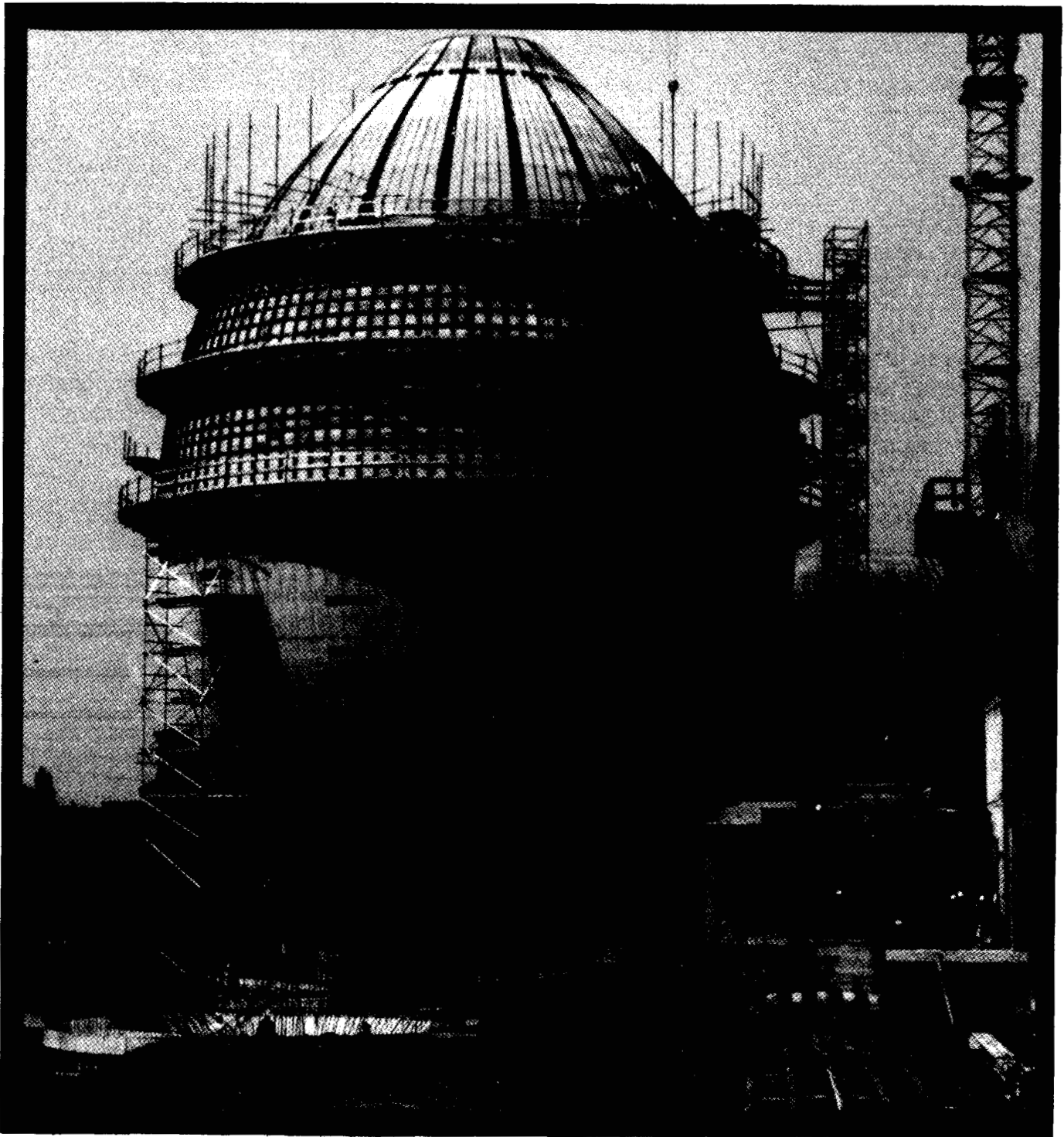


HIGH-PERFORMANCE CONCRETE



Compilation 32

American Concrete Institute

Preface

This ACI Compilation combines material previously published in CONCRETE INTERNATIONAL Magazine, ACI MATERIALS JOURNAL, and SP-149 to provide compact and ready information on a specific topic. The material in such a compilation does not necessarily represent the opinion of any ACI technical committee — only the opinions of the authors of the compiled articles. However, the material presented here is considered to contain useful information for readers interested in the subject.

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Task Group

TAC Subcommittee on High-Performance Concrete (THPC)



On the cover: The world's largest egg-shaped digesters, concrete structures that serve as central sludge handling facilities for wastewater treatment plants, are under construction in Germany. These odd-looking devices are 149 ft in height and have a maximum diameter of 92 ft.

American Concrete Institute, Box 19150, Redford Station, Detroit, Michigan 48219



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High-Performance Concrete

ACI Compilation 32

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Introduction

ACI Compilations are a series of special publications that combine material previously published by the Institute to provide compact and ready reference on specific topics. The fifteen papers in this compilation on *High-Performance Concrete* have been selected by a Task Group of the TAC Subcommittee on High-Performance Concrete (THPC) with a specific objective of providing technical information to readers who would find it useful in practical applications. For readers who may wish to pursue the subject further, a recommended reading list is also provided.

High-performance concrete (HPC) has been broadly defined by THPC as *concrete that meets special performance and uniformity requirements that cannot always be achieved routinely by using only conventional materials and normal mixing, placing, and curing practices. The requirements may involve enhancements in ease of placement, compaction without segregation, long-term mechanical properties, early-age strength, volume stability, or service life in severe environments.* Concretes possessing many of these characteristics often achieve higher strength. Therefore, high-performance concrete is often of high strength, but high-strength concrete may not necessarily be of high performance.

Two papers in this compilation are for general reading. In his paper, Forster discusses the definition of HPC and the basic considerations of when to use HPC. Factors such as workability, placing, and curing are discussed. The paper by Aïtcin and Neville points out that HPC is not that different from ordinary concrete; but, through the application of special know-how and the judicious use of cementitious materials, chemical admixtures, and optimum water-cement ratio, properties of concrete can be enhanced and applications of concrete expanded. Changes in the composition, pore structure, and bond of the matrix with the aggregate are described as the result of using this know-how in selecting materials and proportions.

To produce HPC, it is essential to have good quality coarse aggregates. Aïtcin and Mehta point out in their paper the importance of evaluating the characteristics of coarse aggregates to be used in very high-strength concrete. Test results of three Canadian and four Californian aggregates producing concrete strengths ranging from 85 to 105 MPa (12 to 15 ksi) are summarized. It is shown that coarse aggregate mineralogy, strength, and/or bonding with the cement paste play an important role in concrete strength and elastic modulus. Collins discusses proportioning high-strength concrete to control creep and shrinkage. Several different high-strength concrete mixtures having compressive strengths ranging from 56 to 65 MPa (8 to 9.3 ksi) are compared. They show satisfactory creep and drying shrinkage behavior. The use of a high-range water reducer does not have a significant effect on deformations, and the factors that reduce deformation in normal strength concrete apply also to high-strength concrete.

Economics drives the use of high-strength concrete, since its load-carrying capacity increases more than its cost. Commercial production of high-strength concrete has had a long history. Moreno documents in his paper the development of 41 MPa (6 ksi) concrete in 1962 to 96 MPa (13.7 ksi) concrete in 1982 and demonstrates the commercial capability of producing 117 MPa (16.7 ksi) concrete. To translate laboratory trial mixtures of HPC for use in field applications is not a simple task. In the paper by Schemmel et al., lessons learned from field trials with high-early-strength, rapid slump loss pavement patching mixtures designed to produce 35 MPa (5 ksi) concrete in 24 hr are discussed. Very practical advice and guidelines are given by the authors.

For HPC, large amounts of cementitious materials are usually used, thus generating more heat

of hydration and producing larger thermal gradients in newly cast large-sized HPC members. These thermal effects are of serious concern. Cook et al. in one paper and Miao et al. in another present their results of laboratory and field studies on large HPC columns. Analytical predictions are confirmed by thermocouple measurements. It is demonstrated that HPC is less susceptible to thermal cracking than normal strength concrete because of the higher tensile strength of HPC.

Two papers deal with testing of high-strength concrete. The paper by Lassard et al. examines the problems associated with testing high-strength concrete with standard size cylinders. Comparisons are made with the smaller size (100 x 200 mm [4 x 8 in.]) cylinders. The capping method is also compared with grinding and the effect of the bearing block size evaluated. The paper by Yuan et al. evaluates and compares test results for cylinders and cores from a large column with high-strength concrete. Standard cylinders were both field- and moist-cured. The effect of core location in large columns and the difference between cores and moist-cured and field-cured cylinders are determined. Temperature gradient and its effect on large columns are also evaluated.

Long-term characteristics of a very high-strength concrete are discussed by Aïtcin et al. Their paper presents an evaluation of an experimental concrete mock column made from 85 MPa (12 ksi) concrete. Cores taken from the structure 2 and 4 years after casting showed strength equal to the 28-day cylinder strength and durability of 300 coulombs in the rapid chloride permeability test. Strain gages showed no internal strains after cooling within 4 days after casting and only surface shrinkage was observed. A microstructural examination also confirmed an absence of internal microcracks. In the paper by Li et al., freezing and thawing durability of high-strength pastes, mortar, and concretes with and without silica fume is discussed. All test samples were non-air-entrained, and the air void spacings in the concretes were between 0.75 and 1.3 mm, well above the 0.2 mm normally required for durable concrete. In all cases, samples with $w/c = 0.24$ showed very good freeze-thaw durability (to 1000 cycles) despite not being air-entrained. With $w/c = 0.27$ or higher, the samples did not perform well. The test results showed that freeze-thaw performance is independent of silica fume content.

Finally, three papers are included in this compilation as illustrations of several applications of HPC. Ozyildirim describes the experience of Virginia DOT in producing very low-permeability concretes for repairs of pavement and bridge decks. He also discusses their experience in producing concretes of very high-early-strength and very low permeability for fast-track operations. Test results of both laboratory and field concrete are presented. The economic advantage of using HPC for columns of high-rise buildings and several design approaches to achieve structural ductility for such applications are discussed by Webb in his paper, which covers the continuing development of HPC in Australia. In the area of long-span bridges, Malier et al. describe the design and construction of a three-span externally post-tensioned bridge using 60 MPa (8.7 ksi) concrete. A comparison with a similar bridge designed for 35 MPa (5 ksi) concrete revealed a 30 per cent reduction in concrete volume. The bridge is instrumented for long-term monitoring of its performance.

As with any publication of the Institute, the material included in this compilation does not necessarily represent the opinion of an ACI technical committee. It reflects the opinions of the individual authors. However, the information presented here is considered to be a valuable source of reference for readers interested in the subject.

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Cores Tested Two and Four Years After Placement

Long-Term Characteristics of a Very High Strength Concrete

by P.-C. Aïtcin, S. L. Sarkar, and P. Laplante

Describes testing on cores taken two and four years after construction of an experimental high-strength concrete column. Core compressive strength equalled that of cube specimens cured in lime-saturated water at room temperature for 28 days, and after four years the concrete had retained a very dense microstructure without network microcracking. Recorded strains show very few volumetric changes in the concrete in the time since placement.

In terms of normal-strength concrete, it is generally accepted that compressive strength testing of a standard specimen is representative of the actual compressive strength of the concrete used in the structural element under study. Due to its nature and composition, high-strength concrete can display characteristics quite unlike those of its plain, normal-strength counterpart. In light of this, comparing the compressive strength of specimens and cores takes on a certain interest.

When specimens are cast, their relatively low volume, combined with the thinness of mold walls, allows significant heat loss. On the other hand, large structures, such as columns and shearwalls, provide for nearly adiabatic curing conditions at much higher temperatures. In other words, the curing temperatures of a

given field concrete and its specimens are significantly different, resulting in rather disparate curing temperature profiles.

In a previous article,¹ one of the authors proved that the higher temperature recorded in the field with very high strength concrete is in fact beneficial when the mix includes supplementary cementitious materials (pozzolans). In very low water-cement ratio concretes, the temperature rise during initial setting actually enhances the pozzolanic reaction, which otherwise would have been impeded by self-desiccation following hydration of the very fine cement particles.

One issue left unresolved in that experiment, however, relates to the element's internal temperature gradient during cooling. At this juncture, it is not clear if the gradient generates sufficient tensile stress to cause formation of a microcrack network that would substantially decrease compressive strength and imperviousness.

Experimental program

Experimental columns

To research this important aspect of high-performance concrete behavior, two sets of cores were taken in 1986 and 1988 from an experimental mock column built in Montreal in 1984 (Fig. 1) with a high-performance concrete testing at 85.4 MPa (12,300 psi) at 28 days. The com-

pressive strength was derived from 100 x 200 mm (4 x 8 in.) specimens cured in lime-saturated water at room temperature. Earlier detailed studies have been cited elsewhere.^{2,4}

Twelve thermocouples were inserted at various places in the mock column and in the active one to measure the temperature profile during the first 72 hours. Four vibrating wires were also placed in both these columns (two pairs of vertical and horizontal wires placed on the surface and inside the columns). The surface wires were attached four days after casting upon removal of the forms, at which time the center of the elements had already cooled.

Concrete composition

The main characteristics of the concrete used are:

- water/cementitious ratio: 0.24
- cement dosage (ASTM Type I): 500 kg/m³ (840 lb/yd³)
- silica fume dosage: 30 kg/m³ (50 lb/yd³)
- coarse aggregate (10 mm [$\frac{3}{8}$ in.] crushed dolomitic limestone): 1100 kg/m³ (1850 lb/yd³)
- sand (natural siliceous; fineness modulus of 3.0): 700 kg/m³ (1180 lb/yd³)

A naphthalene superplasticizer (14 L/m³ [2.83 gal/yd³]) was used in conjunction with a retarding agent (1.8 L/m³ [0.37 gal/yd³]). Since delivery took 45 minutes, part of the

Keywords: chlorides; compressive strength; cores; C-S-H; electron microscopes; ettringite; high-strength concretes; long-time study; microcracking; microstructure; performance; permeability; shrinkage; strains.

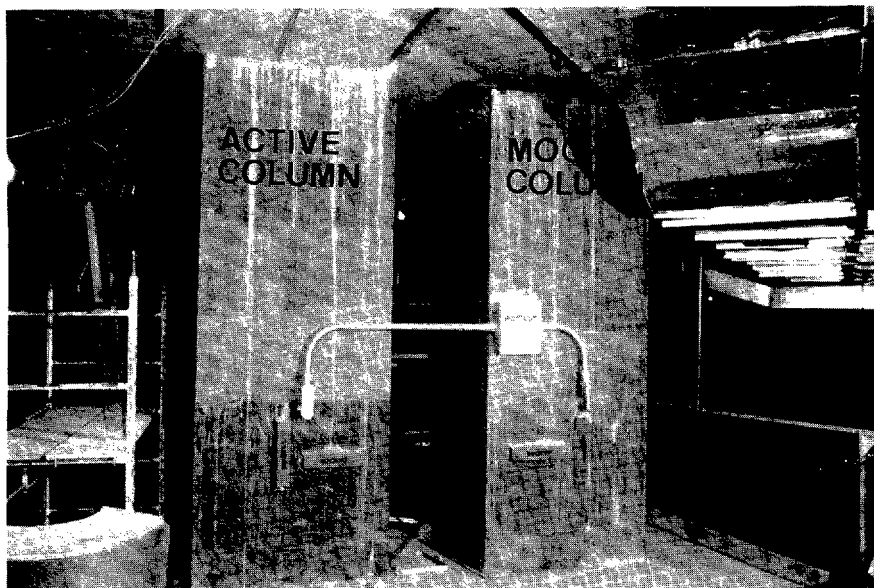


Fig. 1—La Laurentienne experimental columns.

superplasticizer was added to the load upon arrival at the jobsite to adjust the slump to 250 mm (10 in.).

The concrete was produced in a central mixer and placed with a 1.5-m³ (2-yd³) bucket in just two lifts per column.

Testing

Compressive strength was tested at 7, 28, and 91 days as well as at one year on 100 x 200 mm (4 x 8 in.) specimens cured until testing in lime-saturated water maintained at room temperature. They were removed from the lime water 24 hours prior to testing. Before testing, a very thin (3 mm [$\frac{1}{8}$ in.]) cap of high-strength testing compound was applied.

The cores taken after two and four years from the mock column measured 100 mm (approximately 4 in.) in diameter. Care was taken to insure an aspect ratio as close to 2:1 as possible when preparing the cores. Capping for testing was the same as for the specimens.

Fig. 2 gives the temperature increase recorded at three locations in the column from which the thermal gradient can be calculated during the cooling of the concrete.

As Fig. 2 depicts, the column's maximum gradient was reached just after the surface temperature peaked at 58 C (136 F). This figure further shows that the maximum

thermal gradient occurred in the outer portion of the column from where the cores were taken. In any case, it was lower than 17 C/m.

Two years after the column was cast, two cores were examined by SEM on a 2.5 x 2.5 cm (1 x 1 in.) basis. At four years, two 100 mm (4 in.) cores were evaluated for permeability with a chloride-ion permeameter. Two sections of the concrete, taken from a 2.5 cm diameter core, the first being the "skin" concrete and the other the internal concrete (1.5 cm inside), were also examined by SEM.

Length changes in the concrete are still being monitored by means of the internal and surface vibrating wires.

Results and discussion

Compressive strength

The compressive strengths of the cores and specimens reported in Table 1 show nearly identical average core strengths at two and four years: 83 and 83.9 MPa (12,040 and 12,170 psi), respectively. Individual values of the compressive strength of the cores are given in Table 2. These values correspond roughly to the 28-day compressive strength of the 100 x 200 mm specimens cured in lime-saturated water at room temperature: 85.4 MPa (12,380 psi). It should be noted that between 28 days and 1 year the specimen compressive strength increased to 97.6

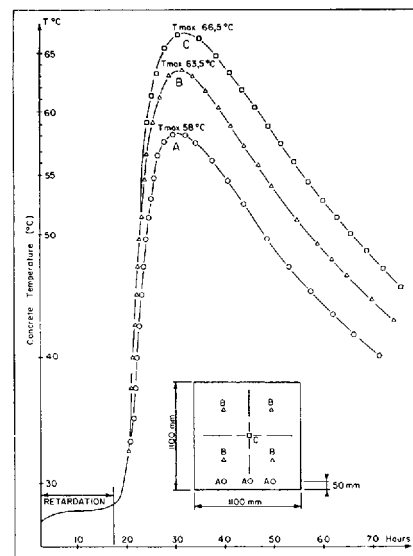


Fig. 2—Temperature rise of concrete inside column.²

MPa (14,150 psi) due to the additional curing.

Chloride-ion permeability

Two cores taken in 1988, four years after casting, were tested for chloride-ion permeability. The averaged result, 300 coulombs (Table 1), is very low, corresponding to the range of latex-modified concretes.⁵ This extremely low value suggests that very few, if any, microcracks developed as the mock column cooled and dried.

Long-term length changes

Referring to Fig. 3, the strains recorded by the vibrating wires indicate that the concrete inside the mock column experienced no substantial change in length from approximately the fourth day after casting, which coincides with the end of the cooling period. However, a shrinkage of 250 microstrains due to cement hydration and concrete cooling was recorded during the first four days following the casting of the column.

Length change measurements at the surface of the column started just after this four-day period when the forms were stripped. When comparing concrete deformations at the surface and inside the column after this period (Fig. 4), it is seen that the shrinkage of the concrete is higher at the surface than inside the column.

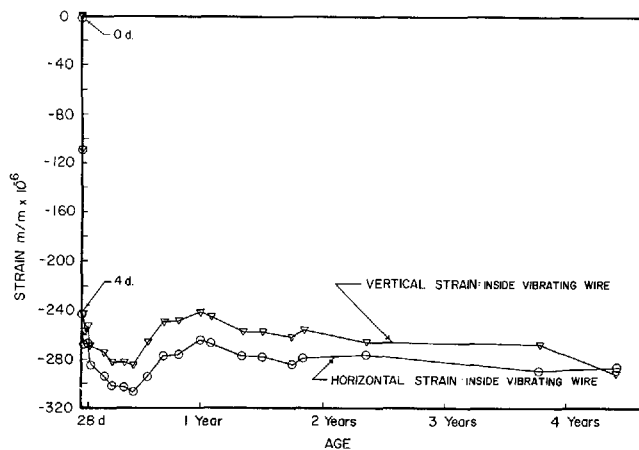


Fig. 3—Strain inside column from 0 days to four years.

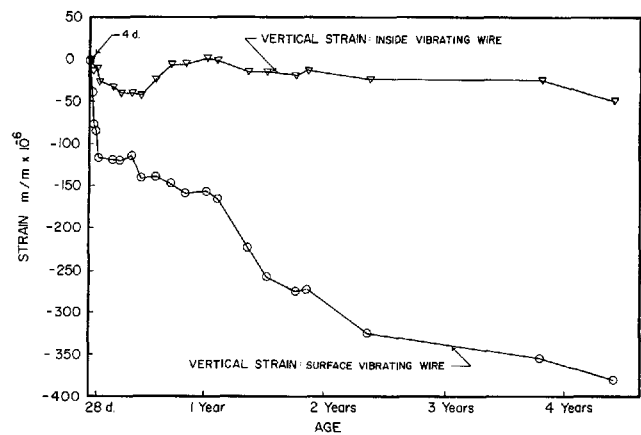


Fig. 4—Inside and surface shrinkage strain after four days.

Table 1 — Compressive strength over four years and average chloride-ion permeability at four years

Compressive strength	100 × 200 mm specimens				100 × 200 mm cores from column	
	7-day	28-day	91-day	1-year	2-year	4-year
MPa	70.8	85.4	89.7	97.6	83	83.9
psi	10,270	12,380	13,010	14,150	12,040	12,170
Chloride ion permeability, coulombs					300	

Table 2 — Individual values for 2 and 4 year cores, MPa

Sample number	2 years	4 years
1	84.9	81.9
2	81.0	87.6
3	—	82.2
Average	83.0	83.9
$\frac{f'_c \text{ core}}{f'_c \text{ 28-day}} \times 100$	97 percent	98 percent

The concrete experienced only some superficial drying shrinkage of the “skin,” which may have resulted in tensile stress concentration at the surface. However, as seen in Fig. 5, the length changes measured in the field are still much less than the one measured in the laboratory on 100 x 100 x 375 mm (4 x 4 x 15 in.) specimens. This is obvious when one considers the surface/volume ratio of the two concrete elements, the one in the laboratory specimen being much higher than the field concrete.

The changes in length recorded with the vibrating wires were made at different temperatures. Due to the fact that the thermal expansion coefficient of the steel of the vibrating wire and that of concrete are not the same, all the length changes were calculated from a reference value: 0 days for the concrete inside the column and 4 days for the concrete at the surface of the column; the thermal corrections were then made using

$$\frac{\Delta \ell}{\ell} = 625 \times 10^{-5} \times K (N^2 - N_o^2) + \Delta \theta (\alpha - \beta)$$

where

- $\frac{\Delta \ell}{\ell}$ = the length change expressed in microstrains
- N_o = the initial vibration frequency of the wire
- N = the measured vibration frequency at time t
- K = the extensometer coefficient
- $\Delta \theta$ = the temperature change in deg C from the initial temperature
- α = the thermal expansion coefficient of steel (11.5×10^{-6})
- β = the thermal expansion coefficient of concrete (9×10^{-6})

Microstructural examination

The microstructure of this concrete, which was first examined in 1986 two years after construction of the column,⁴ was once again examined in 1988 to find out if any changes had occurred in the microstructure or if any deterioration had taken place since the last examination.

Skin concrete — The hardened paste was still found to exhibit a thin (1-2 mm) porous layer (Fig. 6), which resulted during the placing of the concrete. The placing, done by internal vibration, tends to create an

external layer with a higher water/cementitious ratio, very much like a normal water-cement ratio concrete.

Only a few well crystallized CH crystals, though not as large as in a normal concrete (Fig. 7), could be located in this zone. A few fissures ($\leq 1 \mu\text{m}$ in width) are also present (Fig. 8). Otherwise the concrete is composed mainly of the C-S-H gel (Fig. 8) characteristic of very high strength concrete.⁶

The honeycomb structure of C-S-H seen in Fig. 8 and 9 probably represents the original site of a cement grain dislodged during fracturing of the specimen. It indicates the presence of water-cement ratio gradient in the concrete, the ratio being higher at the paste-aggregate interface, which has given rise to a different type of C-S-H in this region.

Partly hydrated to unhydrated grains of cement (Fig. 10) are still a common feature. This is attributable to the low water-cement ratio used to make this concrete, which left several grains unhydrated due to lack of sufficient water.⁷

Internal concrete — Its microstructure is very similar to that of the

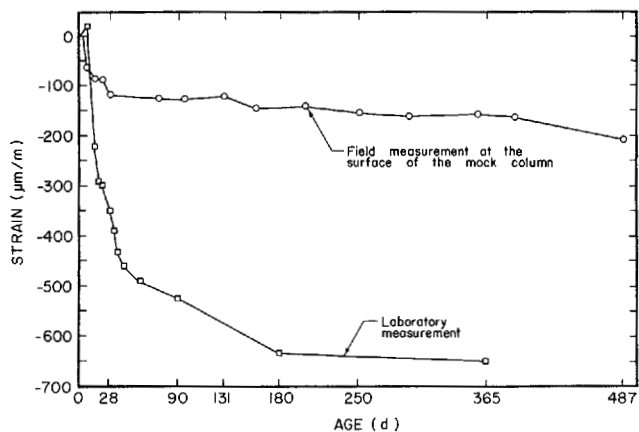


Fig. 5—Comparison of field and lab drying shrinkage.

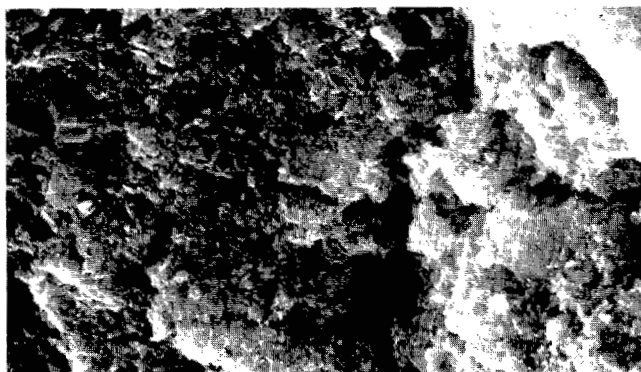


Fig. 6—Skin concrete showing the porous layer.

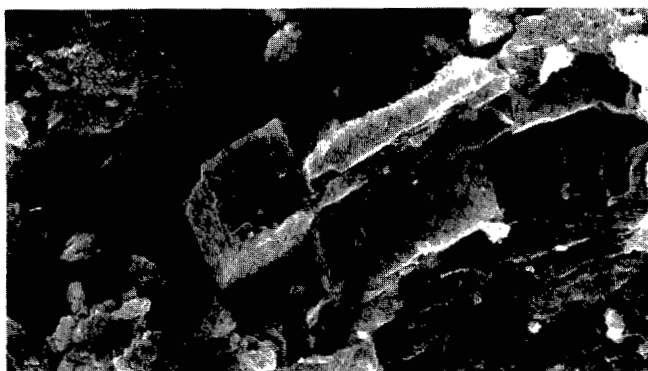


Fig. 7—Small, well crystallized CH crystals associated with C-S-H gel in the skin concrete.

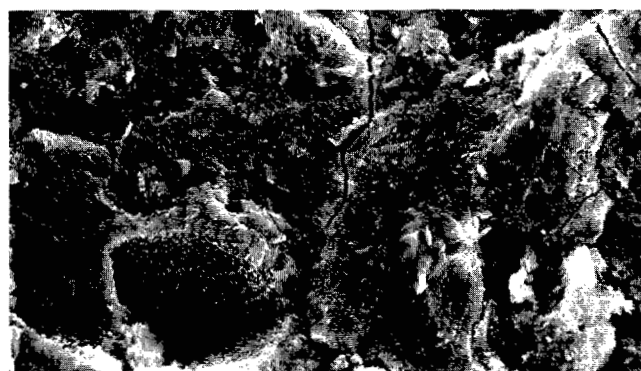


Fig. 8—Skin concrete showing dense microstructure (composed of C-S-H) with the exception of a few fissures.

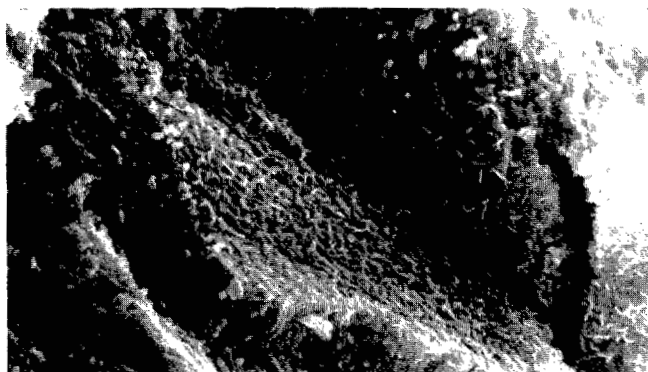


Fig. 9—Honeycomb C-S-H structure in the skin concrete and inside.

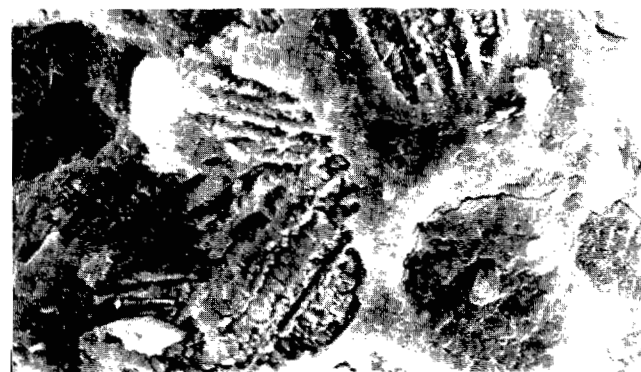


Fig. 10—A partly hydrated cement grain in which the lamellas of belite can still be distinguished.

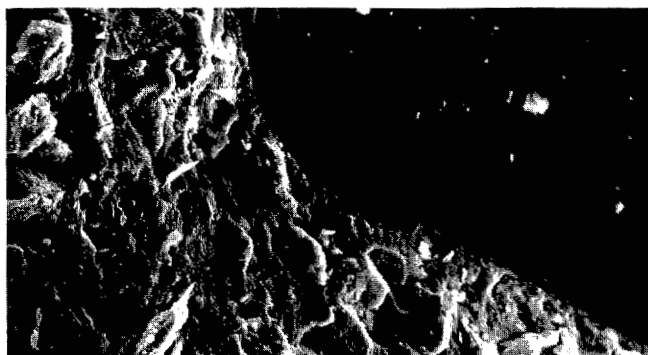


Fig. 11—Microstructure of internal concrete showing dense C-S-H and strong paste-aggregate bonding.

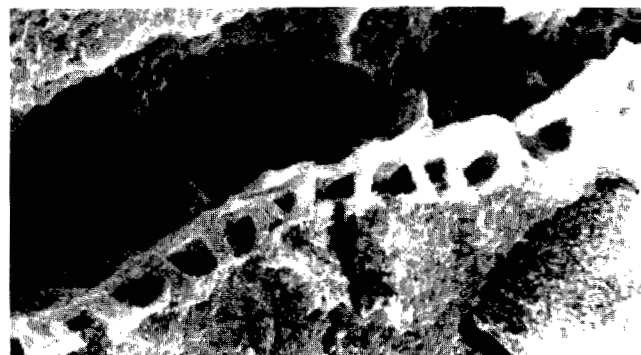


Fig. 12—Spinal cord structure seen in internal concrete.

ternal part, except that the matrix is much more compact (Fig. 11), as is expected in very high strength concrete made with silica fume and superplasticizer.⁸ The most noticeable feature is the strong and rigid bonding between paste and aggregate, which still holds firm after four years of construction (Fig. 11).

No ettringite crystals were detected, and CH crystals in this part of the concrete are also very rare.

The partly hydrated lamellar belite grains (Fig. 10), which are common, can be readily distinguished from C-S-H and other particles from their lamellar structure. A comparison of the degree of corrosion of these lamellas in the present concrete with the one examined in 1986 strongly suggests that no further hydration had occurred.

The honeycomb C-S-H structure seen in the external part is also present in the internal concrete. An interesting, though rare feature ob-

served is the spinal-cord structure (Fig. 12), which from EDX probe analysis gives a strong Ca signal. It is most likely the bidirectional orientation of CH, formed due to localized concentration of superplasticizer. No definite explanation, however, can be presented at this stage regarding its formation.

Conclusions

The results show that the 28-day compressive strength (83 MPa) of 100 mm cores taken from a mock column at two and four years after casting are nearly identical to that of specimens (85 MPa) cured for 28 days in lime-saturated water at room temperature. There is no evidence of strain inside the column, and surface shrinkage is also lower than in laboratory concrete.

From the microstructural study of the concrete it is obvious that the paste-aggregate bonding in this concrete is still very strong, both in the skin layer and in the interior. It still retains its dense microstructure, especially in the internal part. No detrimental microstructural features have developed during the first four years that could affect the strength of this concrete. Finally, it still can be classified as a very durable concrete due to its low chloride-ion permeability.

Acknowledgments

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Selected for reader interest by the editors.

ACI Fellow P.-C.

Aïtcin, a professor of civil engineering at the Université de Sherbrooke, Quebec, Canada, obtained his PhD degree from the Université de Toulouse, France. His main areas of interest in concrete technology include high-strength concrete, using industrial byproducts in concrete, and concreting practices in the arctic. He is a member of ACI Committees 209, Creep and Shrinkage in Concrete; 234, Silica Fume in Concrete; and 363, High Strength Concrete.



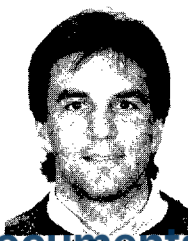
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High-Strength Concrete: Economics, Design and Ductility

by John Webb

High-strength concrete is a construction material that has been used in increasing volumes in recent years. It is popular for two obvious reasons: it is economical purely on the basis of cost to resist a specified load, and it allows the use of smaller columns.

The American code, ACI 318-89, does not have specific upper limits on concrete strength. The current Australian Concrete Structures Code, AS3600, recognizes only the use of concrete up to 50 MPa (7500 psi) in strength, although a modification allowing the use of 60 MPa (8600 psi) concrete, which has been commonly used in Australia for a number of years, is expected in the near future. More recently, concrete of 70, 80, and even 90 MPa (10,000, 11,500, and 13,000 psi) have been used on a small number of projects in Australia. In other countries, concrete up to 130 MPa (19,000 psi) has been used.

Concerns have been raised that design methods and safety factors cannot be extrapolated for concretes of this strength, principally because of the reduced ductility of the higher strength concrete. This article addresses some of these concerns and approaches that have been or could be followed.

High-strength concrete (60 MPa and above) has been used for various reasons,¹ including early stripping, greater stiffness, durability, and water retention, but this article is limited to the major reason of higher axial strength. And, although axially loaded elements include both columns and walls, only columns are considered here for simplicity.

The same economic advantages apply with high-strength concrete in walls as in columns, with the added advantage that a saving in wall area generally improves net rentable area.

This is not the case with columns in Australia.²

Economics: basic parameters

High-strength concrete is the most economical way to resist axial load, and methods that take advantage of this fact are the most economical column construction methods. Traditionally, this has meant conventional reinforced concrete columns, but other systems also have been developed over the last 20 years that utilize the material in composite column designs.

The approximate relative costs of 30 and 60 MPa (4300 and 8600 psi) concrete and Grade 350 steel (50 ksi) to resist the same load, as shown in Table 1, indicate high-strength concrete's economic advantage. These ratios don't give an absolute feel for the cost of the finished product, but two things are obvious: an all-steel column can't compete with a column that gets most of its strength from 60 MPa (8600 psi) concrete, and 30 MPa (4500 psi) concrete is never going to compete with 60 MPa for columns, as other costs for the 30 MPa (4300 psi) column are going to be higher than those for the 60 MPa column.

The relative cost of 100 MPa (14,500 psi) concrete to resist a given load extrapolated using current concrete costings could be as low as 0.75. This obviously means that developing the ability to produce this concrete commercially will be an advantage; thus, establishing design methods using the material must be a priority.

However, concrete or steel alone do not make a finished column in a building. Concrete needs reinforcement, formwork, and cladding and painting, and steel needs fire protection, cladding, and painting. Combining these elements in the most economical way leads to economical columns.

Several exercises have been performed on the economics of various column alternatives.* These have consistently shown that high-strength reinforced concrete columns are the cheapest columns, and that other options such as composite columns (small steel shafts in a reinforced concrete column), encased struts (a steel column with concrete encasement), and traditional all-steel columns all are more expensive, in general the cost increases with increasing amounts of steel in the column. Some of these comparisons are given later.

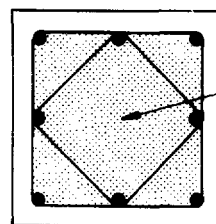
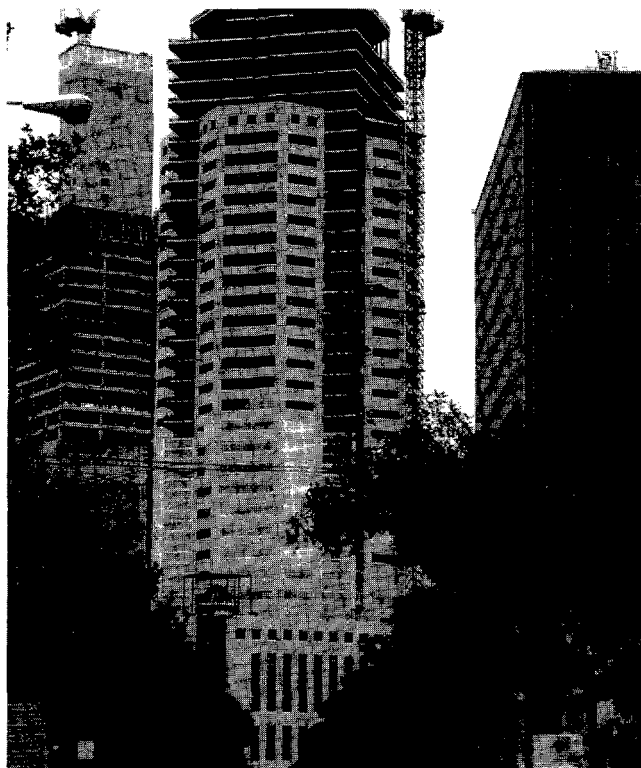
The most recent studies have shown that the system currently being used on the 43-level Casselden Place project (Fig. 1) in Melbourne, Australia — a fire-protected steel tube filled with plain concrete — can be built for around the same cost as a reinforced concrete column. This is perceived to be a technological breakthrough, as it allows a steel-framed building to be constructed without the cost penalties associated with traditional steel columns, while still providing all the constructability advantages.³

This type of column is also the ideal vehicle for very high-strength concrete, and possibly would be cheaper than reinforced high-strength concrete columns because of the design advantages of this solution. These are discussed further later.

Design

As mentioned earlier, some concerns have been expressed about extrapolating design methods for normal strength concrete to high-strength concrete. There are three principle areas of concern: ductility, consistency of the strength of the material in place, and

*Research done at Connell Wigney (Vic) Pty. Ltd., South Melbourne, Australia.



SHADED AREA IS CORE OF COLUMN WHICH RETAINS STRENGTH AFTER OUTER CONCRETE IS SPALLED OFF

Fig. 2 — Column showing core section.

Table 1 — Relative costs for different materials to resist the same load

30 MPa concrete :	60 MPa concrete :	Grade 350 steel
1.5 :	1 :	2

Fig. 1 — (left) Casselden Place, Melbourne, Australia, under construction.

fire safety. The first area is discussed here in some detail, and the others peripherally.

Ductility

Plain concrete is not a particularly ductile material. Different reinforced and prestressed concrete elements have varying degrees of ductility; this is recognised in a variety of ways by codes, including using different strength-reduction factors. For example, in AS3600, the strength-reduction factor for beams is 0.8, as compared to that of 0.6 for the less ductile, axially loaded columns. In ACI 318-89, the same effect is reflected in the strength-reduction factors, with the factor for beams being 0.90 and 0.70 for columns.

Plain high-strength concrete is less ductile than lower strength concrete, although the relative ductilities of the two materials when reinforced is less well understood.

The approach that can be used to improve the ductility of concrete columns substantially is the use of additional tie reinforcement. This approach already is used in designing columns in buildings required to be designed for earthquake loads.⁴

The current ACI and AS3600 tie requirements address only the problem of buckling of longitudinal reinforcement and can be shown to have a small

effect in confining the column's concrete core. A much more ductile column can be provided by increasing the volume of ties in the column. If sufficient ties are used, substantial residual strength will remain after the outside shell of the concrete has completely spalled off.

Therefore, it seems logical that an increased strength-reduction factor would be appropriate if additional ties were provided to a normal strength column. This logic can be followed through for high-strength columns, attempting to provide comparable ductility for the same factor, or varying both to provide suitable capacity-reduction factors for the column ductility.

Code authors have a number of options to provide a suitable design method for high-strength columns:

1. Determining an appropriate strength-reduction factor for use with high-strength concrete. This may need to vary with strength, and be less than the current factors, e.g., 0.6 in AS3600.

2. Determining the amount of ties that would be required to provide comparable ductility to that provided by the existing tie requirements for 50 MPa (7500 psi) concrete.

3. Allowing designers to design column ties to provide increased ductility and use increased strength-reduction factors because of this ductility improvement.

Obviously Options 1 and 2 are more desirable to maintain a consistent approach within the code.

The key questions here are how to measure ductility and how to calibrate an appropriate capacity-reduction factor to this measure of ductility.

Crawley⁴ has suggested that the capacity of the confined core (Fig. 2) after the spalling of the outer concrete should be 0.9 times the ultimate capacity. He calculated the capacity of the core using the method proposed by Mander, Priestley, and Park.⁵

Although spalling of the outer layer is more likely in high-strength concrete, the proposed requirement is considered excessive, as it demands a far higher residual strength than would be the case for columns designed to the requirements of current codes. Consider a 600 x 600 mm (24 x 24 in.) column with 1 percent reinforcement and 2 closed ties, as shown in Fig. 3. The minimum tie requirement for this column from AS3600 would be R10 ties at 360 mm (3/8 in. ties at 14 in.) centers, which would have a residual core capacity of 0.73 times its maximum strength. The core of the same column in 120 MPa (17,300 psi) concrete would have 0.70 times its maximum strength.

The ties are minimal and have a small confining effect, slightly enhancing the strength of the core of the 30 MPa (4300 psi) column. They have

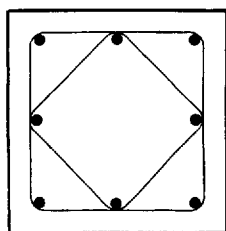


Table 2 — Column tie requirements for a 600 x 600 mm column, 8Y24

	Concrete compressive strength		
	30 MPa (4300 psi)	60 MPa (8600 psi)	120 MPa (17,300 psi)
ACI 318-83 tie requirement	R10 — 300 3/8 in. at 12 in.	R10 — 300 3/8 in. at 12 in.	R10 — 300 3/8 in. at 12 in.
AS 3600 tie requirement	R10 — 360 3/8 in. at 14 in.	R10 — 360* 3/8 in. at 14 in.	R10 — 360* 3/8 in. at 14 in.
Confined section capacity/ full section capacity†	0.73	0.72	0.70
Tie requirement to provide 0.9 capacity ratio† (as recommended by Crawley)	Y16 — 200 5/8 in. at 8 in.	Y24 — 200 1 in. at 8 in.	Y32 — 200 1 1/4 in. at 8 in.
Tie requirement to provide 0.8 capacity ratio†	Y12 — 200 1/2 in. at 8 in.	Y16 — 200 5/8 in. at 8 in.	Y24 — 200 1 in. at 8 in.

*Concrete strength is in excess of that allowed by code for these tie layouts.

†Calculated using method of Mander, Priestley, and Park,⁵ as recommended by Crawley.

virtually no enhancing effect on the core of the 120 MPa column. The effect of providing heavier ties is to enhance the strength of the core. To achieve enhancement to a capacity ratio of 0.9, the ties in the 30 MPa (4300 psi) column must be increased to Y16 at 200 mm (5/8 in. at 8 in., or about 8 times as much reinforcement), while in the 120 MPa column they must be increased to Y32 at 200 mm (1 1/4 in. at 8 in.) to achieve the same effect. This reinforcement requirement seems severe, especially given that current column designs would not achieve the required measure of ductility. Ties to achieve a capacity ratio of 0.8 also are shown. The ties found using Crawley's method are almost identical to those required by ACI 318-89⁶ for earthquake-resistant construction.

The need to provide additional ties in high-strength columns is probable, though not clear cut. The level of ductility implicit with the current tie requirements and strength-reduction factors also is not clear to the user of current codes of practice.

There is an obvious lack of compatibility between the numbers resulting from Crawley's method and current practice in lower strength concretes. This suggests a vacuum exists in the information available on the ductility of columns, particularly those of high-strength. No doubt, this is partially due to the difficulty of testing re-

inforced columns of reasonable size, particularly in recording their unloading behavior. The most appropriate measure of ductility for columns needs to be determined, and the level of ductility provided by columns currently used in practice needs to be thoroughly understood.

Some ductility criteria for composite beams related to the amount of deformation before failure have been developed by Patrick and Bridge.⁷ The deformation is such as to provide a certain amount of load redistribution. Perhaps a more suitable criterion could be evaluated on this basis. Having established the criteria, the amount of ties required to provide similar levels of ductility in higher strength columns can be established.

Undoubtedly, considerable testing is required, and careful evaluation of the answers is needed. Design practitioners must participate in this evaluation if the most suitable answer is to be achieved. The potential economic advantage means that additional costs (such as increased ties) or reduced gains (by reduced strength-reduction factors) can be absorbed and still produce a substantial cost advantage. If ties of the order of Y32-200 (1 1/4 in. at 8 in.) are necessary to allow the use of the example 120 MPa (17,300 psi) column with the same capacity-reduction factors, the likely best economic answer may not be this type of tie, but

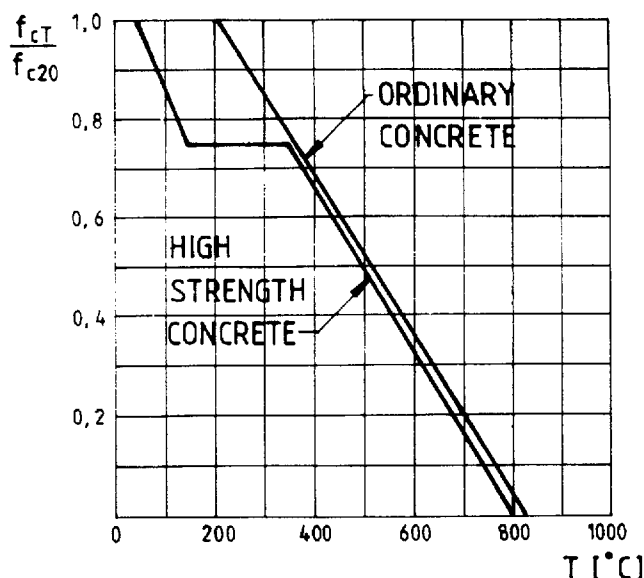


Fig. 3 — Effect of temperature on the compressive strength of concrete.

rather using smaller strength-reduction factors.

Obviously, more thinking, research, and testing is necessary to determine the correct answer, but the likely solution will probably involve:

- limits on longitudinal reinforcement ratios;
- limits on spacing of longitudinal bars and ties;
- detailing requirements at joints;
- some calculated volume of tie reinforcement that depends on the concrete strength; and possibly
- changes in strength-reduction factors.

Fire engineering

Apparently, spalling of the concrete occurs more easily in higher strength concretes because of their lower water contents. This problem has been researched in various countries. For example, according to Inha et al.,⁸ the Finnish Code shows the degradation of high-strength concrete to drop off at a lower temperature than that of conventional concrete, as shown in Fig. 4. More research is needed to verify this performance for high-strength concrete.

Ideal environment for high-strength concrete?

A fire-protected steel tube filled with plain concrete moves in perfect way through high-strength concrete.



Fig. 4 — Concrete being placed into tubes at Casselden Place. A pump hose is connected to the base of the column and pumped up as far as six stories at a time. No vibration of the concrete or scabbling of the interface between placements is required.

The confining action of the tube on the concrete provides the column with ductility far in excess of the requirement suggested by Crawley,⁴ as well as boosting concrete strength.⁹ The fire protection is an added advantage.

These columns also provide a better environment for very high-strength concrete because:

- Excellent compaction is possible using pumping techniques, as used recently at Casselden Place,
- perfect curing conditions exist inside the tube, and
- minimal creep and shrinkage occur in the effectively sealed conditions, causing minimal moisture loss.

Eurocode 4⁹ recognizes the improved quality of concrete in a filled tubular section by removing the 0.85 factor on the concrete strength in the rectangular stress block.¹⁰

This form of column is seen as a technological breakthrough because it provides the ability to construct a steel building without the cost penalties associated with traditional steel columns. It is a natural cutting-edge method for

using very high strength concrete. Research is now under way using these columns with 100 to 120 MPa (14,500 to 17,300 psi) concrete.*

Placing concrete for these columns using a pumping technique has been pioneered on the Casselden Place project. The concrete is pumped into the steel tubes through a nozzle connection (Fig. 5). In any one lift, concrete is placed from the bottom and pumped up as many as six stories at a time. The placement method, which eliminates the need to vibrate the concrete, was validated by a full-scale prototype test.

The tube is erected much like a traditional steel column, with floor reinforcing, concreting, etc. occurring in an identical fashion. The tubes are erected in either two- or three-story lengths and connected temporarily by turnbuckles. These allow the crane to get away quickly from erecting a column, and also facilitates plumbing and alignment. Plumbing and alignment are easy using this method and excellent tolerances have generally been achieved at Casselden Place. The splice (Fig. 6) is completed using butt-welding, and the turnbuckles are then removed.

Concrete with strength up to 70 MPa (10,000 psi) is being used on the project. The superplasticised concrete mix contains silica fume, principally to eliminate bleeding, thereby providing consistency over the height of the placement and eliminating the need to scabble the interface between placements. The method is extremely efficient, and large numbers of columns can be filled at a time by a small workforce and with minimal material waste.

The bare steel tube is capable of supporting six floors of construction, giving the contractor many options for time of concrete placement, which can be done whenever concrete pumphines and two workers are available.

Cost of column types

The costs of different column types are compared in Fig. 7. Three different columns using 120 MPa (17,300 psi) concrete are included, two conventional reinforced concrete columns and one concrete-filled steel tube. The first 120 MPa concrete reinforced column option uses Crawley's recommendations⁴ for ties and a strength-reduction factor of 0.6 (as required by AS 3600). In the second option, the strength-reduction factor is notionally reduced to

*Research done at Connell Wagner (Vic). Pty. Ltd., South Melbourne, Australia.



Fig. 5 — Splice between two tubular sections. Columns are supported temporarily on turnbuckles, which facilitate plumbing and alignment. The splice is completed by butt welding.

NO.		COLUMN	RELATIVE COST
1.		60 MPa REINFORCED CONCRETE [8600 psi] ($\phi = 0.6$) 840 X 840 6Y24 [33" sq 6-1" bars] AS3600 TIES (R10 - 360)	1.0
2.		120 MPa REINFORCED CONCRETE [17300 psi] ($\phi = 0.6$) 600 X 600 8Y24 [24" sq 8-1" bars] CRAWLEY (1989) TIES (Y32-200)	0.79
3.		120 MPa REINFORCED CONCRETE [17300 psi] ($\phi = 0.5$) 660 X 660 8Y24 [26" sq 8-1" bars] AS3600 TIES (R10 - 360)	0.77
4.		60 MPa [8600 psi] CONCRETE IN GRADE 250 STEEL TUBE ($\phi_c = 0.6$, $\phi_s = 0.85$) 740 X 10 CHS [29" dia X $\frac{3}{8}$ " thick]	0.98
5.		120 MPa [17300 psi] CONCRETE IN GRADE 250 STEEL TUBE ($\phi_c = 0.6$, $\phi_s = 0.85$) 570 X 8 CHS [23" dia X $\frac{5}{16}$ " thick]	0.71
6.		STEEL COLUMN GRADE 350 ($\phi = 0.9$) 600 X 40 FLANGES [24" X 1 1/2"] 520 X 40 WEB [21" X 1 1/2"]	2.21

RELATIVE COST OF DIFFERENT COLUMN TYPES TO RESIST THE SAME LOAD

Fig. 6 — Relative costs of different column types to resist the same load.

0.5, with minimal ties provided as per the current code. The validity of the assesment is dependent on the validity of the costing of the high-strength concrete.

The following conclusions can be drawn readily from the data show in Fig. 7:

- Steel columns are far more expensive than any other column type.
- The tube-column can be built for roughly the same cost as the conventional reinforced concrete column.
- The 120 MPa (17,300 psi) columns provide a 20 to 25 percent saving even with either the heavy ties or the substantial drop in strength-reduction factor to 0.5.
- The additional ties are a substantial cost penalty and make a column with 21 percent more area slightly cheaper. In fact, they represent around 18 percent of the finished cost of Column 2, compared with less than 1 percent of Column 3.
- The tube-column becomes relatively more competitive as the concrete strength increases.

The messages here are that very high-strength concrete can provide economies even with cost penalties not associated with conventional strength

columns, and that the tube-column is an ideal first step for using this material.

One further note is that using high-strength concrete means that very little real gain can be made trying to reduce column size by adding reinforcement. This can be seen in Fig. 8, which shows that an 800 x 800 mm, 80 MPa (32 x 32 in., 11,500 psi) column with 1 percent steel is as strong as a 750 x 750 mm, 80 MPa (30 x 30 in., 11,500 psi) column with 4 percent steel. However, the latter column costs 35 percent more. Therefore, using reinforcement rates higher than about 1 percent steel in very high-strength columns is difficult to justify unless there is improved performance (e.g., ductility).

The tube-column offers the potential for reducing column sizes more cheaply. Compare the graph in Fig. 9 to the line for 60 MPa concrete in Fig. 8. To reduce the size of the concrete column to 700 mm (28 in.), 6 percent steel must be used at a cost penalty of 50 percent, but the tube-column can be reduced to 700 mm diameter by increasing the plate thickness to 16 mm ($\frac{5}{8}$ in.), incurring a cost penalty of only 20 percent. This is due to a number of reasons; in particular, adding

further steel not only gives additional squash strength, but enhances the confining effect on the concrete.

Conclusions

High-strength concrete offers substantial benefits over lower strength concretes and over structural steel as an axial load bearing construction material. As well as reducing the cross-sectional area of columns, it provides cheaper columns.

Concrete of 60 MPa (8600 psi) has been used extensively, but higher strengths of concrete will be used increasingly. A coordinated research program to determine what we have to show to allow the use of 100 to 120 MPa (14500 to 17300 psi) concrete in columns needs to be undertaken. This program should involve both design and construction practitioners. Some relationship between the quantities of longitudinal and tie reinforcement, concrete strength, and ductility needs to be established. To achieve acceptable structural design, it may be necessary to tune the strength-reduction factors for the columns to the amount of ductility achieved. This must be with regard to construction cost and

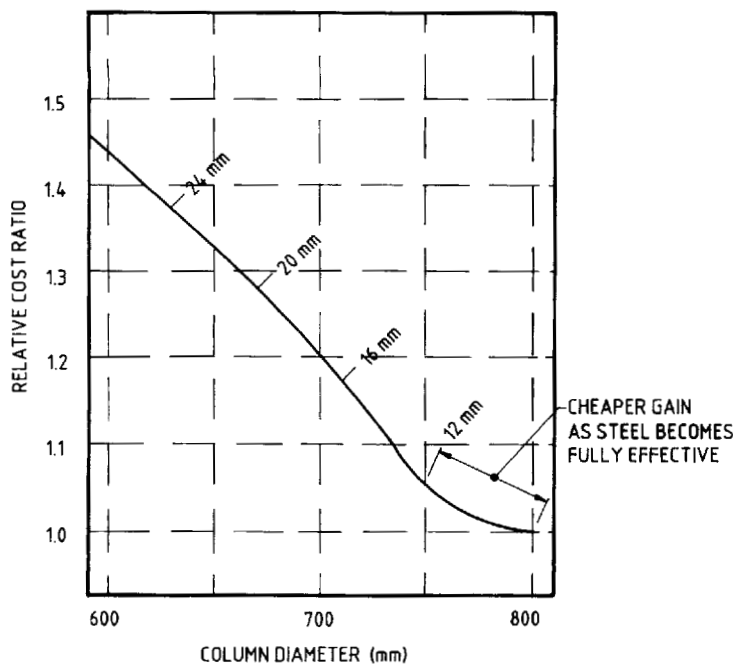


Fig. 7 — Cost to reduce the size of an 800 x 10 CHS G350 steel tube filled with 60 MPa concrete by using thicker plate.

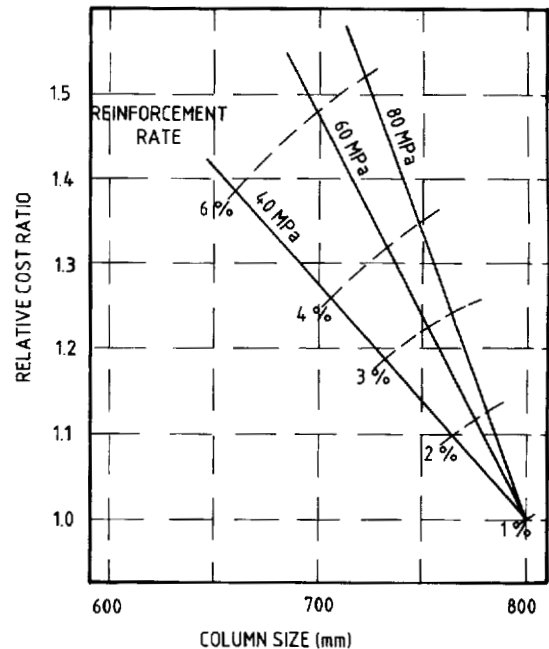


Fig. 8 — Cost to reduce the size of an 800 x 800 mm RC column with 1 percent reinforcement by adding reinforcement.

constructability will be needed to produce the optimum result.

The first step in using higher strength concretes will be its use in steel tubes, and research is under way to justify this use of the material. Tube-column construction is a construction breakthrough that provides great constructability and economy.

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Connell Wagner have been extensively involved in using high strength concrete in their structures in Australia, particularly in the last five years. Webb was project leader on the Casselden Place project, which featured the first Australian use of high strength concrete in unreinforced tubes.

The State of the Art of High-Strength Concrete in Chicago



225 W. Wacker Drive

by Jaime Moreno

The development of high-strength concrete began in Chicago in 1962 with 6000 psi (41 MPa) concrete, culminating in 1989 with the delivery of silica fume mixtures of 14,000 psi (96 MPa) for six stories of columns and one 17,000 psi (117 MPa) experimental column to the 225 W. Wacker project. The experimental column has provided information on structural properties of this new concrete strength. This project is representative of the state of the art of high-strength concrete in terms of strength, concrete quality, testing, construction techniques, and structural design.

Strength is one of concrete's structural properties that the concrete construction industry has been continually striving to improve. This has been to satisfy developers' need for higher structures as well as for larger rentable floor areas, which is accomplished by using smaller columns.

Keywords: chlorides; creep properties; heat of hydration; high-rise buildings; high-strength concretes; mix proportioning; modulus of elasticity; performance; permeability; reviews; shrinkage; silica fume; tests.

In the Chicago area, the race for higher and higher compressive concrete strengths began in 1962, and Material Service Corporation was the first supplier to develop and deliver to the job site the strengths shown in Table 1. Other suppliers subsequently have delivered concrete of strengths up to 12,000 psi (83 MPa) in the Chicago area.

The 17,000 psi (117 MPa) concrete listed in Table 1 has been delivered on four different occasions to Construction Technology Laboratories for investigation of shear strength, bond, and structural properties testing of high-strength concrete. 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.

Structural design

The columns that support the 31 stories at 225 W. Wacker Dr. were designed with concretes from 4500 to 14,000 psi (31 to 96 MPa). High-

strength concrete was used in the columns as an economical means of satisfying the structural requirements for the job.

The book price for 3000 psi (21 MPa) concrete is \$45/yd³; the price is \$140/yd³ for 14,000 psi (96 MPa) concrete. This increase of only 3.1 times in price for an increase of 4.7 times in load-carrying capacity clearly demonstrates the economy of using high-strength concrete in multistory building columns.

The first five stories and the basement columns of the building contain 14,000 psi (96 MPa) concrete. Concrete strengths change from 14,000 to 4500 psi (96 to 31 MPa) at the appropriate floor levels to obtain close to 1 percent reinforcing in the columns. This technique produced the most economical columns.

To satisfy strength requirements, column mixtures of over 10,000 psi (69 MPa) were designed with silica fume, which lends a dark color to the concrete as compared to the light color of the mixtures not used in the slabs (Fig. 1).



Fig. 1—Color contrast of column silica fume concrete with slab concrete.

For esthetic reasons, the architect requested that the same color be provided where the slab and column concrete are both visible, as is the case in the garage. To satisfy this requirement, a 14,000 psi (96 MPa) mixture with no silica fume was delivered for 18 columns, a total of 52 yd³. Optimum use of cement, flyash, additives, and granulometry in the mixture resulted in concrete strength of over 14,000 psi (96 MPa) without the use of silica fume. (A similar mixture using superplasticizer for the first time was delivered to the Chicago Mercantile Exchange in 1982.)

Mix design

Continuous monitoring of cementitious materials and additives and a thorough understanding of concrete technology have enabled quality control personnel to develop a series of optimum mixtures, all of which have been designed to perform specific functions, and to enhance the physical properties of concrete. These mixtures are called "performance concrete."

Maintaining a statistical history of test records from the production facility in accordance with Section 4.3.1 of ACI 318-83 allows this type of procedure for the acceptance of performance concretes. Generally, the specifications that stipulate the

parameters of performance are more successful than those attempting to prescribe ingredients. For 225 W. Wacker Drive, the test results provide the statistical history shown in Table 2.

Table 3 is a summary of the statistical history of the high-strength concrete used in this project delivered from the same production facility.

The total amount of concrete delivered was 40,100 yd³ (30,476 m³). The volumes of 14, 12, 10, and 8,000 psi (96, 83, 69, and 55 MPa) concrete were 3.6, 2.5, 5.2 and 4.5 percent of the total, respectively. These percentages of high-strength concrete are larger than the 2 to 2.5 percent previously delivered to similar jobs.

Although they are small percentages of the total, they are commercially important to the producer because they indicate performance ability for future contracts and the experience increases the knowledge of quality control personnel, improves consistency of low-strength concrete, and enhances the technological image and credibility of the company in the industry. Also, the profit is higher than with low-strength concrete.

For economical reasons, high-strength concrete mixtures for columns and shearwalls are designed for structural strength at 56 or 90

Table 1—High-strength concretes delivered for the first time to Chicago projects

1962	Outer Drive East	6,000 psi	41 MPa
1965	Lake Point Tower	7,500 psi	52 MPa
1972	Mid-Continental Plaza	9,000 psi	62 MPa
1976	River Plaza	11,000 psi	76 MPa
1982	Chicago Mercantile E.	14,000 psi	96 MPa
1988	Construction Technology Laboratories	17,000 psi	117 MPa

Table 2—Statistical history of 14,000 psi concrete for 225 West Wacker Drive Project

Production facility	Yard 1	—
f'_c at 56 days	14,000 psi	96 MPa
f'_c	15,687 psi	108 MPa
Mean	16,139 psi	111 MPa
Standard deviation	939 psi	6.5 MPa
Coefficient of variation	5.8 percent	—
Maximum	18,040 psi	124 MPa
Minimum	14,165 psi	98 MPa
Number of Consecutive Tests	40	—
ACI Requirement	Passes	—

days. The price of the mixture is that of a lower strength mixture at 28 days but it will develop the specified strength at 56 days. The code variation from the traditional 28 days is justified because load-supporting members are not fully loaded until several months after the concrete is placed. In this project, the strength requirements were specified at 56 days.

Another performance requirement specified by the contractor was a concrete that would develop such strength as to permit the removal of the forms 16 hours after the concrete had been placed. This requirement was met by means of a compromise between the amount of retardant used (to reduce the heat of cement hydration) and the tensile strength in the concrete needed to surpass the cohesive force between the forms and the concrete.

Silica fume mixture

Silica fume was used with cement and fly ash as one of the cementitious materials in most of the concrete over 10,000 psi (69 MPa) delivered to this project. The high content of silica in this product makes it an excellent pozzolanic material, reacting with calcium hydroxide and water to form additional cementitious material. Silica fume is commercially available as a dry densified granular product or as

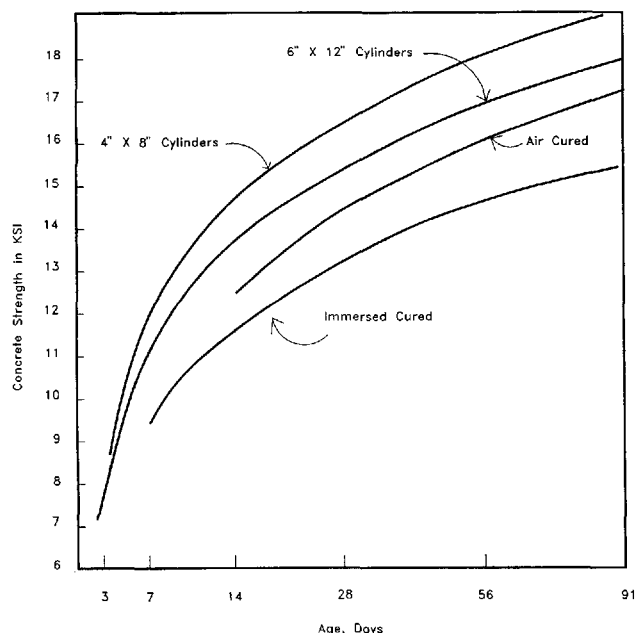


Fig. 2—Strength gain of 17,000 psi (119 MPa) concrete.

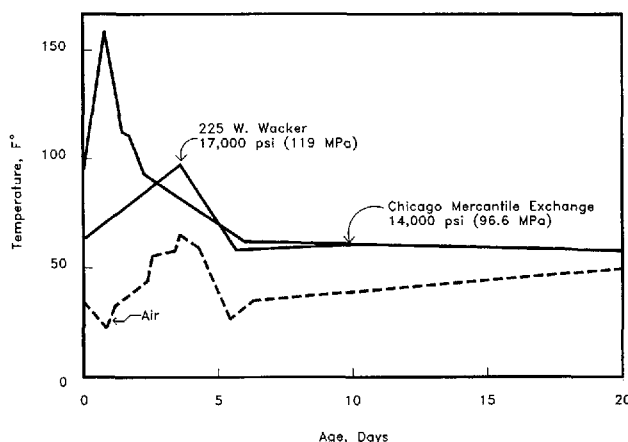


Fig. 3—Measured temperatures.

a liquid slurry. The liquid form was used in this project.

The high densities of silica fume concrete and high-strength concrete columns prevent saturation of the concrete surface when exposed to occasional wetting. Also, the vertical position of the columns eliminates the possibility of water ponding. Consequently, there is no water available on the concrete to freeze and damage the concrete surfaces. Because of these factors, no air has been added to high-strength concrete columns.

Low electrical conductivity in silica fume mixtures inhibits the electrolytic reaction, greatly diminishing corrosion in the reinforcement. Results of conductivity tests performed according to the AASHTO T277-83T and reported by Construction Technology Laboratories are shown in Table 4.

Because silica fume mixtures show very low chloride permeability they are well suited for use in structures in corrosive environments. The availability of silica fume and high-range water reducers has allowed the ready-mixed concrete industry to attain strength levels previously delivered only by producers with outstanding technology and quality control. Material Service has delivered experimental performance concretes with strengths up to 17,000 psi (117

MPa). These concrete strengths exceed the present structural market requirements for most high-rise buildings.

Strength testing

A set of 6 x 12 in. (15 x 30 cm) cylinders were made in plastic molds according to the ASTM C 31 recommendations and were tested for strength, creep, and shrinkage. The compressive testing of the cylinders was performed using a high-strength capping compound. Due to the amount of retardant in the 14,000 psi (96 MPa) mixture, cylinders were left in the field for 40 hours to obtain adequate concrete setting, then moved to the testing laboratory.

A parallel set of 4 x 8 in. (10 x 20 cm) 14,000 psi (96 MPa) cylinders was tested and compared with the standard 6 x 12 in. (15 x 30 cm) cylinders. The results can be seen in Fig. 2. Also, a parallel set of 4 x 8 in. (10 x 20 cm) 12,000 psi (83 MPa) cylinders was tested and compared with 6 x 12 in. cylinders. Table 5 gives the statistical history of the comparison.

Placing the concrete

The contractor elected to have high-strength concrete mushrooming in the slabs as a way to transfer the load throughout (Fig. 1). This method is more reliable and eco-

nomical than using dowels and spiral reinforcement. The column and shearwall concrete and the concrete for the slab mushrooming was placed using 2 yd³ construction buckets. Two different strengths and careful scheduling were required to place the concrete in the slabs to comply with the ACI requirement. All of the 4000 psi (28 MPa) concrete for the slabs was pumped.

Variations in site conditions

There were two variations in construction techniques at the job site due to the use of high-strength concrete:

- There was some difficulty releasing the form wedges that was attributed to concrete expansion due to the high increase in temperature during cement hydration.
- The contractor requested a performance mixture that would allow column form release 16 hours after concrete placing. The original mixture did not have this capability. The cohesive force between the concrete surfaces and form surfaces in some areas were larger than the internal cohesive concrete force. This condition produced spalling of some column and shearwall surfaces when the forms were removed. These unexpected conditions were corrected by reducing the amount of retardant in the original

Architects and engineers: Perkins & Will
 Developers: The Palmer Group, Ltd.
 General contractor: Crane Construction Company
 Testing laboratory: Flood Testing Laboratories
 Ready-mixed concrete supplier: Material Service Corporation

Table 3—Summary of statistical history for high-strength concrete for 225 West Wacker Dr. Project

Concrete strength,		Coefficient of variation, percent	Standard deviation		Mean		Quantity of concrete,	
psi	MPa		psi	MPa	psi	MPa	yd ³	m ³
17,000	117	—	—	—	—	—	52	40
14,000	96	5.8	939	6.5	16,139	111	1460	1110
12,000	83	6	684	4.71	3,331	92	1020	775
10,000	69	5	346	2.4	11,526	79	2100	1596
8,000	55	5	409	2.8	10,207	70	1810	1376

Table 4—Results of conductivity tests on high-strength concretes

Average strength		Chloride permeability, charged passed (coulombs)
psi	MPa	
6,990	48	4237
10,020	69	2520
12,580	87	1580
10,070	74	650 (10 percent silica fume)
17,490	120	67 (15 percent silica fume)

Table 5—Comparison of test results on cylinders of 12,000 psi (83 MPa) concrete at 56 days

Cylinder size	6 x 12 in.	4 x 8 in.
Mean	13,444	13,546
Standard deviation	463	515
Coefficient of variation, percent	3.4	3.8
Number of tests	29	29

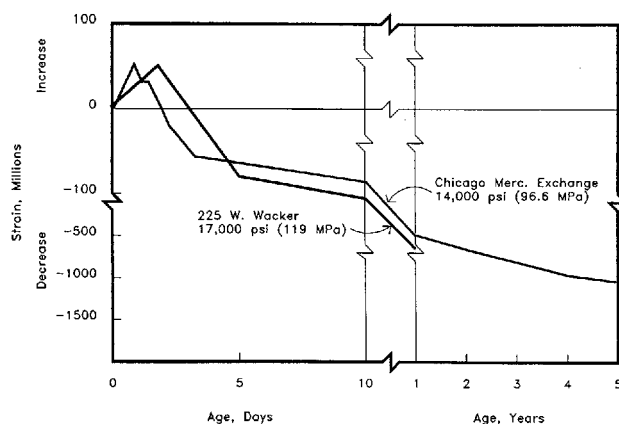


Fig. 4—Column strains.

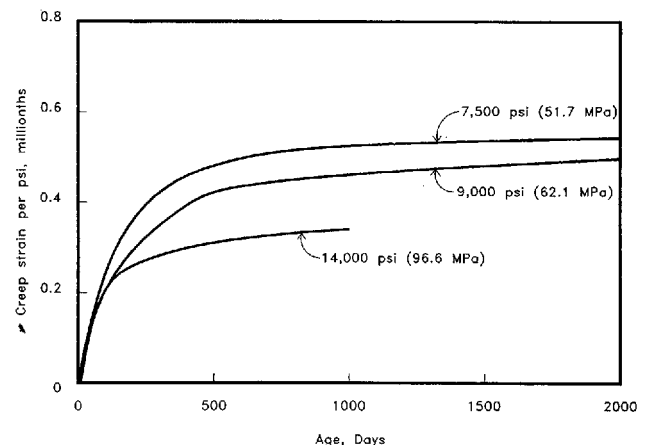


Fig. 5—Measured creep.

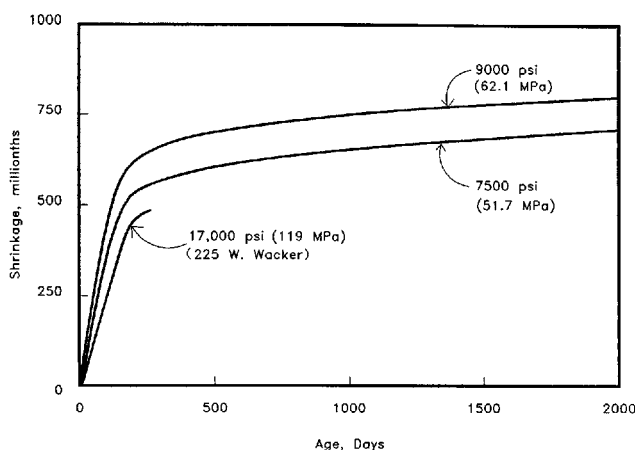


Fig. 6—Measured shrinkage.

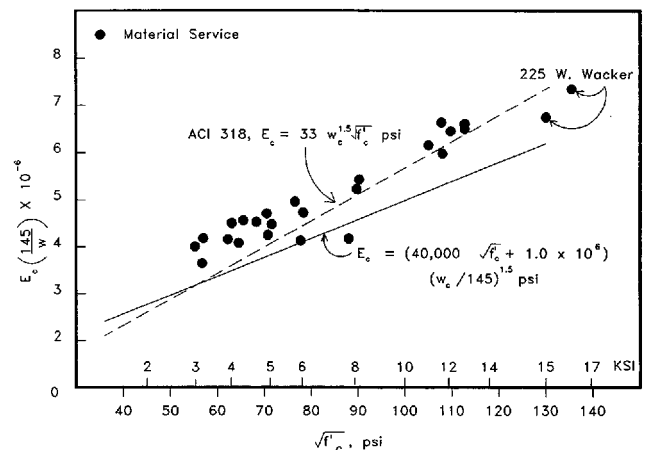


Fig. 7—Modulus of elasticity.

mixture. Subsequently, forms were consistently removed after 16 hours without spalling of the concrete surface.

Research project

To continue the research initiated with strengths of 7.5, 9, 11 and 14,000 psi (52, 62, 76 and 96 MPa) concretes, a new concrete strength of 17,000 psi (117 MPa) was placed in a column at the basement level. The strength development of this mixture is shown in Fig. 2. Strain and temperature meters were placed inside the column and cylinders were made to obtain data on the physical and structural properties of this new strength as follows:

Heat of hydration was diminished to about 100 F (38 C) with by using a retardant from the more than 150 F (66 C) recorded at the Chicago Mercantile Exchange (Fig. 3). High temperatures due to cement hydration usually necessitate variations in construction procedures because of effects such as form removal problems or excessive shrinkage cracks after removal of the forms. The graph also shows that the concrete temperature approximates the air temperature after about 20 days.

Shortening in columns has been measured in high-strength concrete columns using strain meters. Fig. 4 shows that after a few days the strains are about the same for 14 and 17,000 psi (96 and 117 MPa) concrete. Measurements for more than five years of 14,000 psi (96 Mpa) concrete show the columns becoming stable at a strain of about 1000 in./in.

Creep and shrinkage testing have been carried out on cylinders obtained from concrete delivered to projects where high-strength concrete has been used. Fig. 5 and 6 show the decrease of creep and shrinkage with the increase of concrete strength.

Modulus of elasticity of high-strength concretes including this new strength of 17,000 (117 Mpa) is shown in Fig. 7. These results are less than values given by the traditional ACI equation, however, they exceed those produced by the new equation proposed by the ACI Committee 363, High Strength Concrete. This suggests that the equation recommended by Com-

mittee 363 is more representative for high-strength concrete.

Conclusions

The experience and information obtained from this project have provided the following conclusions:

1. This project is representative of the state of the art of high-strength concrete in Chicago in terms of strengths, concrete quality, testing, construction techniques, and structural design.

2. The statistical history of test results assures that the 14,000 psi (96 MPa) concrete mixture delivered to this project is reliable for delivery in the Chicago area.

3. The existing physical and structural properties of this mixture allow structural engineers to use concrete of this strength.

4. No critical problems confronted the contractor at the job site with the handling, placing, and curing of concrete of this strength.

5. Information on 17,000 psi (117 Mpa) concrete is being developed for safe use of this material in the near future by the construction industry.

6. The use of any new concrete strength is a team effort of the whole concrete industry: the ready-mixed concrete company has to produce it, the designer has to specify it, the contractor to handle it, and the city and code officials to accept it. Any reluctance on the part of any member of this team would strongly inhibit the use of a new concrete strength.

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Proportioning High-Strength Concrete to Control Creep and Shrinkage



by Therese M. Collins

Results of an experimental creep and shrinkage program on high-strength concrete are presented. Variations in paste content and maximum aggregate size also are investigated. Five test mixes were prepared based on historical data. These mixes varied in paste content, aggregate size, and the use of high-range water-reducing admixtures. The creep test conditions varied only in intensity of load.

The results of the test program showed that creep and shrinkage deformations are somewhat less for concrete mixtures with lower paste contents and larger aggregate size. Although creep and shrinkage may be reduced by careful mix proportioning and aggregate selection, the applied stress level (creep load) has a much more significant effect on column shortening.

Keywords: aggregate size; cement content; creep properties; creep tests; high-strength concretes; mix proportioning; shrinkage; strains.

Concrete is commonly used in constructing building frames. Recent developments in high-strength concrete have extended its use to tall buildings. As taller building frames are being constructed with high-strength concrete, creep and shrinkage deformations have become an important concern. In very tall buildings, 100 to 150 stories, creep may be as important as strength for floor levelness and exterior skin movement. However, little information is available on the creep and shrinkage characteristics of high-strength concrete, and the information that is available is inconclusive. This lack of information makes it nearly impossible to draw direct conclusions and make design judgments on the resulting deformations of high-strength concrete in tall buildings. In the future, concrete technologists will be faced with the design of concrete with the most desirable creep and shrinkage characteristics while still meeting strength requirements.

Creep is defined as the "time dependent increase in strain in a solid resulting from force." Concrete creep is the deformation and flow of matter associated with the gel structure of the cement paste. The shrinkage and swelling of concrete is produced by the movement of water into and out of the gel due to external load and drying. Concrete creep is influenced by mixture proportions, type of aggregate, and age at loading. Con-

crete shrinkage is directly related to water-cement ratio, mixture proportions, type of aggregate, and curing conditions.

Many concrete investigators believe creep and shrinkage are closely related. While shrinkage of concrete can be associated with changes in ambient humidity and differences in moisture conditions between the concrete and the environment, creep is associated more with the effect of applied pressure and the resulting gel water movement. Shrinkage is generally an undesirable quality of concrete, while in some instances creep may be considered desirable. If the creep is predicted accurately and design compensations are made, the result will be better distribution of stresses in reinforced concrete structures. Unfortunately, when the deformations and deflections due to creep are excessive, or design compensations are not made, the result can be expensive repairs and possible loss of prestress in prestressed concrete members.

The amount of information available on the creep characteristics of high-strength concrete is limited. Some creep data on normal strength concrete is available; however, the structure of high-strength concrete varies significantly with that of normal strength concrete, thus making direct comparisons invalid. Because the creep of high-strength concrete is associated directly with the degree of hydration (concrete creep will decrease as cement hydration increases), creep measurements must be made over a long period of time. The majority of creep testing performed to date has elapsed over a short period of time (90 days or less), thus providing information while the cement is still hydrating at a relatively rapid pace. This information is less accurate than creep testing performed over a longer

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period of time (1 year or more) that allows the cement to hydrate more completely.

The relationship of paste content to creep in the field of high-strength concrete has long been of interest. Several sources suggest that the lower the paste content, the lower the resulting creep. Also, the creep of concrete will decrease as the water-cement ratio decreases.

For low creep and shrinkage, a hard, dense, low absorption aggregate such as dolomitic limestone is desired. Studies have shown that creep and shrinkage generally decrease as maximum coarse aggregate size is increased. Generally, the lower the water-cement ratio, the larger the aggregate size . . . and the lower the paste content of the concrete mixture, the more desirable the creep and shrinkage characteristics will be.

An experimental investigation was conducted on high-strength concrete produced in the St. Paul-Minneapolis area. The test program consisted of preparing five concrete mix designs and documenting the strength, modulus of elasticity, creep, and shrinkage characteristics.

MATERIALS

The materials used in the test program consisted of the following:

Portland cement—According to the Chicago Committee on High Rise Building, the choice of a brand and type of cement is probably the most important factor in the selection of materials for high-strength concrete. Variations in the properties of the cement will cause the concrete compressive strength to fluctuate more than any other single material. The portland cement used was a regular Type I portland conforming to ASTM C 150 specifications. From February 1976 to February 1977, the National Ready Mix Concrete Association conducted a test program on portland cement. Grab samples were taken from 25 ton (22.68 Mg) lots of cement from approximately 100 cement plants over a wide geographic area in the United States and Canada. The information from the test program shows the uniformity of the cement used in the testing program is within the top 5 percent of the 100 cement mills tested.

Fly Ash—The fly ash used was Class C, and the plant burned 100 percent nontreated Western coal.

Fine aggregate—The high-strength concrete produced for this study incorporated two local sources of fine aggregate. Both fine aggregates are processed natural sand of glacial origin. Fine aggregate #1 was a natural sand with a fineness modulus of approximately 2.6. Fine Aggregate #2 was a mined gravel deposit with a fineness modulus of about 2.80.

Coarse aggregate—The aggregate used was a crushed dolomitic limestone. The aggregate was similar to that used in the Chicago and Dallas areas for high-strength

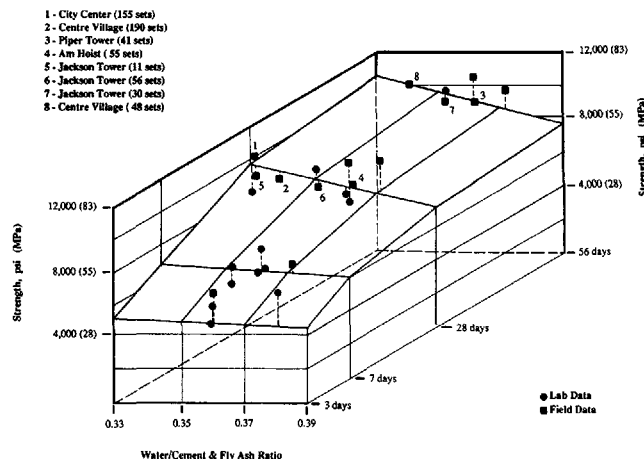


Fig. 1 — Historical data

concrete. Two local sources of coarse aggregate were used in the program. Coarse aggregate #1 was mined southeast of the metropolitan area. In making a material to meet #67 size, the material from #6 and #7 stockpiles were combined at the proper ratio. The crushed stone was washed immediately before loading onto barges, which were shipped to the metropolitan plants. Coarse aggregate #2 was a dolomitic limestone from a quarry in Burnsville, a southern suburb. The dolomitic limestone was from the Shakopee formation.

The high-strength concrete for the test program contained aggregate with a maximum size of 1-½ and 1 in. (3.81 and 2.54 cm). The 1-½ in. (3.81 cm) aggregate was a blend of dolomitic limestone meeting MN/DOT CA3 gradation requirements and ASTM C 33, size #67 materials. The aggregate was blended at a ratio of 30 percent CA3 to 70 percent #67.

Chemical admixtures—Chemical admixtures play an important part in consistently attaining high strength in concrete. The concrete batch contained two admixtures meeting ASTM C 494 requirements for a Type A water-reducing admixture. Also, a high-range water-reducing admixture meeting ASTM C 494 requirements for a Type F admixture was used.

EXPERIMENTAL PROGRAM

The mix designs used in the test program were based on field experience. In selecting the proportions of concrete, a concrete production facility must have a record based on at least 30 consecutive strength tests that represent similar materials and conditions to those expected. After reviewing the available historical information, a design average strength of 1000 psi (7 MPa) over the f'_c [8000 psi] (55 MPa) was selected.

A review of the historical data indicated water/cement + fly ash ratios of 0.33 and 0.40 have been used in making high-strength concrete in the St. Paul-Minneapolis area. Fig. 1 is a composite of this test information. The figure indicates water/cement + fly ash ratio of 0.36 would yield an average compressive strength of 9000 psi (62 MPa) at an age of 56 days.

Table 1 — Concrete mix proportions

Mix number	1	2	3	4	5
Cementitious material					
Type I portland, lb	700	630	680	620	700
Fly ash, Class C, lb	120	110	120	110	120
Total	820	740	800	720	820
Fine aggregate, lb	1145	1295	1135	1275	1140
Course aggregate, lb					
1½ to ¾ in.	—	—	555	555	—
¾ in. - #4	1760	1760	1295	1295	1750
Total	1760	1760	1850	1850	1750
Admixture, oz					
Water reducer	33	—	32	—	29
HRWR	—	90	—	85	—
Water, net lb	295	265	280	250	295
Water/cement + fly ash ratio	0.36	0.36	0.35	0.35	0.36

1 lb = 0.4536 kg; 1 oz = 29.57 cm³.**Table 2 — Plastic concrete test results**

Mix number	1	2	3	4	5
Slump, in.	3½	4¾	3¾	2¾	4¾
Cement factor, sx/yd ³	817.8	742.6	789.6	714.4	808.4
Air content, percent	1.8	1.1	1.6	1.8	1.7
Unit weight, lb/ft ³	149.7	152.9	151.4	151.0	149.3

1 in. = 2.54 cm; 1 yd³ = 0.765 m³; 1 lb/ft³ = 0.0624 kg/m³.

The five mix designs used in the test programs had a targeted water/cement + fly ash ratio of 0.36. Mix #1 was a commonly used high-strength mix that gave an average strength of 9019 psi at an age of 56 days. Mixes #2, #3, and #4 contained a lower paste content while maintaining a water/cement + fly ash ratio of 0.36.

Mix #2 incorporated a high-range water-reducing admixture conforming to ASTM C 494, Type F requirements, which permit a reduction in cementitious materials of 80 lb (36 kg). Mix #3 used a larger maximum size aggregate, permitting a reduction in the cement factor of approximately 20 lb (9 kg). Mix #4 incorporated a combination of the high-range water-reducing admixture and a large coarse aggregate to reduce the cement and paste content 90 lb/yd³ (41 kg/yd³). Mix #5 was similar to Mix #1, with the exception of an alternate crushed dolomitic limestone source. Table 1 summarizes mix proportions used in the test program.

Concrete batching

The five concrete test batches were prepared at two local ready-mix concrete plants. The concrete was batched in two yd³ batches. The concrete was delayed at the plant for 15 min to simulate transit to the jobsite. The slump, air content, unit weight, and the temperature of each test mix were recorded and are shown in Table 2. At the completion of testing, forty 6 x 12 in. (15.2 x 30.5 cm) standard cylinders were cast for each mix for use in compressive strength testing. The compressive strength specimens were cast in steel molds. For each mix, eighteen 6 x 12 in. (15.2 x 30.5 cm) standard cylinders were cast in plastic molds for use in creep and shrinkage testing. The concrete test specimens remained at the ready-mix concrete plant for 1 day prior to shipment to a testing laboratory.

The cylinders were then placed in a moist-curing room [73 F, 100 percent relative humidity (R.H.) (23 C)] prior to further testing. The compressive strength as documented at 3, 7, 28, 56, and 180 days. The modulus of elasticity at 28 days was also documented.

Creep and shrinkage testing

Creep and shrinkage testing was performed in accordance with ASTM C 512, "Standard Test Method of Creep of Concrete in Compression" at the University of Texas at Arlington. The test program was conducted in an environmental room maintained at 70 F (21 C) and 50 percent R.H. Shrinkage testing was performed on concrete specimens after moist curing for 7 and 28 days, while creep testing was performed only on specimens after moist curing for 28 days. Gage points were attached to the test specimens at 90-deg angles. The longitudinal deformations of creep and shrinkage were measured using a 10-in. (25.4-cm) mechanical strain indicator. For each set of creep and shrinkage specimens, an electrical strain gage was mounted on one cylinder.

Three test specimens were loaded in each creep rack. A total of 20 racks were used for the test program. Creep loads of 0.2 f'_c , 0.3 f'_c , and 0.4 f'_c were applied to 28-day-old unsealed test specimens. An additional set of sealed 28-day-old test specimens was loaded at 0.3 f'_c .

Elastic strains were measured immediately after loading, and creep strains were measured at intervals proportional to the rate of deformation (see Table 3).

EXPERIMENTAL RESULTS

Compressive strength testing

A review of the test data indicates that the compressive strengths of Mixes #1 and #5 were higher than the remaining three mixes. The 28-day strengths ranged from 8050 to 9280 psi (56 to 64 MPa) for Mixes #4 and #5, respectively. The 56-day strengths ranged from 8650 psi (60 MPa) for Mix #3 to 9290 psi (64 MPa) for Mix #1. A summary of the compressive strengths for all five mixes appears in Table 4.

Table 3 — Summary — Creep loads and elastic strain of creep specimens

Mix number	1	2	3	4	5
Stress-strength ratio f_c/f'_c	0.4 0.3 0.3* 0.2	0.4 0.3 0.3* 0.2	0.4 0.3 0.3* 0.2	0.4 0.3 0.3* 0.2	0.4 0.3 0.3* 0.2
Applied stress f_o , psi	3514 2635 2635 1757	3294 2470 2470 1647	3422 2566 2566 1711	3218 2414 2414 1609	3712 2784 2784 1856
Elastic strain, 10-6 in./in.	808 644 572 405	593 488 509 287	801 553 570 416	750 545 538 409	814 636 639 364

1 psi = (1.45E - 04)Pa; 1 in. = 2.54 cm.

*Sealed specimen.

Table 4 — Compressive strength results

Mix number	1	2	3	4	5
3-day compressive strength, psi					
Cylinder #1	5130	4450	4320	5010	5400
Cylinder #2	5200	4320	4480	5130	5310
Average	5170	4390	4400	5070	5360
7-day compressive strength, psi					
Cylinder #1	6580	6190	6320	6160	6360
Cylinder #2	6070	6180	6370	6010	6470
Average	6330	6190	6350	6090	6420
28-day compressive strength, psi					
Cylinder #1	8760	8300	8560	7880	9140
Cylinder #2	8810	8170	8550	8210	9420
Average	8790	8240	8550	8050	9280
56-day compressive strength, psi					
Cylinder #1	9360	8940	8300	8850	9230
Cylinder #2	9210	9200	8590	9010	9310
Average	9290	9070	8450	8930	9270
108-day compressive strength, psi					
Cylinder #1	10,520	10,310	9970	10,100	10,980
Cylinder #2	10,350	10,240	9750	10,000	10,750
Average	10,440	10,280	9860	10,050	10,870

1 psi = (1.45E - 04)Pa; 1 in. = 2.54 cm.

Table 5 — Shrinkage strain data, 10-6 in./in.

Mix number	1			2		3		4		5	
Time, days	28U*	7U	28S†	28U	28S	28U	28S	28U	28S	28U	28S
1	12	30	9	16	16	15	17	10	12	19	12
3	37	79	26	29	28	21	30	24	25	23	24
7	55	114	48	57	60	50	56	48	46	61	59
29	170	277	175	169	178	156	161	182	169	182	214
56	262	389	262	266	267	230	241	262	253	269	306
90	280	431	291	289	295	258	273	318	304	344	383

1 in. = 2.54 cm.

*U = Unsealed.

†S = Sealed.

Table 6 — Creep strain at various stress-strength ratios, 10-6 in./in.

Mix number	1	2	3	4	5
Time, days	0.4 0.3 0.2	0.4 0.3 0.2	0.4 0.3 0.2	0.4 0.3 0.2	0.4 0.3 0.2
1	263 167 88	172 133 89	244 166 132	182 131 114	291 196 141
3	336 192 141	249 218 173	337 234 206	332 208 176	405 289 183
7	497 285 178	329 297 182	443 293 230	445 264 237	576 412 244
29	867 516 335	572 486 300	740 502 372	805 483 376	968 664 388
56	1139 705 478	797 704 449	978 647 480	1095 658 507	1261 872 499
90	1391 872 590	975 880 539	1155 810 558	1310 812 585	1482 1031 614

1 in. = 2.54 cm.

Shrinkage testing

The shrinkage deformation of 7-day moist-cured specimens was higher than the deformation of 28-day moist-cured specimens. For Mix #1, the shrinkage strain at 90 days was 431 μ -in./in. and 280 μ -in./in. for 7 and 28-day-old specimens, respectively.

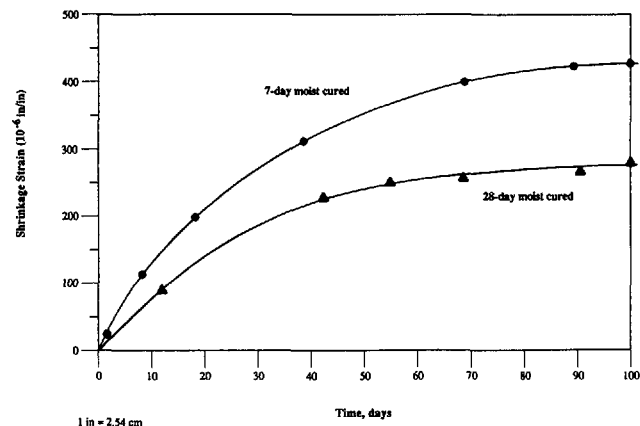
Mix #5 showed the highest shrinkage deformation, while Mix #3 showed the lowest. At 90 days, the shrinkage strain of the 28-day moist-cured specimens was 344 μ -in./in. for Mix #5 and 258 μ -in./in. for Mix #3. A summary of shrinkage data is shown in Table 5. Shrinkage-time curve for Mix #1 appear in Fig. 2.

Creep testing

The creep-time test data indicate that the creep strains increase with an increase in stress levels. At 90 days, Mix #5 had creep strain at 1482 μ -in./in., 1031 μ -in./in., and 614 μ -in./in. for creep loads of 0.4 f'_c , 0.3 f'_c , and 0.2 f'_c , respectively. These values represent the highest creep strain of the five test mixes. Mix #2 exhibited the lowest creep strains. At 90 days, Mix #2 and creep strains of 275 μ -in./in., 880 μ -in./in., and 539 μ -in./in. for creep loads of 0.4, 0.3, and 0.2, respectively. A summary of creep data is shown in Table 6.

CONCLUSIONS

The following conclusions are based on the test program described in this paper:

**Fig. 2 — Shrinkage strain versus time curves: Mix #1**

1. Considering all factors outlined in the test program (compressive strength, creep, and shrinkage characteristics), Mix #1 and Mix #3 would be the optimal choices to provide the most desirable creep and shrinkage characteristics while meeting strength requirements.

2. The shrinkage deformation is inversely proportional to the moist-curing time. The longer the moist-curing time, the lower the shrinkage deformation. For Mix #1, the shrinkage of a 7-day moist-cured specimen is approximately 50 percent higher than that of a 28-day moist-cured specimen.

3. The creep deformation increases directly with an increase in the applied stress level. For the five test mixes, the average creep strain at $0.3 f'_c$ is approximately 50 percent higher than that of $0.2 f'_c$, and the average creep strain at $0.4 f'_c$ is 125 percent higher.

4. The creep and shrinkage deformations are lower for Mix #3 than for Mix #1. The mix proportions of the two mixes are relatively the same, except that Mix #3 utilizes a larger maximum aggregate size and a lower paste content. This suggests that a concrete with a large maximum aggregate size [$1\frac{1}{2}$ in. (3.81 cm)] and a low paste content will provide more desirable creep and shrinkage characteristics.

5. The use of a high-range water-reducing admixture did not have a significant effect on the creep and shrinkage deformations.

6. By lowering the paste content and maximizing the coarse aggregate, a high-strength concrete can be pro-

duced with a modest improvement in creep and shrinkage deformations. However, the reduction in stress level has a much more significant effect on the resulting creep and shrinkage strains. To lower the stress level, a concrete facility should explore the highest strength, most dense concrete possible.

7. Future research projects including the use of silica fume and chemical admixture are necessary to develop high-strength concrete with desirable creep and shrinkage characteristics.

ACKNOWLEDGMENTS

The work described was a portion of a comprehensive test program performed by Twin City Testing Corporation, St. Paul, Minnesota. Creep and shrinkage testing was performed at the University of Texas at Arlington under the direction of Professor Robert Yuan. Special thanks to the J. L. Shiely Company and the Model Stone Company for donating their time and materials to this project.

REFERENCE

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Evaluation of Core Strength in High-Strength Concrete

by Robert L. Yuan, Mostafa Ragab, Robert E. Hill, and James E. Cook

High-strength concrete has gained popularity in recent years, mainly for its use in high-rise concrete buildings and in major structural components with large volumes. Concretes with strengths exceeding 8000 psi (55 MPa) are produced commercially using conventional methods and materials and are not unusual in construction today. As high-strength concrete becomes more technically and economically feasible and its use increases, it becomes necessary to understand the strength gain of in-place concrete and the relationship between actual concrete strength and standard cylinder strength to realistically evaluate the performance of structural components at any given age of the concrete.

The evaluation of in-place strength with regard to concrete mix design, placement schedule, and quality assurance is important from the construction point of view. Questions are often raised about whether or not the cylinder strength accurately represents the actual strength of concrete, and what the ratios are between the two strengths.

The core test is generally employed to determine the strength of in-place, hardened concrete. Cores are drilled from concrete structures, cut into the required sizes for testing, and, in most cases, are tested for compressive strength. The test procedures are the same as those for the standard cylinder test, but the core strength is generally lower than standard cylinder strength. The difference is due to the drilling process

(which disturbs the concrete), the ratio of the core diameter to the maximum size of the aggregate, and the location and direction of drilling.

The research reported here was undertaken by the authors to correlate high-strength concrete core strength to the strength of standard moist-air cured and field-cured cylinders.

Experimental program

Test specimens included two 6 x 6 x 6 ft (1.8 x 1.8 x 1.8 m), unreinforced concrete mock columns and 96 standard 6 x 12 in. (152 x 304 mm) cylinders. Column I was cast at a concrete placement temperature of 92 F (33 C), while Column II was cast at 103 F (40 C). Half of the cylinders were field-cured, and half were moist-air cured at 73 F (23 C) and 100 percent RH. All specimens were cast on the same day to maintain a proper comparison of strength gain between specimens at any testing age.

Mix characteristics, test procedures, and results

Column and cylinder specimens were prepared from the same concrete mix from a ready-mixed concrete truck. By weight, 70 percent

of the cementitious material was Type I portland cement from a single mill source, and 30 percent was ASTM C 618 Class C fly ash from a sub-bituminous coal. The coarse aggregate was a hard, crushed limestone with a maximum size of 1 in. sampled from the quarry stockpile and the fine aggregate was a water-classified natural sand. The water-cementitious material ratio was 0.30. Details of the mix proportions are given in Table 1.

A set of 48 standard cylinders was cast with each column. Twenty-four hours after casting, 24 cylinders from each group were moved to a moist-air curing room with controlled temperature and humidity. The remaining 24 cylinders of each group were kept in the field and cured in the ambient environment with the columns. Column forms were removed 24 hours after placement and the surfaces were sealed immediately with curing compound and wrapped with thermal insulation blankets for 7 days.

Four-inch (100 mm) cores were drilled from each column after 7, 28, 56, 180, and 365 days. For each operation, one core was taken from the west face to the center of the column (3 ft [1 m]) while the other core was taken from the east face to

Table 1 — Mix proportions

Specimens	Cement (lb)	Fly Ash (lb)	Water (lb)	Coarse Aggregate (lb)	Fine Aggregate (lb)	Retarder (oz)	SP (oz)
Column I	604	253	248	1995	1026	41	163
Column II	603	256	250	2000	1000	42	163

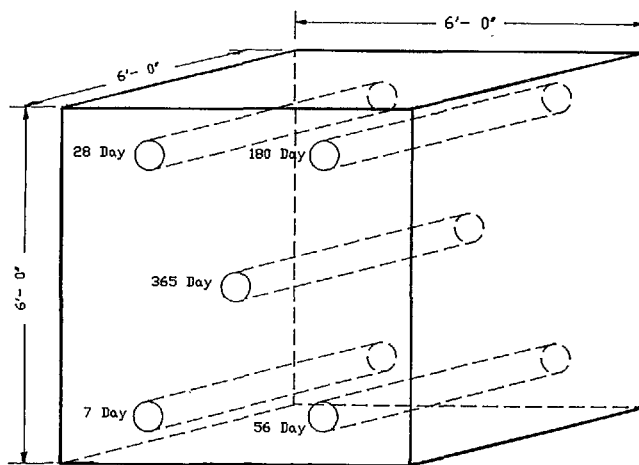


Fig. 1 — Coring procedures and locations.

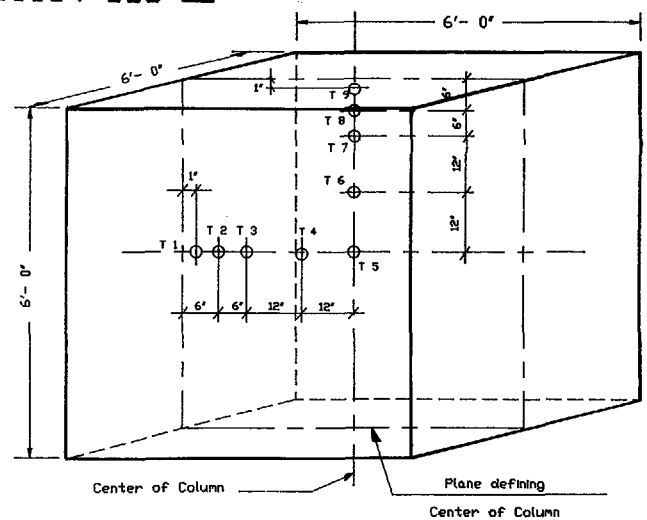


Fig. 2 — Location of thermocouples.

the center of the column (Fig. 1). Each core was cut into four cylinders with length-diameter ratios of 2. Thus, each column had a total of 8 cored specimens capped and tested for compressive strength at each testing age. Half of the cored

specimens were tested in Laboratory A and half were tested in Laboratory B to allow for equipment deviation. The average test results from both laboratories were used to determine core strength.

The moist-air cured and field-

cured cylinders were tested at 3, 7, 28, 56, 180, and 365 days. At each testing age, two field-cured and two moist-air cured cylinders from each group (a total of 8 specimens) were tested in Laboratory A and the same number of specimens were tested in Laboratory B. The average test results from the two laboratories were used for cylinder strength analysis.

Core specimens were prepared and tested in accordance with ASTM C 42, while cylinder specimens were capped and tested in accordance with ASTM C 617 and ASTM C 39.

Column temperatures during hydration were monitored continuously for 7 days using 9 Type T thermocouples installed in each column along the horizontal and vertical center lines (Fig. 2). The ambient temperature was also recorded.

The strength data for the moist-air cured and field-cured cylinders are

Table 2 — Strength development of concrete cylinders

Age (days)	Compressive strength of 6x12 in. cylinders, psi							
	Group I concrete Placement temperature 92 F				Group II concrete Placement temperature 103 F			
	Field cured		Moist cured		Field cured		Moist cured	
	Lab A	Lab B	Lab A	Lab B	Lab A	Lab B	Lab A	Lab B
3	7896	7864	7778	8076	7648	7898	7488	7714
7	9038	8545	9447	9695	9132	9153	9139	9445
28	10823	10080	11523	11805	10832	11135	12055	12130
56	11176	10727	12556	12396	10738	11414	12810	13172
180	11806	—	13753	14634	11413	—	14178	14924
365	—	—	14642	14738	—	—	14789	14808

Table 3 — Strength development of core specimens

Age (days)	Compressive strength of 4 x 8 in. cores, psi															
	Column I Placement temperature 92 F								Column II Placement temperature 103 F							
	Core no.								Core no.							
	1 Lab B	2 Lab A	3 Lab A	4 Lab B	5 Lab B	6 Lab A	7 Lab A	8 Lab B	1 Lab B	2 Lab A	3 Lab A	4 Lab B	5 Lab B	6 Lab A	7 Lab A	8 Lab B
7	8891	8248	7977	7343	8022	8257	9063	9416	8140	8004	7587	7379	7506	7660	8248	8429
28	8696	8239	8266	7529	8085	8141	8791	9280	8406	8656	8846	8521	8098	8447	8393	8629
56	9308	8650	8538	7538	8420	8990	9443	10159	9294	9271	8674	—	—	8311	9190	8673
180	10394	9681	9747	9429	10059	10451	11040	11107	10983	10540	10007	10014	9697	9357	9507	11062
365	11531	—	—	9382	11044	—	—	11793	11116	—	—	11553	11182	—	—	11069

shown in Table 2. The test results were produced by two independent testing laboratories, with each laboratory testing four specimens at each testing age.

Core strength data are shown in Table 3. The core specimens were drilled dry and tested in an air-dry condition. Four specimens were

tested by each lab at each testing age.

The average strengths of standard cylinders and cores are presented in Table 4. The laboratory moist-air cured cylinders had significantly higher strengths at all ages than the field-cured cylinders and the cores. The results indicate that

the compressive strength of moist-air cured cylinders is higher than that of field-cured cylinders and that field-cured cylinder strength is higher than core strength.

Strength development

The strength-age relationships for standard cylinders and cores are shown in Fig. 3 and 4. For moist-air cured specimens, there is rapid strength development at early ages, and the strength gain continues even after 6 months. For field-cured specimens, a moderate strength development is observed initially, and the rate of strength gain decreases with time. For core specimens, there is a rapid strength gain in the first 7 days, then little or no strength gain from 7 to 28 days, followed by rapid strength development after 2 months; after 1 year, the core strength approaches field-cured cylinder strength but is only equal to 28-day moist-air cured cylinder strength.

The effect of the curing environment on the change of strength ratios for high-strength concrete is shown in Table 5. Comparing field-cured cylinders to moist-air cured cylinders, the strength ratio decreases as the age increases and has an average value of 0.906. Comparing the cores and the moist-air cured cylinders, the strength ratio decreases for 2 months and quickly stabilizes with an average ratio value of 0.750. Comparing cores to field-cured cylinders, the average strength ratio is 0.850.

Effect of drilling location

For normal-strength concrete, it is generally recognized that the con-

Table 4 — Average strength values for standard cylinder and core specimens, psi

Age (days)	Group I concrete Placement temperature 92 F			Group II concrete Placement temperature 103 F		
	Field cured	Moist cured	Core	Field cured	Moist cured	Core
3	7880	7927	—	7773	7601	—
7	8792	9571	8402	9143	9292	7869
28	10452	11664	8492	10984	12093	8500
56	10952	12476	8881	11076	12991	8902
180	11806	14194	10239	11413	14551	10146
365	—	14690	10938	—	14799	11230

Table 5 — Effect of curing conditions on the change of strength ratio with age

Age (day)	Group I Concrete			Group II Concrete		
	Field-cured Moist-cured	Core Moist-cured	Core Field-cured	Field-cured Moist-cured	Core Moist-cured	Core Field-cured
3	0.99	—	—	1.02	—	—
7	0.92	0.88	0.96	0.98	0.85	0.86
28	0.90	0.73	0.81	0.91	0.70	0.77
56	0.88	0.71	0.81	0.85	0.69	0.80
180	0.83	0.72	0.87	0.78	0.70	0.89
365	—	0.75	—	—	0.76	—

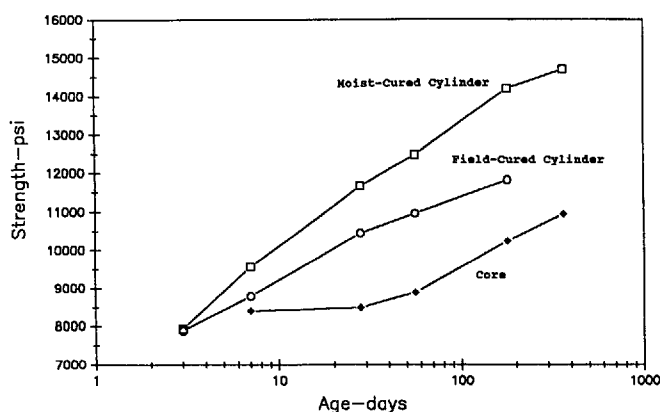


Fig. 3 — Strength-age relationships for cylinders and cores cast at 92 F.

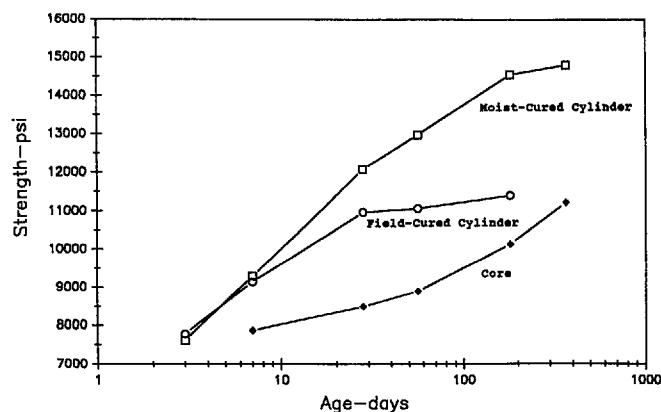


Fig. 4 — Strength age relationships for cylinders and cores cast a 103 F.

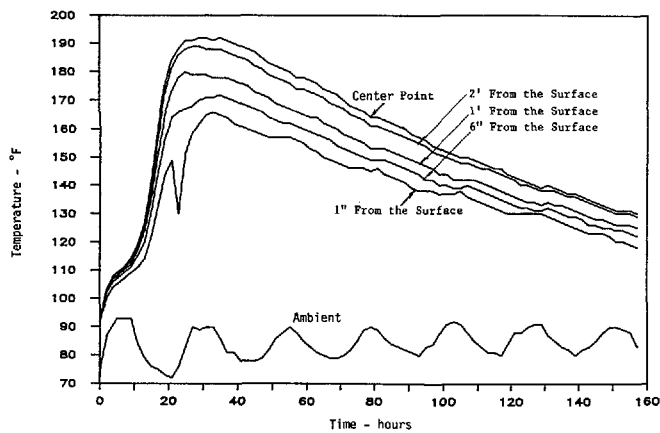


Fig. 5 — History of temperature changes in Column I along the horizontal cross section.

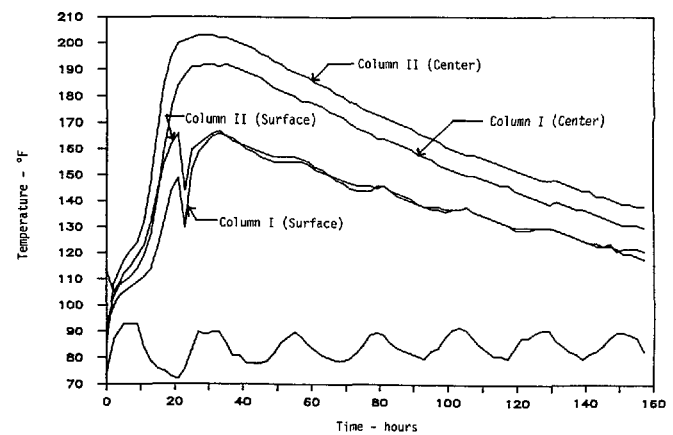


Fig. 6 — Differential thermal curves (time vs. temperature) for Column I and Column II.

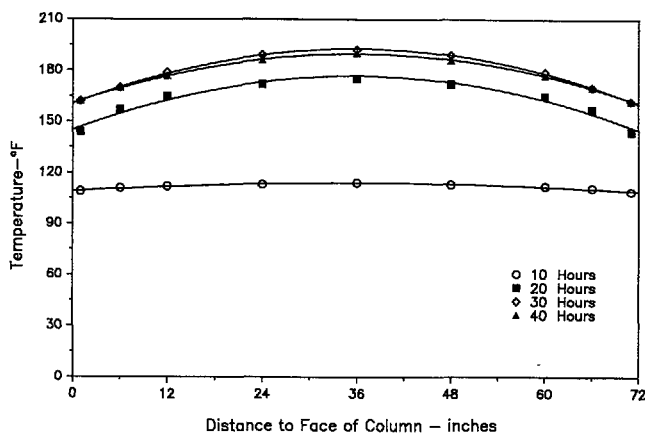


Fig. 7 — Temperature distribution curves along the cross section of Column I.

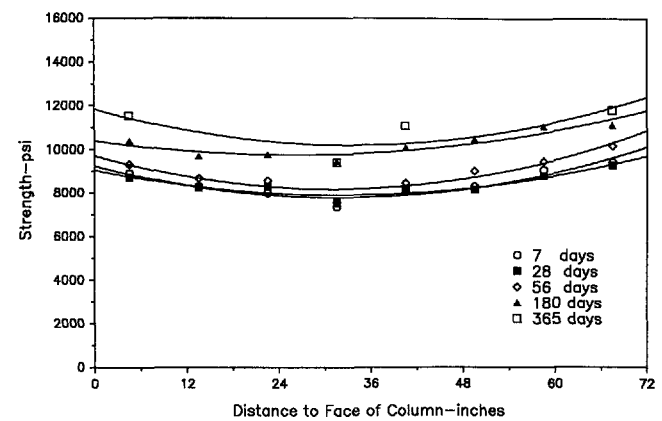


Fig. 8 — Core strength distribution curves along the cross section of Column I.

crete strength varies in the vertical direction due to water gain effect, and that the concrete placed near the top is weaker than the concrete near the bottom of a cast. However, for high-strength concrete, the core strength-age relationship presented in Fig. 3 and 4 appears to be a uniform strength distribution. The results indicated no significant decrease in strength for cores extracted from the upper portion of the mass concrete (i.e. the 28-day and the 180-day core specimens). Also, there was no significant increase in strength for cores extracted from the lower portion of the mass concrete (i.e. the 7-day and the 56-day core specimens). Therefore, the results showed that the strength increased uniformly, although the cores were extracted from various locations in the concrete at different ages. The bleeding and particle segregation that usually occur in conventional concrete

do not seriously develop in the casting of high-strength concrete.

Temperature variations and core strength

Fig. 5 provides a history of temperature changes in the high-strength concrete. The family of curves represents the temperature profile at various locations in Column I along the horizontal center line. The peak temperature at the center of the mass concrete reached 191 F (88 C) in 30 hours. There was a drop in surface temperature at the time of form removal, but the temperature rose again immediately after the thermal blanket was wrapped around the column to reduce the temperature differential near the surface. For Column II, the thermal curves showed the same pattern as Column I except that the profile was higher due to a higher original placement temperature (Fig. 6). The peak temperature at the center of

Column II reached 203 F (95 C).

Fig. 7 shows a temperature distribution at early ages along the 72 in. (183 cm) wide cross section of Column I. The temperature in the middle portion of the column was significantly higher than that near the surface in the first 20 to 40 hours; it took more than 7 days to reach a temperature equilibrium in the high-strength concrete.

Fig. 8 illustrates the core strength distribution along the cross section of Column I. The core strength at the center section was significantly lower than that near the surface during the first 3 months; it took more than a year to develop a uniform strength in the mass concrete.

It appeared that the high temperature in the middle section of the column during the hydration period delayed the chemical reaction between the water and cementitious particles. In the meantime, the columns were sealed with a membrane.

pound and were covered with a thermal blanket; there was no additional moisture available for hydration. Thus, a relatively low core strength was obtained in the central area of the mass concrete.

The results also indicated that the placement temperature, ranging from 92 F (33 C) to 103 F (39 C), had little or no effect on the strength gain of concrete cylinders and cores. Fig. 3 and 4 show that the moist-air cured and field-cured cylinders as well as core specimens from Columns I and II had identical strength development regardless of the placement temperatures.

Conclusions

Based on the test results from this investigation, the following conclusions can be drawn concerning high-strength concrete:

- The in-situ strength as determined using drilled cores is less than that of moist-air cured and field-cured cylinders tested at the same age. The core strength approaches field-cured cylinder strength after 1 year, but it is only equal to 28-day moist-air cured cylinder strength.
- At any given age, core strength is about 75 percent of moist-air cured cylinder strength and about 85 percent of field-cured cylinder strength. The field-cured cylinder strength is approximately 90 percent of the moist-air cured cylinder strength.
- Although not shown here, the data demonstrates that, for a concrete column with a large cross-sectional area, coring location has little or no effect on core strength. The in-situ strength in the vertical direction appears to be more uniform in high-strength concrete than in normal strength concrete.
- The temperature at the center of a column with a large cross-sectional area is significantly higher than near the surface. Thus, a thermal isolation cover is needed immediately after form removal in order to reduce the large temperature differential between the surface and the middle section of the mass concrete.
- For a concrete column with a large cross-sectional area, the core strength at the center is significantly lower than that near the surface. It took about a year to de-

velop uniform strength across a section of the mass concrete.

- Strength tests done on standard, moist-air cured cylinders are useful for quality control purposes and for checking the acceptability of the concrete as it is produced. However, the research reported here demonstrates that the standard tests do not provide a quantitative measurement of concrete's in-place load-carrying capacity.

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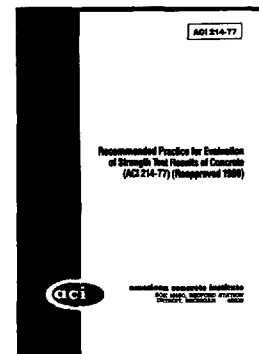
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Recommended Practice for Evaluation of Strength Test Results of Concrete 214-77(89)



Statistical methods provide valuable tools for assessing results of strength tests, and such information is also of value in refining design criteria and specifications. Discusses briefly the numerous variations that occur in the strength of concrete and presents statistical methods useful in interpreting these variations.

ACI Committee 214

Order Code: 214-77(89). C051



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High-Performance Concrete Demystified

by Pierre-Claude Aitcin and Adam Neville

For almost a century, structural concrete has been routinely produced with a 28-day strength in the range of 20 to 30 MPa (2900 to 4400 psi), or even up to 35 MPa (5100 psi). Occasionally, when special circumstances required it, higher strengths were obtained and such concrete was described as high-strength concrete. Thirty years ago the high-strength label was applied to concrete with a strength above 40 MPa (5800 psi) or thereabouts; more recently the threshold rose to 50 or 60 MPa (7300 to 8700 psi). Such concretes were not routinely produced but they were thought to be unremarkable. However, in the last 15 years, concretes of very much higher strength have entered the field of construction of high-rise buildings and bridges: 90, 100, 110 MPa (13,000, 14,500, 16,000 psi), with an occasional 120 MPa (17,000 psi). All of these strengths have been achieved consistently and on a routine basis.

This type of concrete is perceived by *some* engineers, owners and specifiers as a material fundamentally different from the concrete we have all grown up with. They suspect that there is some mystery about the manufacture of such concrete, and possibly even a secret recipe. It is the purpose of this article to destroy this myth and to show that high-strength concrete is just concrete.

What is high-performance concrete

First of all, we have to explain the apparent inconsistency between the term high-strength concrete used in the above paragraphs and the term high-performance concrete used in the title. It was natural, and correct, to refer to concrete that had a higher strength than usual as high-strength concrete. However, in practical applications of this type of concrete, the emphasis has in many cases gradually shifted from the compressive strength to other properties of the material, such as a high modulus of elasticity, high density, low permeability, and resistance to some forms of attack. It is therefore logical to describe such concrete by the more widely embracing term high-performance concrete (HPC). We propose, therefore, to use the term HPC even though the debate on terminology is still continuing.

The benefits of using HPC are many, but they are of value only when they can be exploited. To-date, exploitation has been mainly in high-rise buildings, in bridges, and in structures under severe exposure conditions. There are many potential applications of HPC, but there is no doubt that in many locations the use of HPC is precluded by its unavailability. At the same time, the unavailability is said to be a reflection of the lack of demand. It is just possible that both the designer

and the supplier are reluctant to depart from the familiar, and who can blame them.

Conversely, nothing succeeds like success: in those areas where there are designers keen to exploit HPC, there is always a supplier or two who deliver HPC to the given specification. This availability of HPC, in turn, encourages other designers to use it, and so its use is spreading.

The earlier reference to the reluctance on the part of some people to use HPC is not a criticism but rather recognition of the fact that HPC is considered to be something special or unusual, and certainly different.

What is different about HPC

We would like to start by quoting what we said, as far back as 1963, about the difference between bad concrete and good concrete:¹

"Bad concrete — often a substance of the consistence of soup, hardening into a honeycombed, non-homogeneous mass — is made simply by mixing cement, aggregate, and water. Surprisingly, the ingredients of good concrete are exactly the same, and it is only the know-how, often without additional cost or labour, that is responsible for the difference."

Ignoring the outdated reference to the consistency of concrete, we can make a parallel statement about the difference between normal strength concrete (NSC) and HPC: the know-how is the vital factor. The ingredients of both types of concrete are the same, namely, portland cement, aggregate, water, and admixture. To be more scrupulous about the ingredients, we should say that HPC invariably contains a high-range water reducer (or superplasticizer), while NSC does so only sometimes. As far as other ingredients are concerned, such as retarders, fly ash, blastfurnace slag and silica fume, they may or may not be present in either type of concrete.

The know-how necessary to produce HPC consists of a specific knowledge of the properties of the ingredients and of their interaction. The crucial outcome of the know-how is an extremely low water-cement ratio *w/c* coupled with a satisfactory workability at the time of compaction.

The necessity of using a superplasticizer in the manufacture of HPC merits brief explanation. Without a superplasticizer, even with ordinary water-reducing admixtures, the water content of the mixture cannot be reduced very far as this would result in an unworkable mixture. At the same time, the cement content cannot be raised excessively, not only because of cost, but also because a high cement content may lead to thermal problems. The combination of an upper limit on the content of cement and a lower limit on the content of water means that, without a superplasticizer, the *w/c* cannot be reduced below a value of about 0.4.

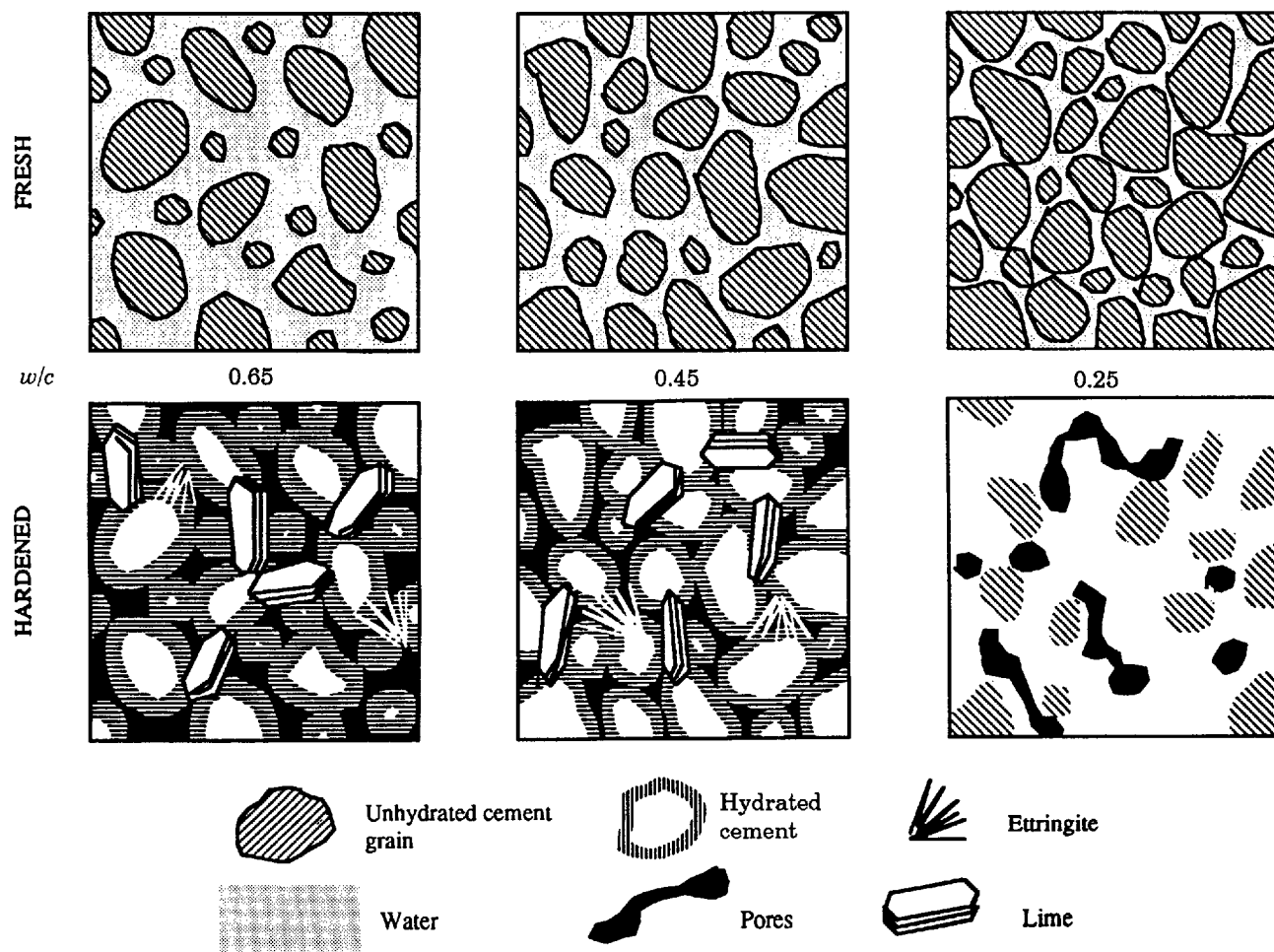


Fig. 1 — Composition of fresh and hardened cement paste at maximum hydration at various w/c values.

The constraint of workability of the mixture arises, in broad terms, from the tendency of cement grains to flocculate and thus to hold water and offer shear resistance during compaction. Without going into the details of electrical surface charges and the like, we can say that the superplasticizer deflocculates the cement particles and thus fluidifies the mixture so that a very low water content is sufficient for an adequate workability. In consequence, it is possible to obtain a mixture with a slump of 180 to 200 mm (7 to 8 in.) at a w/c in the range of 0.2 to 0.3 (based on the free water in the mixture). These values correspond to a water content of 125 to 135 L/m³ (25 to 27 gal/yd³) of concrete, as against 180 to 200 L/m³ (36 to 40 gal/yd³) in non-air-entrained NSC with a slump of 100 to 120 mm (4 to 5 in.). *This is the apparent secret of making HPC.*

The discussion so far has made it plain that, while the ingredients of HPC are substantially the same as those of NSC, their proportions are different. This is especially true of the water content of the mixture coupled with a large dosage of superplasticizer in HPC. Typically, 5 to 15 L/m³ (1 to 3 gal/yd³) of superplasticizer can effectively replace 45 to 75 L/m³ (9 to 15 gal/yd³) of water.

This drastic reduction in mixing water results in a reduced distance between the cement particles. In consequence, a much denser cement matrix than in NSC is achieved, the products of hydration of cement coalescing rapidly. By virtue of this high-density matrix, in addition to the chemical bonds

created by the hydrates (which exist also in NSC), a very high compressive strength can be achieved. Since mixing water combines chemically or physically with cement and is lost by self-desiccation, the resulting hydrated cement paste in HPC has a very low porosity.

This very low porosity of the hydrated cement paste in HPC contrasts with NSC, in which uncombined mixing water creates an open network of pores that reduces the density of the matrix and thus leads to a lower compressive strength than is the case with HPC. The situation is illustrated in Fig. 1, which also shows that in HPC a significant proportion of the original cement particles remains permanently unhydrated. We may add that, in fundamental terms, the strength of concrete is a function of the total void content of the material, provided the bonds of cohesion and adhesion are strong enough; the degree of hydration of cement as such is not important, the unhydrated part of cement being simply a well-bonded, albeit expensive, aggregate.²

Optimum water-cement ratio

The preceding section leads to a number of questions:

- Is there a minimum w/c ?
- Why use more cement than can possibly become hydrated?
- Is there an optimum w/c for maximizing the compressive strength?

It has been shown in the preceding section that the compressive strength of HPC depends not only on the w/c but also