432	57.0	2440	60.5
	2318	40.0	2409	56.0	2443	57.5	2447	58.0
	2423	47.5	2420	56.0	2415	56.5	2430	56.5
Average:	2400	47.5	2415	56.5	2435	58.0	2440	57.0
28 days	2346	53.0	2413	72.0	2450	87.0	2447	90.0
	2506	63.0	2410	76.0	2416	81.0	2431	86.5
	2521	67.0	2425	75.5	2431	86.0	2432	89.0
	2400	61.0	2416	79.0	2429	86.0	2428	87.5
	2507	64.0	2435	77.5	2439	85.5	2429	88.0
Average:	2455	61.5	2420	76.0	2435	85.0	2435	88.0
56 days	2427	69.0	2449	81.0	2432	96.5	2435	95.0
	2335	56.0	2434	85.0	2435	102.0	2441	93.5
	2419	70.0	2403	80.0	2428	95.5	2440	92.5
	2327	58.0	2434	86.0	2424	83.0	2445	97.5
	2411	69.5	2411	72.0	2427	89.0	2437	99.0
Average:	2385	64.5	2425	81.0	2430	93.0	2440	95.5
91 days	2427	68.0	2439	85.0	2451	105.5	2437	100.5
	2425	67.5	2406	83.0	2428	102.5	2459	105.0
	2334	57.5	2444	89.5	2441	107.0	2444	106.0
	2453	73.0	2407	79.5	2436	103.5	2452	103.0
	2451	73.5	2408	78.0	2411	98.0	2443	110.0
Average:	2420	68.0	2420	83.0	2435	103.5	2445	105.0
180 days	2383	61.0	2426	85.0	2448	107.0	2433	111.0
	2424	72.0	2426	72.0	2433	101.0	2438	109.0
	2328	61.0	2425	89.0	2434	103.0	2449	113.0
	2335	63.0	2412	92.0	2457	99.0	2432	108.0
	2428	74.0	2418	91.0	2446	103.5	2434	107.0
Average:	2380	66.0	2420	86.0	2445	102.5	2435	109.5

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1
(3) Superplasticiser dosage is given as the percentage of active solids (Conplast 430) by weight of total binder.

CONDENSED SILICA FUME MIXES : Series 2.

DETAILS OF MIXES:

Free water-binder ratio	= 0.32	
Percentage overfill	= 5	
Total binder content	= 454	kg/m^3
Coarse aggregate content(1)	= 1105	kg/m^3
Fine aggregate content(2)	= 660	kg/m^3
Free water content	= 145	kg/m^3
Total water content	= 169	kg/m^3

MIX No.	H2.		K4.		K5.		K6.	
Level of cement replacement (%):			5		10		15	
Superplasticiser dosage(3) (%)	0.8		0.8		0.7		1.1	
Slump (mm):	190		190		90		170	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
7 days	2460	73.5	2452	77.0	2506	85.0	2473	80.0
	2465	75.5	2454	76.0	2480	81.0	2471	83.5
	2464	78.5	2472	78.0	2473	82.0	2474	78.0
	2457	76.0	2455	76.5	2481	81.0	2484	81.0
	2473	72.0	2463	80.0	2500	81.5	2500	84.0
Average:	2465	75.0	2460	77.5	2490	82.0	2480	81.5
28 days	2453	87.0	2451	94.5	2490	113.0	2484	110.0
	2457	93.0	2484	91.0	2463	98.0	2488	106.5
	2454	86.0	2452	93.5	2465	105.0	2478	102.5
	2455	93.0	2455	99.0	2489	109.0	2468	110.0
	2458	92.0	2451	94.0	2466	105.5	2481	109.0
Average:	2455	90.0	2460	94.5	2475	106.0	2480	107.5
56 days	2460	89.0	2456	106.0	2472	100.0	2474	120.0
	2449	85.0	2479	112.0	2472	102.5	2462	112.0
	2445	84.0	2479	111.0	2485	118.0	2484	118.0
	2459	86.0	2466	105.0	2470	113.0	2468	118.0
	2492	95.0	2470	111.0	2459	118.0	2482	121.0
Average:	2460	88.0	2470	109.0	2470	110.5	2475	118.0
91 days	2449	87.5	2482	118.5	2474	113.0	2476	129.0
	2463	90.0	2486	113.0	2486	130.0	2475	115.0
	2472	92.0	2488	114.5	2502	103.0	2471	127.0
	2461	90.5	2493	117.0	2476	120.0	2467	122.5
	2459	85.0	2452	110.0	2495	116.0	2477	118.0
Average:	2460	89.0	2480	114.5	2485	116.5	2475	122.5
180 days	2483	102.0	2481	118.5	2473	124.0	2478	123.5
	2471	102.0	2479	111.5	2492	113.0	2483	128.0
	2500	105.0	2483	114.0	2480	123.0	2482	131.0
	2452	87.0	2494	111.0	2478	120.5	2495	122.0
	2471	84.0	2461	111.0	2474	111.0	2494	137.0
Average:	2475	96.0	2480	113.0	2480	118.5	2485	128.5

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1
(3) Superplasticiser dosage is given as the percentage of active solids (Conplast 430) by weight of total binder.

CONDENSED SILICA FUME MIXES : Series 3.

DETAILS OF MIXES:		
	Free water-binder ratio	= 0.29
	Percentage overfill	= 5
	Total binder content	= 492 kg/m³
	Coarse aggregate content(1)	= 1105 kg/m³
	Fine aggregate content(2)	= 660 kg/m³
	Free water content	= 143 kg/m³
	Total water content	= 167 kg/m³

MIX No.	H3.		K7.		K8.		K9.	
Level of cement replacement (%):			5		10		15	
Superplasticiser dosage(3) (%)	1.2		1.0		1.0		1.2	
Slump (mm):	165		170		140		170	
	Density kg/m³	Strength N/mm²	Density kg/m³	Strength N/mm²	Density kg/m³	Strength N/mm²	Density kg/m³	Strength N/mm²
7 days	2501	92.0	2493	90.0	2467	77.0	2494	91.0
	2497	87.5	2471	83.5	2454	78.0	2472	86.0
	2475	88.0	2495	89.5	2461	82.0	2492	81.0
	2456	77.5	2465	83.0	2470	85.0	2475	86.5
	2495	86.0	2473	83.0	2472	86.5	2478	88.5
Average:	2485	86.0	2480	86.0	2465	81.5	2480	86.5
28 days	2484	99.0	2462	97.0	2500	115.5	2480	118.5
	2509	111.0	2494	106.5	2481	113.5	2469	113.0
	2463	90.0	2474	92.0	2491	120.5	2475	121.0
	2468	92.5	2493	107.0	2477	115.0	2478	119.0
	2475	90.5	2462	95.0	2456	107.5	2476	121.5
Average:	2480	96.5	2475	99.5	2480	114.5	2475	118.5
56 days	2465	90.0	2472	101.0	2460	115.0	2498	129.5
	2489	104.0	2470	112.0	2456	117.0	2495	128.0
	2465	100.0	2469	107.5	2464	116.0	2486	112.0
	2470	96.0	2507	117.0	2479	119.0	2474	125.0
	2471	100.0	2469	99.0	2468	119.0	2479	127.5
Average:	2470	98.0	2475	107.5	2465	117.0	2485	124.5
91 days	2471	89.0	2467	105.0	2473	130.0	2496	133.0
	2474	89.5	2491	117.0	2497	130.0	2493	137.0
	2483	97.0	2469	110.5	2474	130.5	2474	133.0
	2480	108.0	2492	129.0	2455	116.0	2481	125.0
	2480	112.0	2501	118.0	2451	119.0	2482	135.0
Average:	2480	99.0	2485	116.0	2470	125.0	2485	132.5

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1
(3) Superplasticiser dosage is given as the percentage of active solids (Conplast 430) by weight of total binder.

CONDENSED SILICA FUME MIXES : Series 4.

DETAILS OF MIXES: Free water-binder ratio = 0.26
Percentage overfill = 4
Total binder content = 510 kg/m^3
Coarse aggregate content[1] = 1115 kg/m^3
Fine aggregate content[2] = 670 kg/m^3
Free water content = 133 kg/m^3
Total water content = 157 kg/m^3

MIX No.	H4.		K10.		K11.		K12.	
Level of cement replacement (%):			5		10		15	
Superplasticiser dosage[3] (%)	1.5		1.1		0.9		1.3	
Slump (mm):	190		175		75		145	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
7 days	2488	96.0	2483	87.0	2505	95.0	2484	85.0
	2508	95.0	2496	92.0	2487	87.5	2479	86.0
	2500	91.0	2465	88.5	2509	90.0	2512	95.0
	2503	93.0	2474	90.5	2485	88.0	2469	88.0
	2498	91.0	2479	87.0	2513	95.0	2474	91.0
Average:	2500	93.0	2480	89.0	2500	91.0	2485	89.0
28 days	2513	115.0	2502	99.5	2510	109.0	2482	117.0
	2504	107.0	2504	107.0	2496	118.5	2480	116.5
	2493	109.0	2512	115.0	2506	111.0	2472	109.5
	2499	111.5	2475	118.0	2501	112.0	2468	109.0
	2516	108.0	2479	111.5	2495	117.0	2496	124.0
Average:	2505	110.0	2495	110.0	2500	113.5	2480	115.0
56 days	2510	109.5	2494	121.0	2500	126.0	2494	134.0
	2500	117.0	2510	115.0	2490	126.0	2477	130.5
	2505	113.0	2511	114.5	2512	131.5	2474	134.0
	2517	105.0	2479	104.0	2526	129.0	2476	131.5
	2498	107.5	2504	122.0	2501	134.0	2466	121.5
Average:	2505	110.5	2500	115.5	2505	129.5	2475	130.5
91 days	2504	112.0	2495	113.0	2494	138.0	2479	126.0
	2507	104.0	2476	118.5	2486	131.0	2477	135.0
	2521	115.5	2483	128.5	2488	131.0	2497	137.0
	2509	114.0	2483	125.5	2496	130.0	2473	125.0
	2526	114.0	2504	121.5	2504	132.5	2476	127.5
Average:	2515	112.0	2490	121.5	2495	132.5	2480	130.0
180 days	2512	124.5	2487	128.0	2490	135.5	2500	136.5
	2524	121.0	2507	123.0	2482	133.0	2491	147.0
	2500	113.0	2500	133.5	2512	131.0	2474	135.0
	2505	116.0	2487	129.0	2484	124.0	2498	137.0
	2500	124.0	2473	126.0	2500	141.0	2472	141.5
Average:	2510	119.5	2490	128.0	2495	133.0	2485	139.5

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1
(3) Superplasticiser dosage is given as the percentage of active solids (Conplast 430) by weight of total binder.

CONDENSED SILICA FUME MIXES : Series 5.

DETAILS OF MIXES:

Free water-binder ratio	= 0.23	
Percentage overfill	= 5	
Total binder content	= 547	kg/m^3
Coarse aggregate content(1)	= 1105	kg/m^3
Fine aggregate content(2)	= 660	kg/m^3
Free water content	= 126	kg/m^3
Total water content	= 150	kg/m^3

MIX No.	H5.		K13.		K14.		K15.	
Level of cement replacement (%):			5		10		15	
Superplasticiser dosage(3) (%)	2.3		1.4		1.5		1.7	
Retarder dosage(3) (%)	0.14		0.125		0.125		0.125	
Slump (mm):	115		165		155		155	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
7 days	2524	96.0	2487	104.0	2582	99.5	2482	99.0
	2499	97.0	2462	92.0	2500	97.0	2491	100.0
	2527	97.0	2461	92.5	2480	101.5	2479	97.5
	2513	98.5	2475	104.0	2487	100.0	2506	103.0
	2498	96.0	2481	100.0	2458	93.0	2487	105.0
Average:	2515	97.0	2475	98.5	2500	98.0	2490	101.0
28 days	2520	112.0	2496	112.5	2473	120.0	2493	123.0
	2507	105.5	2471	116.0	2467	116.5	2496	121.5
	2500	112.0	2462	110.0	2447	104.0	2497	126.5
	2487	102.0	2457	105.0	2475	120.5	2486	128.0
	2491	102.0	2466	113.0	2498	119.0	2483	126.0
Average:	2500	106.5	2470	111.5	2470	116.0	2490	125.0
56 days	2516	103.0	2481	123.0	2477	123.0	2480	136.0
	2496	108.0	2480	130.0	2483	131.0	2490	135.0
	2512	102.0	2463	121.0	2478	124.0	2485	129.0
	2511	118.0	2466	108.0	2505	134.5	2491	137.5
	2501	110.0	2498	129.0	2460	122.5	2515	136.0
Average:	2505	108.0	2480	122.0	2480	127.0	2490	134.5
91 days	2502	110.0	2466	119.0	2481	134.0	2477	154.5
	2524	108.0	2487	133.0	2460	124.5	2487	148.5
	2508	106.5	2494	129.0	2485	122.0	2476	134.5
	2495	112.5	2479	125.0	2502	153.0	2500	136.0
	2495	112.5	2482	121.0	2465	124.0	2485	141.0
Average:	2505	110.0	2480	125.5	2480	131.5	2485	143.0
180 days	2505	107.0	2479	136.0	2463	136.5	2493	124.0
	2501	109.5	2485	139.5	2468	131.0	2483	143.0
	2502	114.0	2481	126.0	2518	152.0	2487	129.0
	2501	97.0	2508	141.0	2477	136.0	2487	156.0
	2503	108.0	2514	134.0	2488	137.0	2476	146.0
Average:	2500	107.0	2495	135.5	2485	138.5	2485	139.5

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1
(3) Superplasticiser and retarder dosages are given as the percentage of active solids (Conplast 430 & Conplast R) by weight of total binder.

Combination of mineral admixtures at a water-binder ratio of 0.38. Series 1.

MIX No.	L1.		L2.	
Cement content	167		250	
PFA			167	
GGBFS	250			
CSF	22		22	
Level of cement replacement	5		5	
Total binder content	439		439	
Coarse aggregate	1095		1095	
Fine aggregate	655		655	
Free water				
Total water	190		190	
Superplasticiser dosage[3] (%)	0.5		0.5	
Slump (mm):	180		140	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
7 days	2409	26.0	2425	29.2
	2440	30.6	2434	30.0
	2450	31.0	2420	29.8
	2410	27.6	2430	30.8
	2409	29.4	2434	31.4
Average:	2425	28.9	2430	30.2
28 days	2433	58.5	2413	46.8
	2425	53.5	2416	48.5
	2437	56.5	2417	50.2
	2417	54.5	2405	49.1
	2413	52.0	2414	49.4
Average:	2425	55.0	2415	48.8
56 days	2431	72.0	2426	64.0
	2420	71.0	2410	61.0
	2414	72.5	2422	60.0
	2434	74.0	2418	62.5
	2430	75.0	2417	62.5
Average:	2425	73.0	2420	62.0
91 days	2415	78.5	2418	72.0
	2431	84.5	2421	71.0
	2435	78.0	2423	68.5
	2442	82.0	2416	69.0
	2421	81.0	2435	74.0
Average:	2430	81.0	2425	71.0
180 days	2588	84.0	2428	76.0
	2490	85.0	2435	75.5
	2526	75.5	2417	73.5
	2592	82.0	2427	75.0
	2553	78.0	2430	78.0
Average:	2550	81.0	2425	75.5

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1
(3) Superplasticiser and retarder dosages are given as the percentage of active solids by weight of total binder.

Combination of mineral admixtures at a water-binder ratio of 0.26. Series 2.

MIX No.	L3.		L4.		L5.	
Cement content	194		184		275	
PFA					184	
GGBFS	290		275			
CSF	26		51		51	
Level of cement replacement	GGBFS = 57 CSF = 5		GGBFS = 54 CSF = 10		PFA = 36 CSF = 10	
Total binder content	510		510		510	
Coarse aggregate	1115		1115		1115	
Fine aggregate	670		670		670	
Free water						
Total water	157		157		157	
Superplasticiser dosage(3) (%)	1.1		1.0		1.0	
Slump (mm):	FLOWING		210		220	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
7 days	2476	61.5	2454	53.0	2455	56.5
	2476	60.0	2463	54.0	2452	54.0
	2478	63.0	2462	52.0	2458	60.0
	2473	63.5	2454	53.0	24443	56.0
	2479	64.0	2474	56.0	2455	60.0
Average:	2475	62.5	2460	53.5	2455	57.5
28 days	2484	93.0	2505	96.0	2463	92.0
	2503	93.0	2484	96.0	2477	93.0
	2508	98.0	2466	85.0	2459	90.0
	2488	88.5	2487	88.0	2447	87.0
	2481	94.0	2490	104.0	2466	88.0
Average:	2495	93.5	2485	94.0	2460	90.0
56 days	2484	109.5	2483	92.5	2463	109.0
	2479	106.5	2486	108.0	2471	107.5
	2482	100.0	2482	111.5	2490	108.0
	2482	106.0	2465	107.0	2476	108.0
	2496	107.0	2466	104.0	2455	103.5
Average:	2485	107.0	2475	104.5	2470	107.0
91 days	2479	113.0	2465	105.0	2469	113.0
	2505	122.0	2464	105.0	2450	102.5
	2484	100.0	2494	128.0	2450	103.0
	2480	114.0	2465	105.5	2448	110.0
	2485	102.5	2479	111.0	2453	103.5
Average:	2485	110.5	2475	111.0	2455	106.5
180 days	2477	115.0	2359	126.0	2476	117.5
	2485	112.5	2417	106.0	2457	112.0
	2508	114.5	2368	116.0	2456	114.0
	2481	103.0	2345	103.0	2462	122.0
	2515	129.0	2377	106.0	2481	116.5
Average:	2495	115.0	2373	111.5	2465	116.5

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1
(3) Superplasticiser and retarder dosages are given as the percentage of active solids by weight of total binder.

Combination of mineral admixtures at a water-binder ratio of 0.20. Series 3.

MIX No.	L6.		L7.		L8.		L9.	
Cement content	531		236		354		354	
PFA							177	
GGBFS			295		177			
CSF	59		59		59		59	
Level of cement replacement	CSF = 10		GGBFS = 50 CSF = 10		GGBFS = 30 CSF = 10		PFA = 30 CSF = 10	
Total binder content	590		590		590		590	
Coarse aggregate	1095		1095		1095		1095	
Fine aggregate	655		655		655		655	
Free water	118		118		118		118	
Total water	142		142		142		142	
Superplasticiser dosage(3) (%)	2.0		2.0		2.0		2.0	
Retarder dosage (%)	0.14		0.08		0.08		0.08	
Slump (mm):	155		FLOWING		190		FLOWING	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
7 days	2529	103.0	2494	77.0	2483	88.0	2463	76.5
	2507	101.5	2488	81.0	2488	89.0	2464	79.0
	2495	93.5	2494	86.0	2476	86.0	2467	78.5
	2504	105.0	2489	86.0	2474	87.0	2458	78.0
	2527	107.0	2501	82.5	2477	88.0	2485	78.0
Average:	2510	102.0	2495	82.5	2480	87.5	2465	78.0
28 days	2509	125.0	2492	113.0	2486	120.0	2468	105.0
	2492	112.0	2490	111.0	2481	112.0	2466	104.0
	2494	118.0	2499	120.0	2482	108.5	2465	106.0
	2501	123.0	2493	109.0	2502	124.5	2466	106.0
	2486	111.0	2489	115.0	2478	109.0	2465	97.0
Average:	2495	118.0	2495	113.5	2485	115.0	2465	103.5
56 days	2514	139.5	2491	120.0	2488	99.0	2462	111.0
	2505	132.0	2499	118.5	2512	123.0	2463	124.0
	2487	122.5	2499	116.0	2490	113.5	2458	117.0
	2486	113.0	2491	117.5	2514	124.0	2453	112.0
	2515	137.0	2509	114.0	2505	130.0	2458	122.0
Average:	2500	129.0	2500	117.0	2500	118.0	2460	117.0
91 days	2531	143.5	2499	132.0	2488	139.0	2463	119.0
	2507	136.0	2488	129.0	2489	121.5	2487	120.5
	2524	140.0	2504	138.0	2492	136.0	2476	126.5
	2489	131.5	2500	126.0	2491	112.0	2461	117.0
	2493	118.0	2516	129.0	2484	123.0	2460	128.5
Average:	2510	134.0	2500	131.0	2490	126.5	2470	122.5
180 days	2503	148.0	2488	144.0	2491	127.0	2460	142.0
	2509	157.5	2497	117.5	2498	131.0	2465	125.0
	2486	134.0	2494	146.0	2495	137.5	2459	127.0
	2508	141.0	2487	139.0	2498	118.0	2461	124.0
	2525	143.0	2488	120.0	2497	142.0	2470	128.0
Average:	2505	144.5	2490	133.5	2495	131.0	2465	129.0

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1
(3) Superplasticiser and retarder dosages are given as the percentage of active solids by weight of total binder.

Mixes with 20 mm maximum size of aggregate.
Series 1 : OPC mixes.

MIX No.	M1.		M2.		M3.	
Free water-cement ratio	0.38		0.32		0.26	
Cement	402		439		484	
Coarse aggregate 20 mm 10 mm	 790 425		 790 425		 790 425	
Fine aggregate	655		655		655	
Free water						
Total water	175		163		148	
Superplasticiser dosage(3) (%)	0.5		0.8		1.5	
Slump (mm):	190		195		120	
	Density kg/m³	Strength N/mm²	Density kg/m³	Strength N/mm²	Density kg/m³	Strength N/mm²
7 days	2481	55.0	2476	71.0	2540	97.5
	2475	55.0	2494	81.0	2503	82.0
	2492	57.5	2494	82.0	2527	86.5
	2474	61.0	2494	80.0	2509	92.0
	2471	54.5	2487	71.0	2528	97.0
Average:	2480	56.5	2490	77.0	2520	91.0
28 days	2476	72.0	2522	96.5	2521	100.5
	2479	80.0	2521	97.0	2524	97.0
	2500	83.0	2474	81.5	2527	106.0
	2474	72.5	2498	93.5	2547	97.5
	2484	69.0	2476	86.5	2539	114.0
Average:	2485	75.5	2500	91.0	2530	103.0
56 days	2489	84.0	2523	101.5	2510	111.0
	2498	85.0	2489	83.0	2515	102.0
	2503	84.0	2518	88.5	2518	111.0
	2482	86.0	2509	104.0	2531	102.0
	2492	82.5	2516	94.0	2519	102.0
Average:	2495	84.5	2510	94.0	2520	105.5
91 days	2483	89.0	2531	110.0	2539	102.0
	2481	78.0	2506	97.5	2538	99.0
	2489	79.0	2473	91.0	2514	112.0
	2499	84.0	2486	87.5	2540	100.0
	2499	81.0	2513	97.0	2527	106.5
Average:	2490	82.0	2500	96.5	2530	104.0
180 days	2498	82.5	2500	99.0	2523	106.0
	2488	88.5	2501	101.5	2539	119.0
	2496	90.0	2508	106.5	2528	108.0
	2512	90.0	2496	103.0	2548	109.0
	2499	80.0	2488	97.0	2555	112.0
Average:	2500	86.0	2500	101.5	2540	111.0

(1) Granite aggregate.
(2) Sand with fineness modulus of 2.1
(3) Superplasticiser dosage is given as the percentage of active solids (Conplast 430) by weight of total binder.

Mixes with 20 mm maximum size of aggregate.
Series 2 : CSF mixes.

MIX No.	M4.		M5.		M6.	
Free water-binder ratio	0.38		0.32		0.26	
Cement	362		395		436	
CSF	40		44		48	
Level of cement replacement	10		10		10	
Total binder content	402		439		484	
Coarse aggregate						
20 mm	790		790		790	
10 mm	425		425		425	
Fine aggregate	655		655		655	
Free water						
Total water	175		163		148	
Superplasticiser dosage[3] (%)	0.7		0.9		1.2	
Slump (mm):	150		165		165	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
7 days	2550	64.0	2493	77.0	2481	86.5
	2477	64.0	2500	76.0	2472	85.0
	2492	67.5	2478	69.0	2485	87.5
	2494	63.0	2500	80.0	2500	96.0
	2510	65.5	2480	76.0	2497	89.0
Average:	2505	65.0	2490	75.5	2485	90.0
28 days	2486	88.0	2480	95.0	2505	114.0
	2484	89.0	2500	99.5	2483	101.5
	2471	87.0	2474	95.0	2515	107.5
	2470	81.0	2487	96.0	2500	113.0
	2471	92.0	2484	94.0	2497	117.5
Average:	2475	87.5	2485	96.0	2500	111.0
56 days	2480	102.0	2498	105.5	2480	124.0
	2484	99.0	2513	107.0	2511	118.0
	2478	99.0	2519	105.0	2502	123.0
	2490	104.0	2517	102.0	2494	127.0
	2480	96.5	2495	107.0	2500	123.0
Average:	2480	100.0	2510	105.5	2495	123.0
91 days	2481	99.0	2507	112.0	2492	125.0
	2465	95.0	2489	105.0	2531	134.0
	2492	106.5	2514	107.5	2501	121.0
	2477	96.5	2508	107.5	2519	116.0
	2475	101.0	2476	95.0	2490	124.0
Average:	2480	99.5	2500	105.5	2505	124.0
180 days	2491	111.0	2528	113.0	2534	132.0
	2480	98.0	2488	114.0	2513	131.5
	2519	105.0	2483	107.0	2517	140.0
	2482	102.0	2511	107.0	2489	120.5
	2477	107.0	2492	107.5	2531	140.5
Average:	2490	104.5	2500	109.5	2515	133.0

(1) Granite aggregate.
(2) Sand with fineness modulus of 2.1
(3) Superplasticiser dosage is given as the percentage of active solids by weight of total binder.

Mixes with coarse sand (FM = 2.73).
Series 1 : OPC mixes.

MIX No.	N1.		N2.		N3.	
Free water-cement ratio	0.38		0.32		0.26	
Cement	451		493		543	
Coarse aggregate	1015		1015		1015	
Fine aggregate	750		750		750	
Free water						
Total water	194		180		163	
Superplasticiser dosage[3] (%)	0.4		0.6		1.3	
Slump (mm):	140		155		160	
	Density kg/m³	Strength N/mm²	Density kg/m³	Strength N/mm²	Density kg/m³	Strength N/mm²
7 days	2442	62.0	2453	73.0	2500	94.0
	2451	57.5	2458	78.0	2531	97.0
	2476	66.0	2487	83.0	2497	92.0
	2445	60.0	2452	74.0	2497	89.5
	2472	60.0	2486	77.0	2521	96.0
Average:	2455	61.0	2465	77.0	2510	93.5
28 days	2451	77.0	2476	82.0	2501	94.0
	2456	74.5	2481	87.0	2497	101.0
	2445	78.5	2487	92.5	2497	103.0
	2449	75.0	2488	95.0	2488	111.0
	2484	74.5	2459	86.5	2489	101.5
Average:	2455	76.0	2480	88.5	2490	102.0
56 days	2468	83.5	2470	84.0	2526	108.0
	2459	82.5	2488	91.0	2509	98.5
	2453	78.5	2468	82.0	2503	112.5
	2458	72.0	2493	94.5	2517	98.0
	2481	85.0	2470	90.0	2505	105.0
Average:	2465	80.5	2480	88.5	2510	104.5
91 days	2461	86.5	2468	89.0	2494	109.0
	2458	87.5	2488	99.0	2518	102.5
	2464	80.0	2475	83.0	2491	103.5
	2481	82.0	2472	95.0	2512	104.0
	2450	81.0	2461	93.0	2493	107.0
Average:	2465	83.5	2475	92.0	2500	105.0
	2472	90.0	2435	106.0	2488	117.0
	2457	98.0	2434	103.0	2473	122.0
	2438	95.0	2430	96.0	2491	118.0
	2442	92.0	2429	91.0	2505	106.0
	2458	92.0	2426	104.0	2486	118.0
Average:	2455	93.5	2430	100.0	2490	116.0

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.73.
(3) Superplasticiser dosage is given as the percentage of active solids (Conplast 430) by weight of total binder.

Mixes with coarse sand (FM = 2.73).
Series 2 : CSF mixes.

MIX No.	N4.		N5.		N6.	
Free water-binder ratio	0.38		0.32		0.26	
Cement	406		444		489	
CSF	45		49		54	
Level of cement replacement	10		10		10	
Total binder content	451		493		543	
Coarse aggregate	1015		1015		1015	
Fine aggregate	750		750		750	
Free water						
Total water	194		180		163	
Superplasticiser dosage[(3)] (%)	0.6		0.8		1.1	
Slump (mm):	155		150		160	
	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2	Density kg/m^3	Strength N/mm^2
7 days	2433	57.5	2456	76.5	2495	87.0
	2434	64.5	2434	71.0	2503	89.0
	2444	59.0	2457	73.0	2478	86.0
	2433	57.5	2452	76.5	2492	90.0
	2452	63.5	2488	81.0	2491	86.5
Average:	2440	60.5	2455	75.5	2490	87.5
28 days	2421	87.0	2444	101.0	2476	107.0
	2427	91.0	2473	105.0	2526	110.0
	2434	87.0	2453	106.0	2502	110.0
	2434	83.0	2483	105.0	2476	106.0
	2426	89.0	2440	88.0	2469	108.0
Average:	2430	87.5	2460	101.0	2490	108.0
56 days	2449	94.0	2461	116.0	2498	123.0
	2451	99.5	2465	111.0	2487	119.0
	2427	90.0	2445	101.0	2502	121.5
	2432	93.0	2463	114.0	2512	108.5
	2452	97.5	2441	106.0	2480	109.0
Average:	2440	95.0	2455	109.5	2495	116.0
91 days	2447	96.0	2486	118.0	2495	122.0
	2456	104.0	2464	111.0	2502	113.0
	2453	101.0	2434	110.0	2475	116.0
	2447	98.5	2466	115.0	2474	114.0
	2450	98.0	2429	104.5	2493	121.0
Average:	2450	99.5	2455	111.5	2490	117.0
	2468	100.0	2443	114.0	2474	107.0
	2434	120.0	2437	121.0	2499	132.0
	2437	100.0	2428	114.0	2466	120.0
	2445	105.0	2477	114.0	2478	118.0
	2406	98.0	2446	117.0	2469	121.0
Average:	2440	104.5	2445	116.0	2475	119.5

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.73.
(3) Superplasticiser dosage is given as the percentage of active solids (Conplast 430) by weight of total binder.

APPENDIX 6

DESIGN OF
HIGH STRENGTH CONCRETE MIXES
WITH NORMAL WEIGHT AGGREGATES.

SECTION 1
INTRODUCTION.

1.1 HISTORICAL BACKGROUND.

High strength concrete is often considered a relatively new material. However, its development has been gradual over many years, and as this development has continued, its definition has changed. In the 1950s, concrete with a characteristic compressive strength of 40 N/mm^2 was considered high strength in the USA. In the 1960s, concretes with 40 and 50 N/mm^2 characteristic strengths were used commercially. In the early 1970s, 60 N/mm^2 concrete was being produced. However, in recent years, engineers, particularly in North America, have been faced with increasing demands for improved efficiency and reduced concrete construction costs from developers and governmental agencies. As a result, they are now designing larger structures using higher strength concrete at higher stress levels.

1.2 DEFINITION OF HIGH STRENGTH CONCRETE.

High strength concrete refers to concrete which has a uniaxial compressive strength greater than that which is ordinarily obtained in a region, and therefore its definition varies on a geographical basis. For example, in regions where concrete with a compressive strength of 60 N/mm^2 is already being produced commercially, high strength concrete might be in the range of 80 to 100 N/mm^2 compressive strength. However, in regions where the upper limit on commercially available material is currently 35 N/mm^2 concrete, 60 N/mm^2 concrete is considered high strength concrete.

In this mix design procedure, high strength concrete is considered to be concrete with a compressive cube strength above the present existing limits in national codes, i.e. about 60 N/mm^2, and made using only readily available concrete making materials and conventional production techniques.

1.3 SCOPE OF WORK.

High strength concrete has not been achieved by chance; it is a concrete in which all the factors that contribute toward an increase in strength must be maximised and those that can lessen strength must be minimised. It may or may not require the purchase of special materials, but it definitely requires materials of highest quality and their optimum proportioning. Developing a high strength concrete has required intensive research work, the establishment of an efficient system of quality control and a good knowledge of what helps and what hinders achieving a concrete of good quality.

High strength concrete has been produced using a wide range of quality materials based on the results of trial mixtures. The state of knowledge regarding material selection has been described in Chapter 3 of this thesis. Once the materials have been selected there remains the problem of proportioning them to produce concrete having the specified properties. This becomes more of a critical process as the specified compressive strength increases. Usually, specially selected pozzolanic and chemical admixtures are employed, and the attainment of a low water-binder ratio is considered essential. The mix proportions for making high strength concrete have, usually, been selected empirically by extensive laboratory testing since there are no accepted procedures, such as the ACI and BRE methods of proportioning normal concrete mixtures.

Previous attempts to establish mix design procedures for high strength concrete have generally been based on an empirical extension of the current ACI and BRE methods that are themselves based on compilations of a large number of experimental data. This empirical approach becomes complicated as the production of high strength concrete may not only employ pozzolanas but also water-reducers or superplasticisers, the latter not covered by the present BRE method. Also, because of the variability in physical properties and availability of concrete making materials in different regions this empirical extension of present mix design methods becomes even more inaccurate. What is needed most is therefore a systematic procedure for attaining high strength concrete with readily available materials using conventional production techniques.

Results from my research programme as well as from the literature review have been used to formulate guidelines for producing high strength concrete. These are presented in a similar format as the BRE report "Design of Normal Concrete Mixes".

SECTION 2
BACKGROUND INFORMATION.

2.1 PRINCIPLES OF PROPOSED METHOD.

The proportioning of a concrete mixture is the determination of the quantities of the ingredients which, when mixed together and properly cured, will produce concrete having the desired plastic properties (workability, finishability, etc.) and the required characteristics in the hardened state (strength, durability, etc.) at the lowest possible cost. The heterogeneous nature of concrete and concrete materials creates numerous variables which influence the properties of fresh and hardened concrete. Quoting Cordon:

> "Scientifically trained men of the profession have, in the past, developed sophisticated procedures based on mathematical application of laboratory test results and physical characteristics of the concrete materials. This has resulted in a great deal of mysticism through the years. Much like the secret recipe of the master chef, the magic formula for proportioning concrete mixes has been sought by one and all."

There are many properties of concrete that can be specified, e.g., workability, strength, thermal characteristics, elastic modulus and durability requirements. The main properties considered in this mix design procedure are:

1. The workability of the fresh concrete.

2. The compressive strength at a specified age.

The mix design process must take account of those factors that have a major effect on the characteristics of the concrete, but can, at least at the first stage, ignore those which only have a minor effect on the concrete. The effects of various factors on the workability and compressive strength of concrete are described in Section 3.

The principle behind the method described in this publication is that from the restricted data usually available at the mix design stage, mix proportions are derived in an attempt to produce a concrete having the required workability and strength. Typical data are given in Section 3, but where there is more appropriate information available related to local materials, this can be used instead. A trial mix is then made, but because of the assumptions made at this stage in the design it is probable that this trial mix will not completely comply with the requirements. It may therefore be necessary to adjust the mix proportions and to use these for actual production or to prepare a revised trial mix.

High strength concrete has often been produced with high cement contents which can lead to high heat of hydration temperature rises even though the heat evolved per unit weight of cement is reduced at low water-cement ratios. The mix composition must therefore be optimised, i.e. the cement content must be kept to the minimum required and if possible be partially replaced by a low heat binder, such as PFA and GGBFS. These will affect the compressive strength developement of concrete which effect is considered in Section 3.

The elastic modulus of the concrete will be affected by the water-binder ratio and the elastic modulus of the aggregates. The latter, since it is part of the material selection process, is not described here.

The BRE report specifies durability of normal strength concretes by means of minimum cement content and/or maximum free water-cement ratio requirements. These are not expected to influence the design of high strength concrete mixes as they often have high cement contents and water-binder ratios below 0.40. In some cases, high strength concrete may require, as with normal strength concrete, the use of selected types of materials.

The durability aspect of high strength concrete that has attracted much attention is its freeze-thaw resistance. Concrete that is exposed in service to temperatures below freezing point whilst in a saturated condition may be susceptible to surface spalling, cracking and general deterioration. The types and characteristics of damage are numerous and are collectively referred to as frost damage. The extent of this type of damage is markedly increased when de-icing salts are used.

Air-entraining admixtures, which enable the concrete to withstand better the action of frost and de-icing salts, have not generally been used in North America because of the accompanying strength loss and because many types of application, such as caissons, interior columns, and shear walls, will not normally require air-entrained concrete. However, air-entrainment must be considered for concretes used in highway structures and offshore oil platforms because of their exposure. The question of whether high strength concrete needs a proper air entrained pore system to be frost resistant continues to be a relevant one with conflicting reports in the literature. The conflict concerns laboratory test results and not recorded field performance, of which there is little, (see page 76).

Air entrained concrete has not been studied in my research programme and therefore it is not described in this mix design procedure.

2.2 BASIC CONCEPTS.

2.2.1 Types of aggregates.

The higher the specified strength the more critical is the selection of the type of aggregate. The factors that have been considered when selecting coarse and fine aggregates for high strength concrete have been described on pages 92 to 109.

The data presented here were obtained using 10 mm granite as coarse aggregate and Thames Valley sand with FM of 2.1. The properties of these are shown on pages 237 to 238. The sand was finer than ideal (FM of about 3.0 has been recommended by others) but alternative supplies proved difficult to obtain.

2.2.2 Free water-cement and water-binder ratio.

The total water in a concrete mix consists of the water absorbed by the aggregate to bring it to a saturated surface-dry condition, and the free water available for the hydration of the cement and for the workability of the fresh concrete. In practice aggregates are often wet and they contain both absorbed water and free surface water so that the water added at the mixer is less than the free water required. The workability of concrete depends to a large extent on its free water content; if the same total water content were used with dry aggregates having different absorptions then the concrete would have different workabilities. Similarly the strength of concrete is better related to the free water-cement ratio since on this basis the strength of the concrete does not depend on the absorption characteristics of the aggregates.

The water-cement ratios referred to here are the ratios by weight of free water to cement in the mix and these, as well as the free water contents, are based on the aggregates being in a saturated surface-dry condition.

PFA, GGBFS, and CSF have been used as partial replacement for cement on a one to one basis by weight, and have been included as part of the binder, since they have been shown to possess cementitious properties in the presence of $Ca(OH)_2$. The term water-binder ratio has therefore been used instead of the water-cement ratio.

2.2.3 Measurement of workability.

The slump test is used to define the workability of high strength concrete. Although an attempt has been made to express the workability according to the Bingham model with tests carried out using Tattersall's two point test, this has not been used as further research is still required to establish definitive relationships between mobility and compactibility, (see page 330).

SECTION 3
THE MIX DESIGN PROCESS.

3.1 INTRODUCTION.

In order to clarify the sequence of operation, and for ease of reference, the procedure is divided into five stages. Each of these deals with a particular aspect of the design and ends with an important parameter or final unit proportions.

STAGE 1: deals with strength leading to the free water-cement or water-binder ratio.

STAGE 2: deals with workability leading to the superplasticiser dosage.

STAGE 3: deals with the measurement of the aggregate void content with different proportions of sand.

STAGE 4: deals with the investigation of the combined effect of void content and surface area of the aggregate on the amount of excess paste required to "fluidify" the concrete.

STAGE 5: deals with the determination of the sand proportion that produces the required slump with the minimum cement content.

3.2 THE FLOW PROCESS.

The sequence of operation is now described in more detail.

STAGE 1: Selection of free water-cement or water-binder ratio.

The relationship between 28-day compressive strength and free water-cement ratio obtained in this research programme for OPC mixes is shown in Figure 1, superimposed

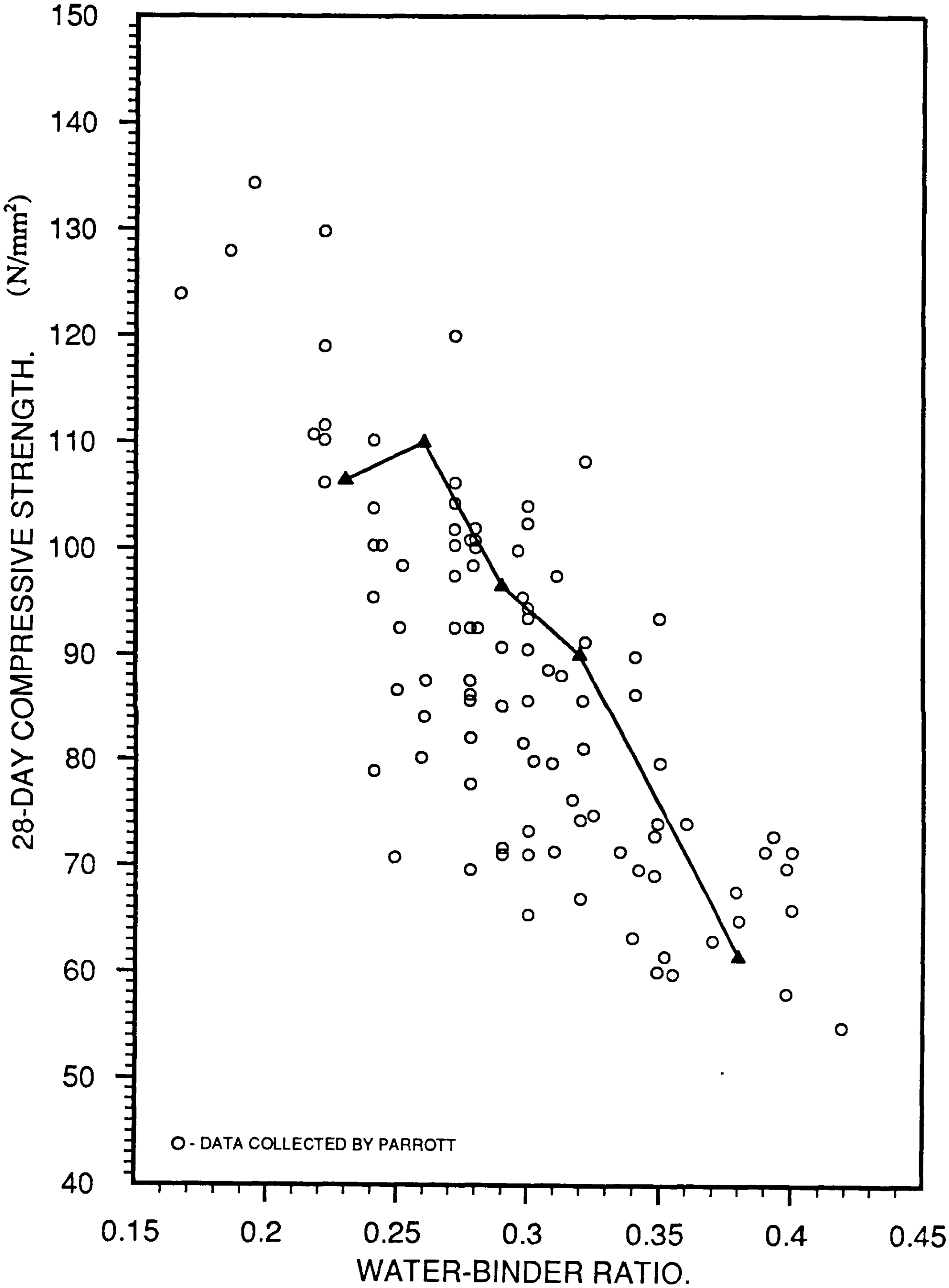


Figure 1: 28-day compressive strength of OPC mixes versus free water-cement ratio. Data collected by Parrott include a wide variety of binders, admixtures and aggregates.

on data collected by Parrott, (see page 206), for concrete with a wide variety of binders, admixtures and aggregates. The following factors must be considered before selecting the target water-cement ratio.

(I) **Strength margin.**

Because of the variability of concrete strengths the mix must be designed to have a considerably higher mean strength than the strength specified. The method of specifying concrete by its minimum strength has been replaced in British Standards and Codes of Practice such as BS 5328 and BS 8110 by a "characteristic strength". The difference between the specified characteristic strength and the target mean strength is called the "margin". This margin is based on knowledge of the variability of the concrete strength obtained from previous production data expressed as a standard deviation, or alternatively a substantial margin is applied until an adequate number of site results is obtained. Thus:

$$f_m = f_c + ks$$

where f_m = the target mean strength

f_c = the specified characteristic strength

ks = the margin, which is the product of:

s = the standard deviation

k = a constant derived from the mathematics of the normal distribution and which increases as the proportion of defectives is decreased.

Values for the margin and standard deviation have been recommended in the BRE report for normal strength concrete. Unfortunately, no recommendations can be made for high strength concrete as the data of its use in actual projects are limited. It has however been shown during big scale production and testing of high strength concrete that, by strict quality control, it is possible to obtain a coefficient of variation below 5 per cent. For example, the construction of the Gullfaks C offshore platform (240,000 m^3 with mean 100 mm cube strength of 79 N/mm^2), and two high rise buildings in Seattle in 1988 (20,000

m^3 with mean 4 x 8 in. (101.6 x 203.2 mm) cylinder strength of 96 N/mm^2) had coefficient of variations below 5 per cent.

(II) Type of binder.

The type of binder and brand of cement will influence the relationship between 28-day compressive strength and free water-cement ratio, (see page 66). Strength differences attributable to different brands of cement arise because of the variations in compound composition and fineness that are permitted by BS 12.

The use of a good quality PFA or GGBFS in the production of high strength concrete has been recommended wherever economically feasible. These materials will, in general, improve the workability, contribute to the strength development at late ages (56 to 91-days), and may reduce the temperature rise of normal strength concrete. The use of CSF has also been recommended because of the resulting improved strengths. The performance of these mineral admixtures in concrete is however affected by the water-binder ratio of the concrete. The percentage of strength, as compared to an OPC concrete, achieved with these mineral admixtures is greater in concrete mixtures which have high water-binder ratio. For example,

(i) The strength of concretes with water-binder ratios of 0.38 and 0.32 and with levels of cement replacement by PFA of 10 and 20 per cent was equal or even higher than that of OPC concrete at 56-days, (Figures 9.8 and 9.9). However, for the low water-binder ratio of 0.26, PFA concrete did not equal the compressive strength of OPC concrete even after 180-days, (Figure 9.10).

(ii) Concretes with water-binder ratios higher than 0.32 and with levels of cement replacement by GGBFS of up to 30 per cent exhibited 28-day compressive strengths equal to the OPC concrete and greater strengths beyond 28-days, (Figures 9.14 and 9.15). However, for the low water-binder ratio of 0.26, GGBFS concrete, even at the low cement replacement level by GGBFS of 10 per cent, did not equal the compressive strength of OPC concrete even after 180-days, (Figure 9.16).

(iii) The compressive strength of CSF concrete was higher than the OPC concrete for all water-binder ratios. However, the 28-day compressive strength improvement obtained by the use of 10 per cent CSF in mixes with 10 mm granite was 38.2 per cent for the water-binder ratio of 0.38 while only 3.2 per cent for the water-binder ratio of 0.26, (Figure 9.29). The strength contribution of CSF is also dependent on the bonding potential of the cement-aggregate combination used, e.g., the contribution of 10 per cent CSF to the strength of gravel mixes with water-binder ratio of 0.26 was 17.2 per cent as compared to the granite mixes which was only 3.2 per cent, (page 388 and Figure 9.7).

The high cement contents that have generally been used for the production of high strength concrete can lead to high heat of hydration temperature rises and therefore the mix composition must be optimised, i.e. the cement content must be kept to the minimum required and if possible be partially replaced by a low heat binder. This requires the development of mixtures which will give a minimum of heat generation but still have the required strength at the specified age of testing. Data from adiabatic temperature rise measurements of concrete mixes with various binder combinations, (described in Chapter 8), have shown that the high levels of cement replacement by PFA and GGBFS required for significant temperature reduction, e.g., 40 and 60 per cent, respectively, also cause a significant reduction in the compressive strength. Combinations of these with 10 per cent CSF were found to offset this; they reduced the temperature rise by more than 10^{0}C while the reduction in the 28-day compressive strength was less than 15 per cent, (Figures 9.31 to 9.34). More detailed data on the strength development of mixes with different binder combinations can be found in Chapter 9.

(III) Test age.

The selection of mix proportions can be influenced by the age at which a given strength performance is required, which varies depending upon the construction requirements. A very common test age for compressive strength of concrete has been 28-days. This has produced good results for concretes within the lower strength ranges. High strength concretes gain considerable strengths at later ages and, therefore, are often evaluated at later ages such as 56 or 90-days. This has been justified where high strength concrete has been placed in columns of high-rise buildings where full loadings may not occur until later ages. Testing at later ages has been desirable in order to take advantage of long-term strength gains so that efficient use of construction materials was achieved. Information of the strength development of mixes beyond 28-days can be found in Chapter 9.

STAGE 2: Selection of superplasticiser dosage.

The use of a water-reducer, retarding water-reducer, high range water-reducer (superplasticiser), or a combination of these becomes necessary to efficiently use all cementitious materials and to maintain the lowest practical water-cement or water-binder ratio. A sulphonated naphthalene formaldehyde based (Conplast 430) superplasticiser was used to obtain the relation between water-cement ratio and superplasticiser dosage required for a neat OPC concrete with a slump of 150 mm, shown in Figure 2. It must be borne in mind that although the slump is retained at 150 mm the yield (g) and plastic viscosity (h) values of the concrete increase as has been shown in Figure 7.10. The reason for choosing a slump value of 150 mm will be explained in Stage 4. The effect of partial cement replacement by PFA, GGBFS, CSF and a combination of CSF with GGBFS can be summarised as follows:

(i) Partial cement replacement by PFA or GGBFS causes a reduction in the amount of superplasticiser dosage required for a given slump from that required for an equivalent OPC concrete, (Figures 7.3 and 7.4). The reduction resulting from 10, 20 and 40 per cent cement replacement levels by PFA correspond to 15, 30, and 60 per cent replacement levels by GGBFS. Advantage can be taken of the improved workability to either reduce the superplasticiser dosage or reduce the water-binder ratio used in concrete and yet to maintain the same workability as the OPC concrete.

(ii) The use of CSF in concrete reduces the workability at low superplasticiser dosages and therefore requires a further addition of superplasticiser to retain the same slump. However, above a certain superplasticiser dosage, believed to be that

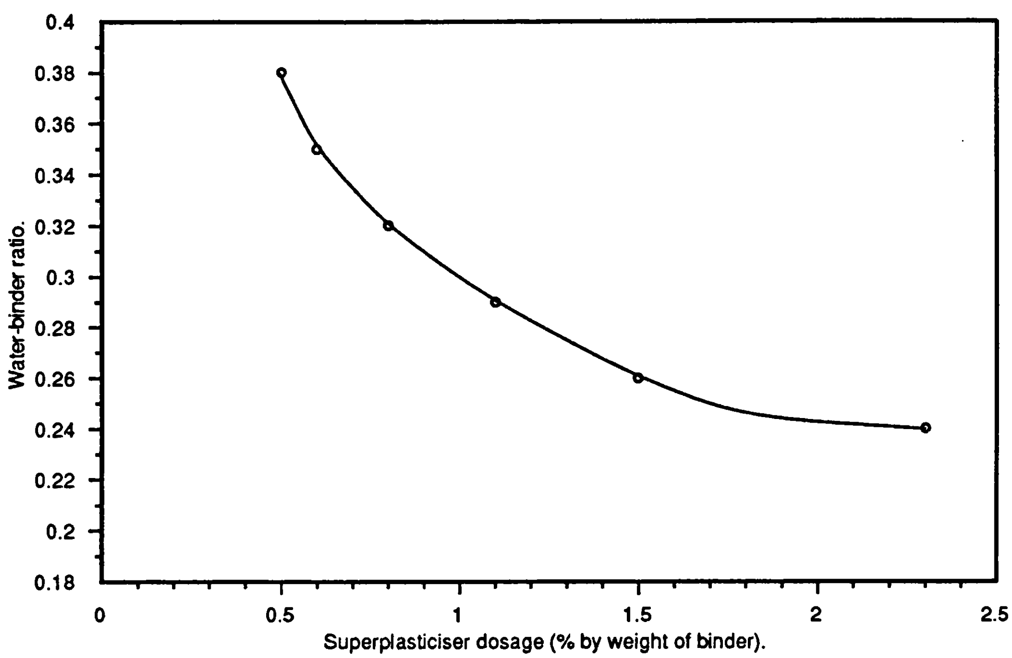


Figure 2: Water-cement ratio versus superplasticiser (Conplast 430) dosage required to produce 150 mm slump concrete.

required to reduce the cohesion of CSF particles sufficiently to disperse them, CSF has a water-reducing effect, (Figure 7.5).

(iii) The decrease of workability, as defined by the slump test, resulting from the use of CSF at low superplasticiser dosages can be counteracted by the use of GGBFS which improves the workability. At higher superplasticiser dosages, when CSF particles are well dispersed and act as fillers, CSF has a water-reducing effect even for the GGBFS mixes, (Figure 7.6). Concrete with 50 per cent GGBFS in combination with 10 per cent CSF, water-binder ratio of 0.20 and a slump of 150 mm can be achieved with 1.3 per cent superplasticiser (Conplast 430) dosage.

STAGE 3: Determination of the void content of aggregate combinations.

The strengths of aggregates are decisive for determining the ultimate load-bearing capacity of high strength concrete. For concretes with strengths of less than 35 N/mm^2, the aggregate strength is generally greater than the paste strength, and failures are therefore characterised by fractures through the transition zone between paste and aggregate. The transition zone is the hydrated cement paste in the immediate vicinity of the aggregate that has significant microstructural differences from that of the cement paste at some distance away from it. In ordinary concrete, the transition zone is typically 0.05 to 0.1 mm wide and contains relatively large pores and large crystals of hydration products. Reduced water-cement ratio improves both the mortar strength and the transition zone, and subsequently the difference in strength between the aggregate and the paste becomes an important parameter.

Ideal coarse aggregate properties seem mostly to relate to strength, aggregate-mortar bond characteristics and mixing water requirements. The use of a strong, 100 per cent crushed, clean irregular coarse aggregate, with a minimum of flat or elongated particles has been recommended. It has also been recommended that the maximum size should be kept to a minimum, at about 10 mm, since the compressive strength of concrete increases gradually as the maximum size decreases. This effect may be reduced by the use of a superplasticiser, in combination with a good quality crushed aggregate and the use of a mineral admixture such as condensed silica fume, (see Figure 9.37).

The use of a coarse sand, with an FM of about 3.0, in high strength superplasticised concrete mixes may be advantageous in reducing the water demand. The increased water demand resulting from the use of finer sands can, however, be counteracted in high strength concrete by increasing the superplasticiser dosage. The use of a coarse sand in OPC mixes with 28-day strengths of less than 100 N/mm^2 may also result in higher strengths than for a finer sand. This is believed to be due to an improvement of the paste-mortar or mortar-aggregate bond. The strength enhancement may not however be apparent in CSF mixes as the use of CSF improves the paste-mortar and mortar-aggregate bond, (see Figure 9.39). It must also be noted that above a certain strength level, (approximately 100 N/mm^2 for the coarse sand used in this research programme), this trend may be reversed, i.e. the use of a coarser sand will result in lower strengths. The failure planes of concretes with coarse sand and compressive strengths above 100 N/mm^2 passed through the coarser sand particles.

Once the aggregates have been selected, the production of high strength concrete is considered to require, firstly, a low water-binder ratio, secondly, sufficient cement paste to slightly overfill the voids, and thirdly, the use of a superlasticiser to ensure sufficient fluidity of the paste with low water-binder ratio. This stage deals with the determination of the void content of coarse and fine aggregate, combined in various proportions. The void content measuring apparatus and the test procedure have been described in Chapter 6. A typical void content graph is shown in Figure 3.

STAGE 4: Investigation of the combined effect of void content and surface area of the aggregate.

Low percentages of overfill, in combination with the aggregate proportions that give minimum void content, produce a very cohesive but low slump concrete. This is because there is a critical percentage of overfill that is required to "fluidify" the concrete. This is thought to be required in order to cover the surface of all the particles and therefore act as a lubricant. The excess volume of paste does not only depend on the surface area of

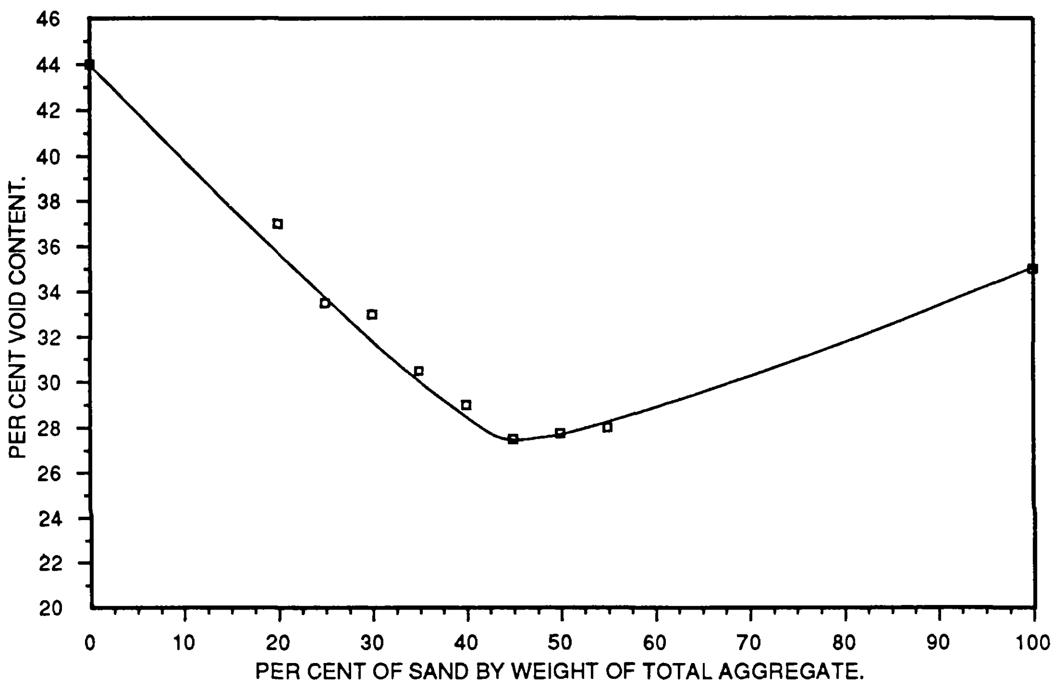


Figure 3: Void content curve for mixtures of 10 mm granite with sand of FM = 2.1.

the particles, but it also depends on its fluidity; the more fluid the paste, the less of it is required.

In order to investigate the effect of surface area on the amount of excess paste required to "fluidify" the concrete, mixes with lower sand proportions than required for minimum void content must be tested for slump. It is necessary at this stage to determine whether the superplasticiser dosage chosen in Stage 2 is sufficient to produce a workable concrete. It is recommended that this is tested with the aggregate proportions that give minimum void content and with a percentage overfill of 4 to 8 per cent. The slump should be around 50 mm or otherwise the superplasticiser dosage must be increased or decreased accordingly. A set of mixes with water-cement ratio of 0.38 are shown in Table 1. It is recommended that at least 14 mixes are tested. The total aggregate contents that were used for these tests have been re-calculated using the void content graph in Figure 3. This means that by reducing the sand proportion the granite content was increased, thus partly compensating for the increase in void content resulting from the reduction of the sand content.

Figure 4 shows that a smaller amount of excess paste is required to fluidify the concrete for lower sand proportions.

STAGE 5: Determination of the sand proportion that produces the required slump with the minimum cement content.

The dotted line in Figure 4 shows the slumps for equal cement contents at a water-cement ratio of 0.38. It can be seen that the highest slump, for a cement content of 440 kg/m^3, is obtained with a fine aggregate proportion of 37.5 per cent of the total aggregate content. This proportion, therefore, results in the minimum required cement content for a particular slump, e.g., a slump of 150 mm, for which the required cement contents with different fine aggregate proportions are shown in Figure 5. The slump was chosen to be 150 mm as at this value the gradients of the slump versus percentage overfill lines were at their highest, i.e. with only a 1 per cent increase in the overfill, the slump value was increased from about 100 to over 150 mm. Sand proportions higher than the optimum 37.5

Mix No.:	1	2	3	4	5	6	7	8
Percentage overfill (%)	3	4	5	4	5	6	5	6
Cement content (kg/m^3)	428.3	438.0	447.5	423.6	433.2	442.6	418.8	428.4
Coarse aggregate(1) (kg/m^3)	1179.6	1168.3	1157.1	1115.4	1104.8	1094.3	1095.2	1084.9
Fine aggregate(2) (kg/m^3) Proportion (%)	635.9 35	629.8 35	623.8 35	668.3 37.5	661.9 37.5	655.7 37.5	728.6 40	721.7 40
Free water (kg/m^3)	162.8	166.4	170.1	161.0	164.6	168.2	159.1	162.8
Total water (kg/m^3)	186.6	190.0	193.4	184.6	188.0	191.4	183.6	187.0
Slump (mm)	90	105	175	80	110	175	60	105

Mix No.:	9	10	11	12	13	14	15
Percentage overfill (%)	7	7.5	7	8	9	8	10
Cement content (kg/m^3)	437.8	442.4	428.4	437.7	446.9	428.4	446.7
Coarse aggregate(1) (kg/m^3)	1074.8	1069.8	1037.4	1027.8	1018.3	986.1	968.2
Fine aggregate(2) (kg/m^3) Proportion (%)	715.0 40	711.6 40	766.4 42.5	759.3 42.5	752.3 42.5	805.6 45	790.9 45
Free water (kg/m^3)	166.4	168.1	162.8	166.4	169.8	162.8	169.7
Total water (kg/m^3)	190.4	192.0	187.3	190.6	193.9	187.3	193.8
Slump (mm)	125	160	70	105	150	55	100

(1) 10 mm granite aggregate.
(2) Sand with fineness modulus of 2.1.

Table 1: Optimisation of mix proportions for mixes with a water-cement ratio of 0.38.

NOTE: The superplasticiser dosage (percentage of Conplast 430 solids by weight of cement) was 0.5 per cent for all mixes

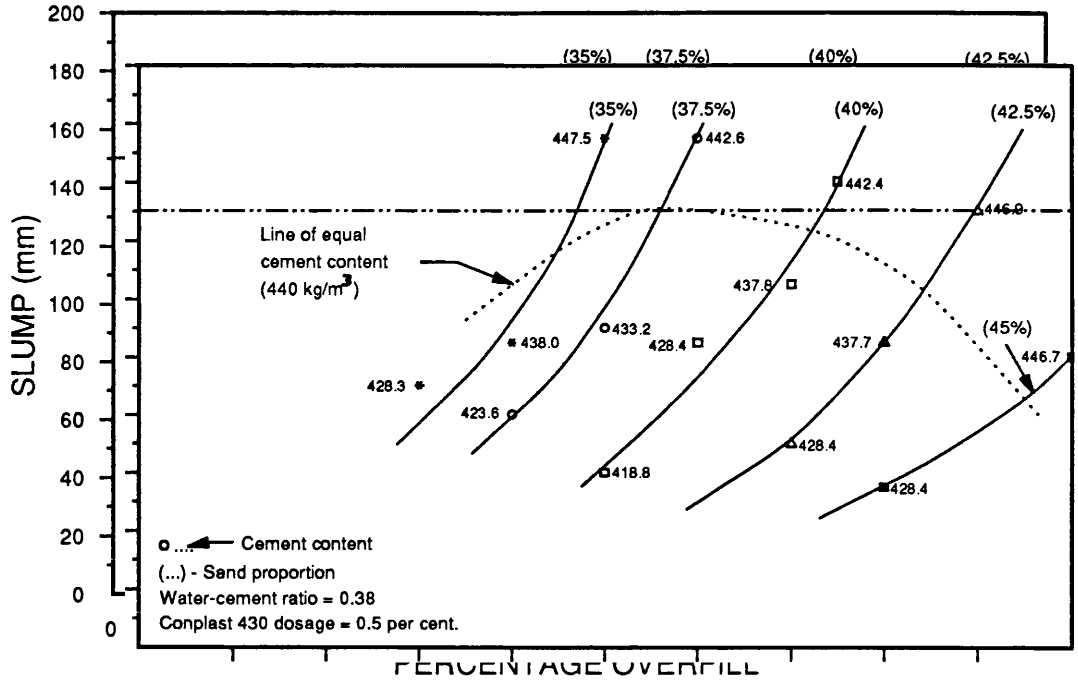


Figure 4: Slump versus overfill for mixes with 10 mm granite and various proportions of sand (FM = 2.1).

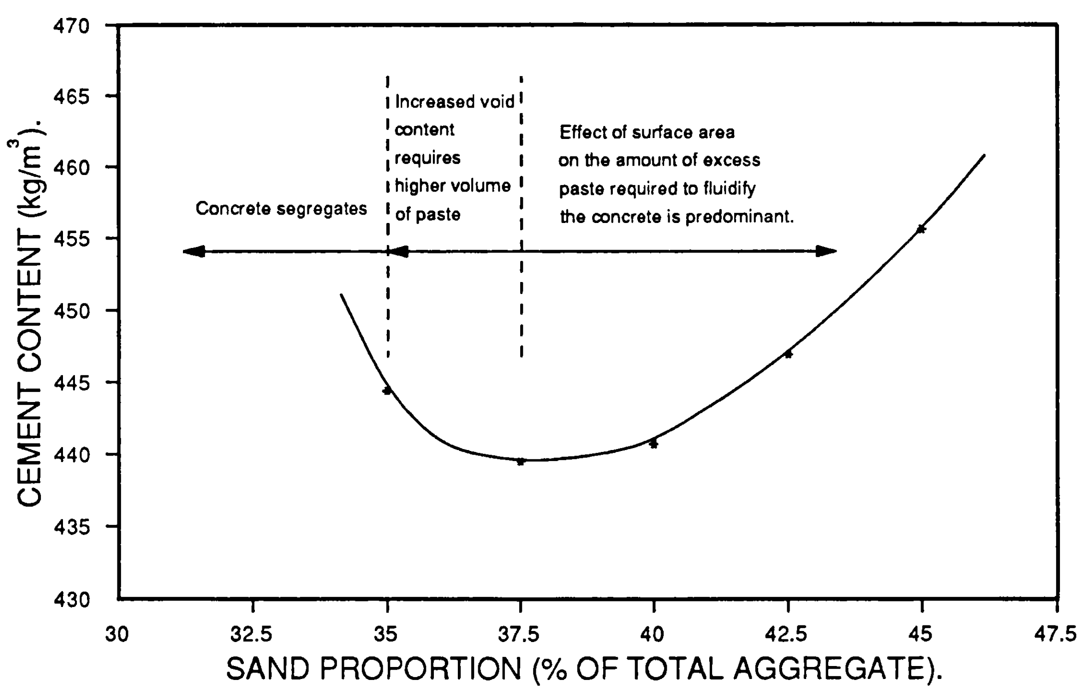


Figure 5: Cement contents required to produce a slump of 150 mm with various sand proportions.

per cent result in a higher requirement of excess paste because of the increased surface area. The increased void content of mixes with lower sand proportions than the optimum requires a larger volume of paste to fill the voids between the aggregates. Thus, despite lower values of percentage overfill required for 150 mm slump, the cement content per cubic metre of these mixes is higher than for the mix with optimum sand proportion.

Using this procedure, the optimum mix proportions of coarse and fine aggregate have been found not to be affected by a change in the water-cement ratio, (Figures 6.18, 6.20 and 6.22). Factors that may affect the optimum proportion of sand are:

(i) *Surface area of binder*. The optimum sand proportion has been found to increase from 37.5 per cent for OPC mixes to 40 per cent for mixes where 10 per cent CSF has been used to partially replace cement, (Figure 6.25). This is due to the increased surface area of the binder, for a constant amount of paste, resulting from the use of CSF.

(ii) *Fineness modulus of sand*. The use of a sand with fineness modulus of 2.73 instead of 2.1 resulted in the optimum sand proportion to be very close to the proportion required for minimum void content, e.g., with the materials used in my research programme the optimum sand proportion was 42.5 per cent, (Figure 6.30), as compared to 45 per cent for minimum void content, (Figure 6.7).

A trial mix is then made but because of the use of different materials than those used to determine the relationship between the water-cement ratio and strength, (shown in Figure 1), this trial mix may not completely comply with the strength requirements. A revised trial mix with a lower water-binder ratio can easily be made, since the optimum mix proportions of coarse and fine aggregate have been found not to be affected by a change in the water-binder ratio.