State-of-the-Art Report on High-Strength Concrete

Reported by ACI Committee 363

Henry G. Russell
Chairman

Arthur R. Anderson
Jack O. Banning
Irwin G. Cantor*
Ramon L. Carraquillo*
James E. Cook
Gregory C. Frantz
Weston T. Hester

Anthony N. Kojundic
Brian R. Mastin*
William C. Moore
Arthur H. Nilson*
William F. Perenchio
Francis J. Principe

Kenneth L. Saucier*
Surendra P. Shah*
J. Craig Williams*
John Wolsiefer, Sr.
J. Francis Young
Paul Zia

Members responsible for individual chapters

ACI Committee 363 Members Balloting 1992 Revisions

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Judith A. Castello
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Ava Szyupa
Dean J. White, II
J. Craig Williams
John T. Wolsiefer
Francis J. Young
Paul Zia

†Deceased

Currently available information about high-strength concrete is summarized. Topics discussed include selection of materials, concrete mix proportioning, batching mixing, transporting placing, control procedures, concrete properties, structural design, economics, and applications. A bibliography is included.

Keywords: bibliographies; bridges (structures); buildings; conveying; economics; high-strength concretes; mechanical properties; mixing; mix proportioning; placing; quality control; raw materials; reviews; structural design.

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Although high-strength concrete is often considered a relatively new material, its development has been gradual over many years. As the development has continued, the definition of high-strength concrete has changed. In the 1950s, concrete with a compressive strength of 5000 psi (34 MPa) was considered high strength. In the 1960s, concrete with 6000 and 7500 psi (41 and 52 MPa) compressive strengths were used commercially. In the early 1970s, 9000 psi (62 MPa) concrete was being produced. More recently, compressive strengths approaching 20,000 psi (138 MPa) have been used in cast-in-place buildings.

For many years, concrete with compressive strength in excess of 6000 psi (41 MPa) was available at only a few locations. However, in recent years, the applications of high-strength concrete have increased, and high-strength concrete has now been used in many parts of the world. The growth has been possible as a result of recent developments in material technology and a demand for higher-strength concrete. The construction of Chicago’s Water Tower Place and 311 South Wacker Drive concrete buildings would not have been possible without the development of high-strength concrete. The use of concrete superstructures in long span cable-stayed bridges such as East Huntington, W.V., bridge over the Ohio River would not have taken place without the availability of high-strength concrete.

1.2-Committee objectives

Since the definition of high-strength concrete has changed over the years, the committee needed to define an applicable range of concrete strengths for its activities. The following working definition was adopted: “The immediate concern of Committee 363 shall be concretes
have specified compressive strengths for design of 6000 psi (41 MPa) or greater, but for the present time, considerations shall not include concrete made using exotic materials or techniques."

The word exotic was included in the definition so that the committee would not be concerned with concretes such as polymer-impregnated concrete, epoxy concrete, or concrete with artificial normal and heavy-weight aggregates.

Although 6000 psi (41 MPa) was selected as the lower limit, it is not intended to imply that there is a drastic change in material properties or in production techniques that occur at this compressive strength.

In reality, all changes that take place above 6000 psi (41 MPa) represent a process which starts with the lower-strength concretes and continues into high-strength concretes. Many empirical equations used to predict properties of concrete or to design structural members are based on tests using concrete with compressive strengths less than about 6000 psi (41 MPa). The availability of data for higher-strength concretes requires a reassessment of the equations to determine their applicability with higher-strength concretes. Consequently, caution should be exercised in extrapolating data from lower-strength to high-strength concretes. If necessary, tests should then be made to develop data for the materials or applications in question.

The committee also recognized that the definition of high-strength concrete varies on a geographical basis. In regions where concrete with a compressive strength of 9000 psi (62 MPa) is already being produced commercially, high-strength concrete might be in the range of 12,000 to 15,000 psi (83 to 103 MPa) compressive strength. However, in regions where the upper limit on commercially available material is currently 5000 psi (34 MPa) concrete, 9000 psi (62 MPa) concrete is considered high strength. The committee recognized that material selection, concrete mix proportioning, batching, mixing, transporting, placing, and control procedures are applicable across a wide range of concrete strengths. However, the committee felt that material properties and structural design considerations given in this report should be concerned with concretes having the highest compressive strengths. The committee has tried to cover both aspects in compiling this state-of-the-art report.

CHAPTER 2-SELECTION OF MATERIALS

2.1-Introduction

The production of high-strength concrete that consistently meets requirements for workability and strength development places more stringent requirements on material selection than for lower-strength concretes. Quality materials are needed and specifications require enforcement. High-strength concrete has been produced using a wide range of quality materials based on the results of trial mixtures. This chapter cites the state of knowledge regarding material selection and provides a baseline for the subsequent discussion of mix proportions in Chapter 3.

2.2-Cements

The choice of portland cement for high-strength concrete is extremely important. Unless high initial strength is the objective, such as in prestressed concrete, there is no need to use a Type III cement. Furthermore, within a given cement type, different brands will have different strength development characteristics because of the variations in compound composition and fineness that are permitted by ASTM C 150.

Initially, silo test certificates should be obtained from potential suppliers for the previous 6 to 12 months. Not only will this give an indication of strength characteristics from the ASTM C 109 mortar cube test, but also, more importantly, it will provide an indication of cement uniformity. The cement supplier should be required to report uniformity in accordance with ASTM C 917. If the tricalcium silicate content varies by more than 4 percent, the ignition loss by more than 0.5 percent, or the fineness by more than 375 cm²/g (Blaine), then problems in maintaining a uniform high strength may result. Sulfate (SO₃) levels should be maintained at optimum with variations limited to ± 0.20 percent.

Although mortar cube tests can give a good indication of potential strength, tests should be run on trial batches. These should contain the materials to be used in the job and be prepared at the proposed slump, with strengths determined at 7, 28, 56, and 91 days. The effect of cement characteristics on water demand is more noticeable in high-strength concretes because of the higher cement contents.

High cement contents can be expected to result in a high temperature rise within the concrete. For example,
the temperature in the 4 ft (1.2 m) square columns used in Water Tower Place which contained 846 lb cement/yd$^3$ (502 kg/m$^3$), rose to 150 F (66 C) from 75 F (24 C) during hydration. The heat was dissipated within 6 days without harmful effects. However, when the temperature rise is expected to be a problem, a Type II low-heat-of-hydration cement can be used, provided it meets the strength-producing requirements.

A further consideration is the optimization of the cement-admixture system. The exact effect of a water-reducing agent on water requirement, for example, will depend on the cement characteristics. Strength development will depend on both cement characteristics and cement content.

### 2.3-Chemical admixtures

#### 2.3.1 General

Admixtures are widely used in the production of high-strength concretes. These materials include air-entraining agents and chemical and mineral admixtures. Air-entraining agents are generally surfactants that will develop an air-void system appropriate for durability enhancement. Chemical admixtures are generally produced using lignosulfonates, hydroxylated carboxylic acids, carbohydrates, melamine and naphthalene condensates, and organic and inorganic accelerators in various formulations. Selection of type, brand, and dosage rate of all admixtures should be based on performance with the other materials being considered or selected for use on the project. Significant increases in compressive strength, control of rate of hardening, accelerated strength gain, improved workability, and durability are contributions that can be expected from the admixture or admixtures chosen. Reliable performance on previous work should be considered during the selection process.

#### 2.3.2 Air-entraining admixtures (ASTM C 260)

The use of air entrainment is recommended to enhance durability when concrete will be subjected to freezing and thawing while wet. As compressive strengths increase and water-cement ratios decrease, air-void parameters improve and entrained air percentages can be set at the lower limits of the acceptable range as given in ACI 201. Entrained air has the effect of reducing strength, particularly in high-strength mixtures, and for that reason it has been used only where there is a concern for durability. See also Section 5.12.

#### 2.3.3 Retarders (ASTM C 494, Types B and D)

High-strength concrete mix designs incorporate high cement factors that are not common to normal commercial concrete. A retarder is frequently beneficial in controlling early hydration. The addition of water to retard the mixture will result in marked strength reduction. Further, structural design frequently requires heavy reinforcing steel and complicated forming with attendant difficult placement of the concrete. A retarder can control the rate of hardening in the forms to eliminate cold joints and provide more flexibility in placement schedules. Projects have used retarders successfully by initially designing mixtures with sufficient retarder dosage to give the desirable rate of hardening under the anticipated temperature conditions.

Since retarders frequently provide an increase in strength that will be proportional to the dosage rate, mixtures can be designed at different doses if it is expected that significantly different rates will be used. However, there is usually an offsetting effect that minimizes the variations in strengths due to temperature. As temperature increases, later age strengths will decline; however, an increase in retarder dosage to control the rate of hardening will provide some mitigation of the temperature-induced reduction. Conversely, dosages should be decreased as temperatures decline.

While providing initial retardation, strengths at 24 hours and later are usually increased by normal dosages. Extended retardation or cool temperatures may affect early (24-hour) strengths adversely.

#### 2.3.4 Normal-setting water reducers (ASTM C 494, Type A)

Normal-setting ASTM C 494 Type A normal-setting ASTM C 494 Type A conventional water-reducing admixtures will provide strength increases without altering rates of hardening. Their selection should be based on strength performance. Increases in dosage above the normal amounts will generally increase strengths, but may extend setting times. When admixtures are used in this fashion to provide retardation, a benefit in strength performance sometimes results.

#### 2.3.5 High-range water reducers

A HRWR in high-strength concrete may serve the purpose of increasing strength at the slump or increasing slump. The method of addition should distribute the admixture throughout the concrete. Adequate mixing is critical to uniform performance. Supervision is important to the successful use of a HRWR. The use of superplasticizers is discussed further in ACI SP-68.

#### 2.3.6 Accelerators (ASTM C 494, Types C and E)

Accelerators are not normally used in high-strength concrete unless early form removal is critical. High-strength concrete mixtures can provide strengths adequate for vertical form removal on walls and columns at an early age. Accelerators used to increase the rate of hardening will normally be counterproductive in long-term strength development.

#### 2.3.7 Admixture combinations

Combinations of high-range water reducers with normal-setting water reducers or retarders have become common to achieve optimum performance at lowest cost. Improvements in strength gain and control of setting times and workability are possible with optimized combinations. In certain circumstances, combinations of normal-setting or retarding
water-reducing admixtures plus an accelerating admixture have also been found to be useful.

When using a combination of admixtures, they should be dispensed individually in a manner approved by the manufacturer(s). Air-entraining admixtures should, if used, be dispensed separately from water-reducing admixtures.

2.4-Mineral admixtures and slag cement

Finely divided mineral admixtures, consisting mainly of fly ash and silica fume, and slag cement have been widely used in high-strength concrete.

2.4.1 Fly ash- Fly ash for high-strength concrete is classified into two classes. Class F fly ash is normally produced from burning anthracite or bituminous coal and has pozzolanic properties, but little or no cementitious properties. Class C fly ash is normally produced from burning lignite or subbituminous coal, and in addition to having pozzolanic properties, has some autogenous cementitious properties. In general, Class F fly ash is available in the eastern United States and Canada, and Class C fly ash is available in the western United States and Canada.

Specifications for fly ash are covered in ASTM C 618. Methods for sampling and testing are found in ASTM C 311. Variations in physical or chemical properties of mineral admixtures, although within the tolerances of these specifications, may cause appreciable variations in properties of high-strength concrete. Such variations can be minimized by appropriate testing of shipments and increasing the frequency of sampling. ACI 212.2R provides guidelines for the use of admixtures in concrete. It is extremely important that mineral admixtures be tested for acceptance and uniformity and carefully investigated for strength-producing properties and compatibility with the other materials in the high-strength concrete mixture before they are used in the work.

2.4.2 Silica fume - Silica fume and admixtures containing silica fume have been used in high-strength concretes for structural purposes and for surface applications and as repair materials in situations where abrasion resistance and low permeability are advantageous. Silica fume is a by-product resulting from the reduction of high-purity quartz with coal in electric arc furnaces in the production of silicon and ferrosilicon alloys. The fume, which has a high content of amorphous silicon dioxide and consists of very fine spherical particles, is collected from the gases escaping from the furnaces. Silica fume consists of very fine vitreous particles with a surface area on the order of 20,000 m²/kg when measured by nitrogen adsorption techniques. The particle-size distribution of a typical silica fume shows most par-titles to be smaller than one micrometer (1 µm) with an average diameter of about 0.1 µm, which is approximately 100 times smaller than the average cement particle. The specific gravity of silica fume is typically 2.2, but may be as high as 2.5. The bulk density as collected is 10 to 20 lb/ft³ (160 to 320 kg/m³); however, it is also available in densified or slurry forms for commercial application.

Silica fume, because of its extreme fineness and high silica content, is a highly effective pozzolanic material. The silica fume reacts pozzolantically with the lime during the hydration of cement to form the stable cementitious compound calcium silicate hydrate (CSH). The availability of high-range water-reducing admixtures has facilitated the use of silica fume as part of the cementing material in concrete to produce high-strength concretes. Normal silica fume content ranges from 5 to 15 percent of portland cement content.

The use of silica fume to produce high-strength concrete increased dramatically in the 1980s. Both laboratory and field experience indicate that concrete incorporating silica fume has an increased tendency to develop plastic shrinkage cracks. Thus, it is necessary to quickly cover the surfaces of freshly placed silica-fume concrete to prevent rapid water evaporation. Since it is a relatively new material to the concrete industry in the United States, the user is referred to several recent symposia and publications for additional information on silica fume.

2.4.3 Slag cement - Ground slag cement is produced only in certain areas of the United States and Canada. Specifications for ground granulated blast furnace slag are given in ASTM C 989. The classes of portland blast furnace slag cement are covered in ASTM C 595. Slag appropriate for concrete is a nonmetallic product that is developed in a molten condition simultaneously with iron in a blast furnace. When properly quenched and processed, slag will act hydraulically in concrete as a partial replacement for portland cement. Slag can be interground with cement or used as an additional cement at the batching facility. Blast furnace slag essentially consists of silicates and alumino-silicates of calcium and other bases. Research using ground slag shows much promise for its use in high-strength concrete.

2.4.4 Evaluation and selection - Mineral admixtures and slag cement, like any material in a high-strength concrete mixture, should be evaluated using laboratory trial batches to establish the optimum desirable qualities. Materials representative of those that will be employed later in the actual construction should be used. Particular care should be taken to insure that the mineral admixture comes from bulk supplies and that they are typical. Generally, several trial batches are made using varying cement factors and admixture dosages to establish curves which can be used to select the amount of cement and admixture required to achieve the desired results.

When fly ash is to be used, the minimum requirement is that it comply with ASTM C 618. Although this specification permits a higher loss on ignition, an ignition loss of 3 percent or less is desirable. High fineness, uniformity or production, high pozzolanic activity, and compatibility with other mixture ingredients are items of primary importance.
2.5. Aggregates

2.5.1 General - Both fine and coarse aggregates used for high-strength concrete should, as a minimum, meet the requirements of ASTM C 33; however, the following exceptions may be beneficial.

2.5.2 - Grading

2.5.2.1 Fine aggregate - Fine aggregates with a rounded particle shape and smooth texture have been found to require less mixing water in concrete and for this reason are preferable in high-strength concrete. The optimum gradation of fine aggregate for high-strength concrete is determined more by its effect on water requirement than on physical packing. One report stated that a sand with a fineness modulus (PM) below 2.5 gave the concrete a sticky consistency, making it difficult to compact. Sand with an FM of about 3.0 gave the best workability and compressive strength.

High-strength concretes typically contain such high contents of fine cementitious materials that the grading of the aggregates used is relatively unimportant compared to conventional concrete. However, it is sometimes helpful to increase the fineness modulus. A National Crushed Stone Association report made several recommendations in the interest of reducing the water requirement. The amounts passing the No. 50 and 100 sieves should be kept low, but still within the requirements of ASTM C 33, and mica or clay contaminants should be avoided. Another investigation found that the sand gradation had no significant effect on early strengths but that “at later ages and consequently higher levels of strength, the gap-graded sand mixes exhibited lower strengths than the standard mixes.”

2.5.2.2 Coarse aggregate

Many studies have shown that for optimum compressive strength with high cement content and low water-cement ratios the maximum size of coarse aggregate should be kept to a minimum, at ½ in. (12.7 mm) or ¾ in. (9.5 mm). Maximum sizes of ¾ in. (19.0 mm) and 1 in. (25.4 mm) also have been used successfully. Cordon and Gillespie felt that the strength increases were caused by the reduction in average bond stress due to the increased surface area of the individual aggregate. Alexander found that the bond to a 3 in. (76 mm) aggregate particle was only about 1/10 of that to a ½-in. (13 mm) particle. He also stated that except for very good or very bad aggregates the bond strength was about 50 to 60 percent of the paste strength at 7 days.

Smaller aggregate sizes are also considered to produce higher concrete strengths because of less severe concentrations of stress around the particles, which are caused by differences between the elastic moduli of the paste and the aggregate.

Many studies have shown that crushed stone produces higher strengths than rounded gravel. The most likely reason for this is the greater mechanical bond which can develop with angular particles. However, accentuated angularity is to be avoided because of the attendant high water requirement and reduced workability. The ideal aggregate should be clean, cubical, angular, 100 percent crushed aggregate with a minimum of flat and elongated particles.

Because, as stated earlier, bond strength is the limiting factor in the development of high-strength concrete, the mineralogy of the aggregates should be such as to promote chemical bonding. Some work has been done with artificial material such as portland and aluminous cement clinkers and selected slags. The long-term stability of the clinkers is in question, however. Harris states that Moorehead measured a potential silica-lime bond of at least 28,000 psi (193 M Pa). Presumably many siliceous minerals would prove to have good bonding potential with portland cement. This would appear to be a promising area for further research.

2.5.3 Absorption - Curing is extremely important in the production of high-strength concrete. To produce a cement paste with as high a solids content as possible, the concrete must contain the absolute minimum mix water. However, after the concrete is in place and the paste structure is established, water should be freely available, especially during the early stages of hydration. During this period, a great deal of water combines with the cement. All of this water loses approximately ¼ of its volume after the chemical reactions are completed. This creates a small vacuum that is capable of pulling water short distances into the concrete which, at this time, is still relatively permeable. Any extra water which can enter the structure will increase the ultimate amount of hydration and, therefore the percent of solids per unit volume of paste, thereby increasing its strength.

If the aggregates are capable of absorbing a moderate amount of water, they can act as tiny curing-water reservoirs distributed throughout the concrete, thereby providing the added curing water which is beneficial to these low water-cement ratio pastes.

2.5.4 Intrinsic aggregate strength - It would seem obvious that high-strength concrete would require high-strength aggregates and, to some extent, this is true. However, several investigators have found that, for some aggregates, a point is reached beyond which further increases in cement content produce no increase in the compressive strength of the concrete. This apparently is not due to having fully developed the compressive strength of the concrete but to having reached the limit of the bonding potential of that cement-aggregate combination.

2.6 Water

The requirements for water quality for high-strength concrete are no more stringent than those for conventional concrete. Usually, water for concrete is specified to be of potable quality. This is certainly conservative but usually does not constitute a problem since most concrete is produced near a municipal water supply. However, cases may be encountered where water of a lower quality must be used. In such cases, test concrete should be made with the water and compared with concrete made...
with distilled water, or it may be more convenient to make ASTM C 109 mortar cubes. In either case, specimens should be tested in compression at 7 and 28 days. If those made with the water in question are at least equal to 90 percent of the compressive strength of the specimens made with distilled water, the water then can be considered acceptable to U.S. Army Corps of Engineers’ requirements and ASTM C 94.

For more detailed information on specific contaminants refer to the literature in References 2.27, 2.28, and 2.29. Test methods for water for special situations are given in AASHTO T26.

2.7-Cited references
(See also Chapter 10-References)


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CHAPTER 3 - CONCRETE MIX PROPORTIONS

3.1—Introduction

Concrete mix proportions for high-strength concrete have varied widely depending upon many factors. The strength level required, test age, material characteristics, and type of application have influenced mix proportions. In addition, economics, structural requirements, manufacturing practicality, anticipated curing environment, and even the time of year have affected the selection of mix proportions. Much information on proportioning concrete mixtures is available in ACI 211.1 and ACI SP-46. 3.1 Included in ACI publication SP-46 is the paper “Proportioning and Controlling High Strength Concrete” (SP-46-9).

High-strength concrete mix proportioning is a more critical process than the design of normal strength concrete mixtures. Usually, specially selected pozzolanic and chemical admixtures are employed, and the attainment of a low water-cementitious ratio is considered essential. Many trial batches are often required to generate the data that enables the researcher to identify optimum mix proportions.

3.2-Strength required

3.2.1 ACI 318- The ACI Building Code Requirements for Reinforced Concrete (ACI 318) describes concrete strength requirements. Normally the concrete has been proportioned in such a manner that the mean average of compressive strength test results has exceeded the specified strength $f'_c$ by an amount sufficiently high to minimize the relative frequency of test results below the specified strength value.

An average value can be calculated for any set of measurement data. The amount that individual test values deviate from the average is usually quantified by calculation of the standard deviation. Calculation of standard deviation on concrete test histories can be a valuable aid in predicting future test result variability.

Many factors can influence the variability of the test results, including the individual materials, plants, contractors, inspection agencies, and environmental conditions. All factors which will affect the variability of strengths and strength measurements should be considered when selecting mix proportions and when establishing the standard deviation acceptable for strength results. Materials and proportions used for qualifying the mixture should not be more closely controlled than is planned for the proposed work. Kennedy and Price have identified factors which contribute to the variability of measured compressive strengths of concretes in lower strength ranges. 3.3, 3.4

Hester identified sources of measured strength variations in high-strength concretes. 3.5 High-strength concrete is recognized to be more difficult to test accurately than normal strength concretes. Testing difficulties may contribute to lower measured values or higher variability.

A high variance in test results will dictate a higher required average strength. If variability is predicted to be relatively low, but proves to be higher, the frequency of test results below the specified strength may be unacceptably high. Therefore, when selecting a target standard deviation the concrete producer should submit the most appropriate test record. 3.6 A higher required average strength may be difficult or impossible to attain when producing high-strength concretes because mix proportions may already be optimized.

ACI 318 recognizes that some test results are likely to be lower than the specified strength. The most common design approach has been to limit the frequency of tests allowed to fall below the specified strength. The concrete has been judged acceptable if the following requirements are met:

a) The average of all sets of three consecutive strength test results shall equal or exceed the required $f'_c$.

b) No individual strength test (average of two cylinders) shall fall below $f'_c$ by more than 500 psi (3.4 MPa).

However, some designers have specified higher or lower overdesign strengths than called for in ACI 318 regardless of established performance.

Schmidt and Hoffman 3.7 report that they do not automatically order removal of concrete which is represented by cylinders 500 psi (3.4 MPa) below specified strength but do order adjustment of the mixture and correction of the deficiency. This is because the ACI 318 Section 4.7.4 was established for concretes with strengths in the range of 3000 to 5000 psi (21 to 34 MPa). High-strength concretes continue to gain considerable strengths above and beyond design requirements with the passage of time, more than lower-strength concretes. 3.7 While the percentage gain of compressive strength of high-strength concretes from 7 days to 90 days may be equal to or lower than concretes in lower strength ranges, the order of magnitude of strength gain expressed in psi is actually much higher. For example, a mixture which averages 2500 psi (17.2 MPa) in 7 days may average 4200 psi (29 MPa) in 90 days. It would have gained strength equal to 68 percent of the 7-day strength, or 1700 psi (11.7 MPa) at the age of 90 days. A mixture averaging 7300 psi (50.3
MPa) in 7 days could average 10,000 psi (69 MPa) in 90 days. That would be an increase of only 37 percent, but it would have gained 2700 psi (18.6 MPa), a full 1000 psi (6.9 MPa) higher total gain than the lower-strength mixture.

ACI 318 allows mix designs to be proportioned based on field experience or by laboratory trial batches. When the concrete producer chooses to select high-strength concrete mix proportions based upon laboratory trial batches, confirming tests results from concretes placed in the field should also be established.

3.2.2 ACI 214-- Once sufficient test data have been generated from the job, a reevaluation of mix proportions using “Recommended Practice for Evaluation of Compression Test Results of Concrete (ACI 214)” may be appropriate. Analyses affecting reproportioning of mixtures based upon test histories are described in Sections 4.8.1 and 4.8.2.

3.2.3 Other Strength Requirements--In some situations, considerations other than compressive strength may influence mix proportions. Detailed discussion of material properties including flexural and tensile strengths is given in Chapter 5.

3.3- Test age

The selection of mix proportions can be influenced by the testing age. This testing age has varied depending upon the construction requirements. Most often the testing age has been thought to be the age at which the acceptance criteria are established, for example at 28 days. Testing, however, has been conducted prior to the age of acceptance testing, or after that age, depending upon the type of information required.

3.3.1 Early Age-- Prestressed concrete operations may require high strengths in 12 to 24 hours. Special applications for early use of machinery foundations, pavement traffic lanes, or slip formed concrete have required high strengths at early ages. Post-tensioned concrete is often stressed at ages of approximately 3 days and requires relatively high strengths. Generally concretes which develop high later-age strengths will also produce high early-age strengths. However, the optimum materials selected, and therefore the mix proportions, may vary for different test ages. For example, Type III cement and no fly ash have been used in a high early-strength design, compared to Type I or II cement and fly ash for a later-age design. Early-age strengths may be more variable due to the influence of curing temperature and the early-age characteristics of the specific cement. Therefore, anticipated mix proportions should be evaluated for a higher required average strength or a later test age.

3.3.2 Twenty-eight days- A very common test age for compressive strength of concrete has been 28 days. Performance of structures has been empirically correlated with moist-cured concrete cylinders, usually 6 x 12 in. (152 x 305 mm) prepared according to ASTM C 31 and C 192. This has produced good results for concretes with-

in lower strength ranges not requiring early strengths or early evaluation. High-strength concretes gain considerable strengths at later ages and, therefore, are evaluated at later ages when construction requirements allow the concrete more time to develop strengths before loads are imposed. Proportions, notably cementitious components, have usually been adjusted depending upon test age.

3.3.3 Later age-- High-strength concretes are frequently tested at later ages such as 56 or 90 days. High-strength concrete has been placed frequently in columns of high-rise buildings. Therefore, it has been desirable to take advantage of long-term strength gains so that efficient use of construction materials can be achieved. This has often been justified in high-rise buildings where full loadings may not occur until later ages.

In cases where later-age acceptance criteria have been specified, it may be advantageous for the concrete supplier to develop early-age or accelerated tests to predict later-age strengths. The ACI publication SP-56, Accelerated Strength Testing, provides information on accelerated testing. Of course, historical correlation data must be developed relative to the materials and proportions to be used in the work. These tests may not always accurately predict later-age strengths; however, these tests could provide an early identification of lower-strength trends before a long history of non-compliance is realized. Later-age acceptance criteria can leave suspect concrete in question for a long time.

Test cylinders have been held for testing at ages later than the specified acceptance age. In cases where the specified compressive strength was not achieved, subsequent testing of later-age or “hold” cylinders has sometimes justified the acceptance of the concrete in question.

3.3.4 Test age in relationship to curing- When selecting mix proportions, the type of curing anticipated should be considered along with the test age, especially when designing for high early strengths. Concretes gain strength as a function of maturity, which is usually defined as a function of time and curing temperature.

3.4- Water-cement ratio or water-cementitious ratio

3.4.1 Nature of water-cement ratio in high-strength concrete- The relationship between water-cement ratio and compressive strength, which has been identified in low-strength concretes, has been found to be valid for high-strength concretes also. Higher cement contents and lower water contents have produced higher strengths. Proportioning larger amounts of cement into the concrete mixture, however, has also increased the water demand of the mixture. Increases in cement beyond a certain point have not always increased compressive strengths. Other factors which may limit maximum cement contents are discussed in Section 3.5.3. When pozzolanic materials are used in concrete, a water-cement plus pozzolan ratio by weight has been considered in place of the traditional water-cement ratio by weight. Fly ash meeting requirements of ASTM C 618 with a loss on ignition of less than 3.0 percent and ASTM C 494 types
A, D, F, and G chemical admixtures have usually been used. Of course the slump of the concrete is related to the water-cementitious ratio and the total amount of water in the concrete. While 0 to 2 in. slump concrete has been produced in precast operations, special consolidation efforts are required. Specified slumps for cast-in-place concretes not containing high-range water reducers have ranged from 2½ to 4½ in. (64 to 114 mm). Field-placed nonplasticized concretes have had measured slumps averaging as high as 4½ in. (121 mm).

The use of high-range water reducers has provided lower water-cementitious ratios and higher slumps. Water-cementitious ratios by weight for high-strength concretes typically have ranged from 0.27 to 0.50. The quantity of liquid admixtures, particularly high-range water reducers, sometimes has been included in the water-cementitious ratio.

**3.4.2 Estimating compressive strength**-- The compressive strength that a concrete will develop at a given water-cementitious ratio has varied widely depending on the cement, aggregates, and admixtures employed.

Principal causes of variations in compressive strengths at a given water-cementitious ratio include the strength-producing capabilities of the cement and potential for pozzolanic reactivity of the fly ash or other pozzolan if used. Different types and brands of portland cement have produced different compressive strengths as shown in Fig. 3.1. 3,10

Specific information pertaining to the range of values of compressive strengths of cements has been published in ASTM C 917 and Peters. 3,10 Fly ashes may vary in pozzolanic activity index from 75 percent to 110 percent of the portland cement control, as defined in ASTM C 618. Proprietary pozzolans containing silica fume have been reported to have activity indexes in excess of 200 percent. 3,10 The water requirement of the particular pozzolan employed has varied and has generally increased with increasing fineness of the pozzolan. Often water requirements for fly ash concrete are lower than for portland cement. This helps to lower the water-cementitious ratio of the mixture.

Perenchio 3,10 has reported variable compressive strength results at given water-cement ratios in laboratory prepared concretes, depending on the aggregates used. In addition, these results have differed from results achieved in actual production with materials from the same area. A range of typical strengths reported at given water-cementitious ratios is represented in Fig. 3.2. Trial batches with materials actually to be used in the work have been found to be necessary. Generally, laboratory trial batches have produced strengths higher than those strengths which are achievable in production, as seen in Fig. 3.2.

**3.5-Cement content**

The cement quantity proportioned into a high-strength mixture has been determined best by the fabrication of trial batches. Common cement contents in high-strength concrete test programs range from 660 to 940 lb per yd³ (392 to 557 kg/m³). 3,10,11,16 In evaluating optimum cement contents, trial mixes usually are proportioned to equal consistencies, allowing the water content to vary according to the water demand of the mixture.

**3.5.1 Strength**--For any given set of materials in a concrete mixture, there may be a cement content that produces maximum concrete strength. The maximum strength may not always be increased by the use of...
cement added to the mixture beyond this optimum cement content. The strength for any given cement content will vary with the water demand of the mixture and the strength-producing characteristics of that particular cement as shown in Fig. 3.1. The “Standard Method of Evaluation of Cement Strength Uniformity from a Single Source” (ASTM C 917) may prove useful in considering cement mill sources. Mortar cube compressive strength data of cements at ages of up to 90 days have been evaluated when proportioning cement in high-strength mixtures.

The strength of the concrete mixture will depend upon the gel-space ratio, which is defined as the “ratio of the volume of hydrated cement paste to the sum of the volumes of the hydrated cement and of the capillary pores.” This is particularly true when air-entraining admixtures are employed. Higher cement contents in air-entrained concrete have not been found to be useful in producing strengths equivalent to, or approaching, strengths attainable with non-air-entrained concretes. Incorporation of entrained air may reduce strength at a ratio of 5 to 7 percent for each percent of air in the mix as shown in Fig. 3.4.

3.5.2 Optimization-- A principal consideration in establishing the desired cement content will be the identification of combinations of materials which will produce optimum strengths. Ideally, evaluations of each potential source of cement, fly ash, liquid admixture, and aggregate in varying concentrations would indicate the optimum cement content and optimum combination of materials. Testing costs and time requirements usually have limited the completeness of the testing programs, but particular attention has been given to evaluation of the brand of cement to be used with the class and source of pozzolan, if a pozzolan is to be used. Prior to 1977, Chicago high-strength experience was based on concretes using Class F fly ash, while other high-strength work has been done in Houston using Class C fly ash. Class C fly ash has been used in Chicago since 1977.

The strength efficiency of cement will vary for different maximum size aggregates at different strength levels. Higher cement efficiencies are achieved at high strength levels with lower maximum aggregate sizes. Fig. 3.5 illustrates this principle. For example, a maximum aggregate size of less than 3/4 in. (9.5 mm) yields the highest cement efficiency for a 7000 psi (48.3 MPa) mixture.
3.5.3 Limiting factors—There are several factors which may limit the maximum quantity of cement which may be desirable in a high-strength mixture. The strength of the concrete may decrease if cement is added above and beyond a given optimum content. The maximum desirable quantity of cement may vary considerably depending upon the efficiency of dispersing agents, such as high-range water reducers, in preventing flocculation of cement particles.

Stickiness and loss of workability will be increased as higher amounts of cement are incorporated into the mixture. Combinations of cement, pozzolans, and sand should be evaluated for the effect of cementitious content upon mixture placeability. Incorporation of an air-entraining admixture may necessitate reevaluation of the effect of the cement upon mixture workability.

The maximum temperature desired in the concrete element may limit the quantity or type of cement in the mixture. Modification of the mixture with ice, set retarders, or pozzolans may be helpful.

Cement-rich mixtures frequently have very high water demands. Therefore, it is possible that special precautions may be necessary to provide adequate curing water, so that sufficient hydration can occur. It may be preferable to reduce the amount of cement in the mixture and to rely upon more careful selection of aggregates, aggregate proportions, etc., optimizing the use of other constituents.

The amount of slump loss experienced, with attendant increase in retempering water, and the setting time of the concrete has varied depending upon the type, brand, and quantity of cement use. Lower cement contents, within limits, are desirable in order to enhance the placement capabilities of the mixture, provided that adequate strengths can be achieved.

3.6 Aggregate proportions

In the proportioning of high-strength concrete, the aggregates have been a very important consideration since they occupy the largest volume of any of the ingredients in the concrete. Usually, high-strength concretes have been produced using normal weight aggregates. Shideler and Holm have reported on lightweight high-strength structural concrete. Mather has reported on high-strength high-density concrete using lightweight aggregate.

3.6.1 Fine aggregates—In proportioning a concrete mixture, it is generally agreed that the fine aggregates or sand have considerably more impact on mix proportions than the coarse aggregates.

The fine aggregates contain a much higher surface area for a given weight than do the larger coarse aggregates. Since the surface area of all the aggregate particles must be coated with a cementitious paste, the proportion of fine to coarse can have a direct quantitative effect on paste requirements. Furthermore, the shape of these sand particles may be either spherical, subangular, or very angular. This property can alter paste requirements even though the net volume of the sand remains the same.

The gradation of the fine aggregate plays an important role in properties of the plastic as well as the hardened concrete. For example, if the sand has an overabundance of the No. 50 and No. 100 sieve sizes, the plastic workability will be improved but more paste will be needed to compensate for the increased surface area. This could result in a costlier mixture, or if the paste volume is increased by adding water, a serious loss in strength could result. It is sometimes possible, although not always practically economically, to blend sands from different sources to improve their gradation and their capacity to produce higher-strength concrete.

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

Consolidation by means of mechanical vibrators may help to overcome the effects of an undersanded mixture, and the use of power finishing equipment can help to offset the lack of trowelability.

Particle shape and surface texture of fine aggregate can have as great an effect on mixing water requirements as those of coarse aggregate. Tests made by Bloem and Gaynor show that concrete-mixing water requirements for each cubic yard of concrete change 1 gal. (3.8 L) for each change of 1 percent in the void content of the sand. Following the work by Bloem and Gaynor, the NSGA-NRMCA Joint Research Laboratory has simplified the procedure for conducting the void content test of sand and a modified gradation is now used. The new procedure is described in Reference 2.12.

3.6.2 Coarse aggregates—The optimum amount and size of coarse aggregate for a given sand will depend to a great extent on the characteristics of the sand. Most particularly it depends on the fineness modulus (FM) of the sand. This is brought out specifically in Table 3.1, which is taken from ACI 211.1. One reference suggests that the proportion of coarse aggregate shown in Table 3.1 might be increased by up to 4 percent if sands with low void contents are used. If the sand particles are very angular, then it is suggested that the amount of coarse aggregate should be decreased by up to 4 percent from the values in the table. Such adjustments in the proportion of coarse aggregate and sand have been intended to produce concretes of equivalent workability, although such changes will alter the water demand for a given slump. When more or less water is needed in a given volume of concrete, to preserve the same consistency of paste, it is also necessary to adjust the amount of cement or cementitious materials if a given water-cement ratio is to be maintained.

Another possible expedient in the proportioning of coarse aggregates for high-strength concrete is to alter
the amount of these aggregates passing certain sieve sixes from the amounts shown in ASTM C 33. This method is described in Reference 3.24 and 3.25 as a means of avoiding “particle interference,” thus permitting a greater amount of coarse aggregate and less total sand. This has helped to reduce the paste requirements or permit the use of a more viscous paste, resulting in a higher strength.

3.6.3 Proportioning aggregates-The amounts of coarse aggregate suggested in Table 3.1 (which is Table 5.3.6 of ACI 211.1) are recommended for initial proportioning. Considerations should be given to the properties of the sand (FM, angularity, etc.) which may alter the quantity of coarse aggregate. In general, the least sand consistent with necessary workability has given the best strengths for a given paste. Mechanical tools for handling and placing concrete have helped to decrease the proportion of sand needed. As previously stated, the use of the smaller sixes of coarse aggregate are generally beneficial, and crushed aggregates seem to bond best to the cementitious paste.

3.7-Proportioning with admixtures

Nearly all high-strength concretes have contained admixtures. Changes in the quantities and combinations of these admixtures affect the plastic and hardened properties of high-strength concrete. Therefore, special attention has been given to the effects of these admixtures (described in Sections 2.3 and 2.4). Careful adjustments to mix proportions have been made when changes in admixture quantities or combinations have been made. Material characteristics have varied extensively, making experimentation with the candidate materials necessary. Some of the more common adjustments are described in Sections 3.7.1 and 3.7.2.

3.7.1 Pozzolanic admixtures- Pozzolanic admixtures are often used as a cement replacement. In high-strength concretes they have been used to supplement the Portland cement from 10 to 40 percent by weight of the cement content. In those cases where a net increase in the absolute volume of the cementitious materials was experienced due to the addition of a pozzolan, a corresponding decrease in the absolute volume of the sand was usually made.

The use of fly ash has often caused a slight reduction in the water demand of the mixture, and that reduction in the volume of water (if any) has been compensated for by the addition of sand. The opposite relationship has been found to be true for other pozzolans. Silica fume, for example, dramatically increases the water demand of the mixture which has made the use of retarding and superplasticizing admixtures a requirement. Proprietary products containing silica fume include carefully balanced chemical admixtures as well.11

3.7.2 Chemical admixtures

3.7.2.1 Conventional water-reducers and retarders-The amount of these admixtures used in high-strength concrete mixtures has varied depending upon the particular admixture and application. Generally speaking, the tendency has been to use larger than normal or maximum quantities of these admixtures. Typical water reductions of 5 to 8 percent may be increased to 10 percent. Corresponding increases in sand content have been made to compensate for the loss of volume due to the reduction of water in the mixture.

3.7.3.2 Superplasticizers or high-range water-reducing admixtures-Adjustments to high-strength concrete made with high-range water reducers have been similar to those adjustments made when conventional water reducers are used. These adjustments have typically been larger due to the larger amount of water reduction, approximately 12 to 25 percent. Corresponding increases in sand content have been made to compensate for the loss of volume from reduction of water in the mixture.

Some designers have simply added high-range water reducers to existing mixtures without any adjustments to the mix proportions to improve the workability of that concrete.

Sometimes cement or cementitious content has been reduced for reasons of economy or to achieve a reduction of the heat of hydration. Usually, however, in high-strength concretes high-range water reducers are used to lower the water-cementitious ratio. These admixtures have been effective enough to both lower the water-cementitious ratio and increase the slump. Due the relatively large quantity of liquid that has been added to the mixture in the form of superplasticizing admixture, the weight of these admixtures has sometimes been included in the calculation of the water-cementitious ratio.

3.7.2.3 Air-entraining agents- Although sometimes required, air-entraining agents have been found to be very undesirable in high-strength concretes due to the dramatic decrease in compressive strength which occurs when these admixtures are used. Modifications to lower the water-cementitious ratio and adjust the yield of the concrete by reduction of sand content have been made. Larger dosage rates of air-entraining admixture have
been found to be required in high-strength concretes, especially in very rich low-slump mixtures and mixtures containing large quantities of some fly ashes.

3.7.2.4 Combinations- Most but not all high-strength concretes have contained both mineral and chemical admixtures. It has been common for these mixtures to contain combinations of chemical admixtures as well. High-range water reducers have performed better in high-strength concretes when used in combination with conventional water reducers or retarders. This is because of the reduced rate of slump loss experienced. It is not unusual for portland-pozzolan high-strength concretes to contain both a conventional and high-range water reducer.

3.8-Workability

Workability is defined in ACI 116R “Cement and Concrete Terminology” as “that property of freshly mixed concrete . . . which determines the ease and homogeneity with which it can be mixed, placed, compacted, and finished.”

3.8.1 Slump- ASTM C 143 describes a standard test method for the slump of portland cement concrete which has been used to quantify the consistency of plastic, cohesive concretes. This test method has not usually been considered applicable to ultra-low and ultra-high slump concretes. Other test methods such as the Vebe consistometer have been used with very stiff mixes and may be a better aid in proportioning some high-strength concretes.

High-strength concrete performance demands a dense, void-free mass with full contact with reinforcing steel. Slumps should reflect this need and provide a workable mixture, easy to vibrate, and mobile enough to pass through closely placed reinforcement. Normally a slump of 4 in. (102 mm) will provide the required workability; however, details of forms and reinforcing bar spacing should be considered prior to development of mix designs. Slumps of less than 3 in. (76 mm) have made special consolidation equipment and procedures a necessity.

Without uniform placement, structural integrity may be compromised High-strength mixes tend to lose slump more rapidly than lower-strength concrete. If slump is to be used as a field control, testing should be done at a prescribed time after mixing. Concrete should be discharged before the mixture becomes unworkable.

3.8.2 Placeability-- High-strength concrete, often designed with % in. (12 mm) top size aggregate and with a high cementitious content, is inherently placeable provided attention is given to optimizing the ratio of sand to coarse aggregate. Local material characteristics have a marked effect on proportions. Cement fineness and particle size distribution influence the character of the mixture. Admixtures have been found to improve the placeability of the mixture.

Placeability has been evaluated in mock-up forms prior to final approval of the mix proportions. At that time placement procedures, vibration techniques, and scheduling have been established since they greatly affect the end product and will influence the apparent placeability of the mixture.

3.8.3 Flow properties and stickiness-Slumps needed for almost any flow can be designed for the concrete; however, full attention must be given to aggregate selection and proportioning to achieve the optimum slump. Elongated aggregate particles and poorly graded coarse and fine aggregates are examples of characteristics that have affected flow and caused higher water content for placeability with attendant strength reduction.

Stickiness is inherent in high-fineness mixtures required for high strengths. Certain cements or cement-pozzolan or cement-admixture combinations have been found to cause undue stickiness that impairs flowability. The cementitious content of the mixture normally has been the minimum quantity required for strength development combined with the maximum quantity of coarse aggregate within the requirements for workability.

Mixtures that were designed properly but appear to change in character and become more sticky can be considered suspect and quickly checked for proportions, possible false setting of cement, undesirable air entrainment, or other changes. A change in the character of a high-strength mixture could be a warning sign for quality control and, while a subjective judgment, may sometimes be more important than quantitative parameters.

3.9- Trial batches

Frequently the development of a high-strength concrete program has required a large number of trial batches. In addition to laboratory trial batches, field-sized trial batches have been used to simulate typical production conditions. Care should be taken that all material samples are taken from bulk production and are typical of the materials which will be used in the work. To avoid accidental testing bias, some researchers have sequenced trial mixtures in a randomized order.

3.9.1 Laboratory trial batch investigations- Laboratory trial batches have been prepared to achieve several goals. They should be prepared according to “Standard Method of Making and Curing Concrete Test Specimens in the Laboratory” (ASTM C 192). However, whenever possible, timing, handling, and environmental conditions similar to those which are likely to be encountered in the field should be approximated.

Selection of material sources has been facilitated by comparative testing, with all variables except the candidate materials being held constant. In nearly every case, particular combinations of materials have proven to be best. By testing for optimum quantities of optimum materials, the investigator is most likely to define the best combination and proportions of materials to be used.

Once a promising mixture has been established, further laboratory trial batches may be required to quantify the characteristics of those mixtures. Strength characteristics at various test ages may be defined. Water
demand, rate of slump loss, amount of bleeding, segregation, and setting time can be evaluated. The unit weight of the mixture should be defined and has been used as a valuable quality control tool. Structural considerations such as shrinkage and elasticity may also be determined. While degrees of workability and placeability may be difficult to define, at least a subjective evaluation should be attempted.

3.9.2 Field-production trial batches. Once a desirable mixture has been formulated in the laboratory, field testing with production-sized batches is recommended. Quite often laboratory trial batches have exhibited a strength level significantly higher than that which can be reasonably achieved in production as shown in Fig. 3.3.12. Actual field water demand, and therefore concrete yield, has varied from laboratory design significantly. Ambient temperatures and weather conditions have affected the performance of the concrete. Practicality of production and of quality control procedures have been better evaluated when production-sized trial batches were prepared using the equipment and personnel that were to be used in the actual work.

3.10-Cited references
(See also Chapter 10-References)

3.1. Proportioning Concrete Mixes, SP-46, American Concrete Institute, Detroit, 1974, 240 pp.

3.2. Blick, Ronald L.; Petersen, Charles F.; and Winter, Michael E., “Proportioning and Controlling High Strength Concrete,” Proportioning Concrete Mixes, SP-46, American Concrete Institute, Detroit, 1974, p. 149.


3.9. Accelerated Strength Testing, SP-56, American Concrete Institute, Detroit, 1978, 328 pp.


CHAPTER 4- BATCHING, MIXING, TRANSPORTING, PLACING, CURING, AND CONTROL PROCEDURES

4.1-Introduction

The batching, mixing, transporting, placing, and control procedures for high-strength concrete are not different in principle from those procedures used for conventional concrete. Thus ACI 304 can be followed. Some changes, some refinements, and some emphasis on critical points are necessary. Maintaining the unit water content as low as possible, consistent with placing requirements, is good practice for all concrete; for high-strength concrete it is critical. Since the production of high-strength concrete will normally involve the use of relatively large unit cement contents with resulting greater heat generation, some of the recommendations given in Chapter 3 on Production and Delivery and Chapter 4 on Placing and Curing in ACI 305R, “Hot Weather Concreting,” may also be applicable.

In addition, the production and testing of high-strength concrete requires well-qualified concrete producers and testing laboratories, respectively.

4.2 - Batching

4.2.1 Control, handling and storage of materials- The control, handling, and storage of materials need not be substantially different from the procedures used for conventional concrete as outlined in ACI 304. Proper stockpiling of aggregates, uniformity of moisture in the batching process, and good sampling practice are essential. It may be prudent to place a maximum limit of 170 F (77 C) on the temperature of the cement as batched in warm weather and 150 F (66 C) in hot weather. Where possible, batching facilities should be located at or near the job site to reduce haul time.

The temperature of all ingredients should be kept as low as possible prior to batching. Delivery time should be reduced to a minimum and special attention paid to scheduling and placing to avoid having trucks wait to unload.

4.2.2 Measuring and weighing- Materials for production of high-strength concrete may be batched in manual, semiautomatic, or automatic plants. However, since speed and accuracy are required, ACI 304 recommends that cements and pozzolans be weighed with automatic equipment. Automatic weigh batchers or meters are recommended for water measurement. To maintain the proper water-cement ratios necessary to secure high-strength concrete, accurate moisture determination in the fine aggregate is essential. A combination of warm weather and high cement content often requires the cooling of mixing water. ACI 305R notes that the use of cold miring water effects a moderate reduction in concrete placing temperature. The use of ice is more effective than cold water; however, this will require ice making or chipping equipment at the batch plant.

4.2.3 Charging of materials- Batching procedures have important effects on the ease of producing thoroughly mixed uniform concrete in both stationary and truck mixers. The uniformity of concrete mixed in central mixers is generally enhanced by ribbon loading the aggregate, cement, and water simultaneously. However, if truck mixers are being used, ribbon loading will prevent delayed miring, which is sometimes used to prevent hydration of the cement during long hauls. This procedure involves stopping the mixer drum after aggregates and three-quarters of the water are charged and before the cement is loaded and not starting the drum again until the job site is reached. Slump loss problems may thus be minimized. High-range water-reducing admixtures are another consideration. These admixtures are very likely to be used in the production of high-strength concrete. According to the guidelines in the Canadian Standards Association’s Preliminary Standard A 266.5-M 1981, tests have shown that high-range water-reducing admixtures are most effective and produce the most consistent results when added at the end of the mixing cycle after all other ingredients have been introduced and thoroughly mired. If there is evidence of improper mixing and nonuniform slump during discharge, procedures used to charge truck and central mixers should be modified to insure uniformity of mixing as required by ASTM C 94.

4.3 - Mixing

High-strength concrete may be mixed entirely at the batch plant, in a central or truck mixer, or by a combination of the two. In general, mixing follows the recommendations of ACI 304. Experience and tests and standards documents of the Concrete Plant Manufacturers Bureau have indicated that high-strength concrete can be mixed in all common types of mixers. It may prove beneficial to reduce the batch size below the rated capacity to insure more efficient mixing.

4.3.2 Mixer performance- The performance of mixers is usually determined by a series of uniformity tests (ASTM C 94) made on samples taken from two to three locations within the concrete batch being mixed for a given time period. Some work has indicated that due to the relatively low water content and high cement content and the usual absence of large coarse aggregate, the efficient mixing of high-strength concrete is more difficult than conventional concrete. Special precautions or procedures may be required. Thus, it becomes more important for the supplier of high-strength concrete to check mixer performance and efficiency prior to production mixing.

4.3.3 Mixing time- The mixing time required is based upon the ability of the central mixer to produce uniform concrete both within a batch and between batches. Manufacturers’ recommendations, ACI 304, and usual specifi-
4.4-Transporting

4.4.1 General considerations- High-strength concrete can be transported by a variety of methods and equipment, such as truck mixers, stationary truck bodies with and without agitators, pipeline or hose, or conveyor belts. Each type of transportation has specific advantages and disadvantages depending on the conditions of use, mixture ingredients, accessibility and location of placing site, required capacity and time for delivery, and weather conditions.

4.4.2 Truck-mixed concrete- Truck mixing is a process in which proportioned concrete materials from a batch plant are transferred into the truck mixer where all mixing is performed. The truck is then used to transport the concrete to the job site. Sometimes dry materials are transported to the job site in the truck drum with the mixing water carried in a separate tank mounted on the truck. Water is added and mixing is completed. This method, which evolved as a solution to long hauls and placing delays, is adaptable to the production of high-strength concrete where it is desirable to retain the workability as long as possible. However, free moisture in the aggregates, which is part of the mixing water, may cause some cement hydration.

4.4.3 Stationary truck body with and without agitator- Units used in this form of transportation usually consist of an open-top body mounted on a truck. The smooth, streamlined metal body is usually designed for discharge of the concrete at the rear when the body is tilted. A discharge gate and vibrators mounted on the body are provided at the point of discharge. An apparatus that ribbons and blends the concrete as it is unloaded is desirable. However, water is not added to the truck body because adequate mixing cannot be obtained with the agitator.

4.4.4 Pumping- High-strength concrete will in many cases be very suitable for pumping. Pumps are available that can handle low-slump mixtures and provide high pumping pressure. High-strength concrete is likely to have a high cement content and small maximum size aggregate—both factors which facilitate concrete pumping. Chapter 9 of ACI 304 provides guidance for the use of pumps for transporting high-strength concrete. In the field, the pump should be located as near to the placing areas as practicable. Pump lines should be laid out with a minimum of bends, firmly supported, using alternate lines and flexible pipe or hose to permit placing over a large area directly into the forms without rehandling. Direct communication is essential between the pump operator and the concrete placing crew. Continuous pumping is desirable because if the pump is stopped, movement of the concrete in the line may be difficult or impossible to start again.

4.4.5 Belt conveyor- Use of belt conveyors to transport concrete has become established in concrete construction. Guidance for use of conveyors is given in ACI 304.4R. The conveyors must be adequately supported to obtain smooth, nonvibrating travel along the belt. The angle of incline or decline must be controlled to eliminate the tendency for coarse aggregate to segregate from the mortar fraction. Since the practical slump range for belt transport of concrete is 1 to 4 in. (25 to 100 mm), belts may be used to move high-strength concrete only for relatively short distances of 200 to 300 ft (60 to 90 m). Over longer distances or extended time lapses, there will be loss of slump and workability. Enclosures or covers are used for conveyors when protection against rain, wind, sun, or extreme ambient temperatures is needed to prevent significant changes in the slump or temperature of the concrete. As with other methods of transport for high-strength concrete, proper planning, timing, and control are essential.

4.5-Placing procedures

4.5.1 Preparations- Preparations for placing high-strength concrete should include recognition at the start of the work that certain abnormal conditions will exist which will require some items of preparation that cannot be provided readily the last minute before concrete is placed. Since workability time is expected to be reduced, preparation must be made to transport, place, consolidate, and finish the concrete at the fastest possible rate. This means, first, delivery of concrete to the job site must be scheduled so it will be placed promptly on arrival, particularly the first batch. Equipment for placing the concrete must have adequate capacity to perform its functions efficiently so there will be no delays at distance
portions of the work. There should be ample vibration equipment and manpower to consolidate the concrete quickly after placement in difficult areas. All equipment should be in the first class operating condition. Breakdowns or delays that stop or slow the placement can seriously affect the quality of the work. Due to more rapid slump loss, the strain on vibrating equipment will be greater. Accordingly, provision should be made for an ample number of standby vibrators, at least one standby for each three vibrators in use. A high-strength concrete placing operation is in serious trouble, especially in hot weather, when vibration equipment fails and the standby equipment is inadequate.

4.5.3 Equipment- A basic requirement for placing equipment is that the quality of the concrete, in terms of water-cement ratio, slump, air content, and homogeneity, must be preserved. Selection of equipment should be based on its capability for efficiently handling concrete of the most advantageous proportions that can be consolidated readily in place with vibration. Concrete should be deposited at or near its final position in the placement. Buggies, chutes, buckets, hoppers, or other means may be used to move the concrete as required. Bottom-dump buckets are particularly useful however, side slopes must be very steep to prevent blockages. High-strength concrete should not be allowed to remain in buckets for extended periods of time, as the delay will cause sticking and difficulty in discharging.

4.5.3 Consolidation- Proper internal vibration is the most effective method of consolidating high-strength concrete. The advantages of vibration in the placement of concrete are well established. The provisions of ACI 309 must be followed. High-strength concrete can be very "sticky" material. Indeed, effective consolidation procedures may well start with mix proportioning. Coarse sands have been found to provide the best workability. The importance of full compaction cannot be overstated. Davies has shown that up to 5 percent loss in strength may be sustained from each 1 percent void space in concrete. Thus, vibration almost to the point of excess may be required for high-strength concrete to achieve its full potential.

4.5.4 Special considerations- Where different strength concretes are being used within or between different structural members, special placing considerations are required. To avoid confusion and error in concrete placement in columns, it is recommended that, where practical, all columns and shearwalls in any given story be placed with the same strength concrete. For formwork economy, no changes in column size in the typical high-rise buildings are recommended. In areas where two different concretes are being used in column and floor construction, it is important that the high-strength concrete in and around the column be placed before the floor concrete. With this procedure, if an unforeseen cold joint forms between the two concretes, shear strength will still be available at the column interface.

4.6-Curing

4.5.1 Need for curing- Curing is the process of maintaining a satisfactory moisture content and a favorable temperature in concrete during the hydration period of the cementitious materials so that desired properties of the concrete can be developed. Curing is essential in the production of quality concrete; it is critical to the production of high-strength concrete. The potential strength and durability of concrete will be fully developed only if it is properly cured for an adequate period prior to being placed in service. Also, high-strength concrete should be cured at an early age since partial hydration may make the capillaries discontinuous. On renewal of curing, water would not be able to enter the interior of the concrete and further hydration would be arrested.

4.5.3 Type of curing- Water curing of high-strength concrete is highly recommended due to the low water-cement ratios employed. At water-cement ratios below 0.4, the ultimate degree of hydration is significantly reduced if free water is not provided. Water curing will allow more efficient, although not complete, hydration of the cement. Klieger reported that for low water-cement ratio concretes it is more advantageous to supply additional water during curing than is the case with higher water-cement ratio concretes. For concretes with water-cement ratio of 0.29, the strength of specimens made with saturated aggregates and cured by ponding water on top of the specimen was 850 to 1000 psi (5.9 to 6.9 M Pa) greater at 28 days than that of comparable specimens made with dry aggregates and cured under damp burlap. He also noted that although early strength is increased by elevated temperatures of mixing and curing, later strengths are reduced by such temperatures. However, work by Pfieffer has shown that later strengths may have only minor reductions if the heat is not applied until after time of set. Others have reported that moist curing for 28 days and thereafter in air was highly beneficial in securing high-strength concrete at 90 days.

4.5.3 Methods of curing- As pointed out in ACI 308, the most thorough but seldom used method of water curing consists of total immersion of the finished concrete unit in water. “Ponding” or immersion is an excellent method wherever a pond of water can be created by a ridge or dike of impervious earth or other material at the edge of the structure. Fog spraying or sprinkling with nozzles or sprays provides satisfactory curing when immersion is not feasible. Lawn sprinklers are effective where water runoff is of no concern. Intermittent sprinkling is not acceptable if drying of the concrete surface occurs. Soaker hoses are useful, especially on surfaces that are vertical. Burlap, cotton mats, rugs, and other coverings of absorbent materials will hold water on the surface, whether horizontal or vertical. Liquid membrane-forming curing compounds retain the original moisture in the concrete but do not provide additional moisture.
4.7-- Quality assurance

4.7.1 Materials- Once the high-strength concrete mixture has been proportioned, the concrete supplier or sampling and testing program is recommended to assure the physical properties required. The use of ASTM Standard Method for Evaluation of Cement Strength Uniformity from a Single Source (ASTM C 917) with appropriate limits will provide the proper basis for such uniformity. It is desirable that the aggregates and admixtures specified in the mixture be uniform and come from the same source for the duration of the project.

4.7.2 Control of operations - Effective coordination and control procedures between the supplier and the contractor are critical to the operations. The supplier normally has full control of high-strength concrete until it is placed in the forms. Control of the slump, time on job, mixing, and mixture adjustments is under the jurisdiction of the supplier. The contractor must be prepared to handle, place, and consolidate the concrete promptly as received. Cement hydration, temperature rise, slump loss, and aggregate grinding during mixing all increase with passage of time; thus it is important that the period between initial mixing and delivery be kept to an absolute minimum. The dispatching of trucks is coordinated with the rate of placement to avoid delays in delivery. When elapsed time from batching to placement is so long as to result in significant increases in mixing water demand, or in slump loss, mixing in the trucks is delayed until only sufficient time remains to accomplish mixing before the concrete is placed.

4.7.3 Communication equipment- Equipment for direct communication between the supply and placement locations for use by the inspection force is essential. The need for other equipment such as signaling and identifying devices depends on the complexity of the project and the number of different concrete mixtures employed. The project engineer will normally advise the contractor of the equipment that is necessary and require him to present plans or descriptions of the equipment for review well in advance of the start of placement.

4.7.4 Laboratory - A competent concrete laboratory must be available for testing the concrete delivered to the job site. This laboratory should be inspected regularly by the Cement and Concrete Reference Laboratory (CCRL) and conform to the requirements of ASTM E 329. A minimum of one set of cylinders is normally made for each 100 yd³ (76 m³) of concrete placed, with at least two cylinders cast for each test age; that is, 7, 28, 56, and 90 days.

4.7.5 Contingency plans- Plans need to be developed to provide for alternate operations in case difficulty is experienced in the basic placing concept. Backup equipment is essential, especially vibrators. Batch sixes are reduced if placing procedures are slowed. For truck-mixed concrete, rush hour traffic delays can cause serious problems. It may be desirable to reduce the elapsed time between contact of the cement and water (mixing and transporting), especially during warm weather. A maximum elapsed time of 1 1/2 hr after the cement has entered the drum until completion of discharge is frequently specified (See ASTM C 94). Reduction to 45 minutes may be necessary under hot weather conditions or where severe slump loss is experienced. For extreme job temperatures, field production trial batches are often made.

4.8-Quality control procedures

4.8.1 Criteria- The first consideration for selecting quality control procedures is determining that the distribution of the compressive strength test results follows a normal distribution curve. It has been suggested that a skew distribution may prevail due to the mean approaching a limit. This may be the case for very-high-strength concrete, 15,000 psi (103 MPa) or higher. However available data indicate that in the range of 6000 to 10,000 psi (41 to 69 MPa) normal distribution is achieved. Thus ACI 214 will normally be a convenient tool for quality control procedures for high-strength concrete. Another point which needs consideration both in the quality control and the design phase is the question of the age at the time of testing for acceptance for high-strength concrete. Compressive strength tests show that a considerable strength gain may be achieved after 28 days in high-strength concrete. To take advantage of this fact, several investigators have suggested that the specification for compressive strength should be modified from the typical 28-day criterion to either 56 or 90 days. This extension of test age would then allow, for example, the use of 7000 psi (48 MPa) concrete at 56 days in lieu of 6000 psi (41 MPa) at 28 days for design purposes. In this case the same mixture could be used to meet this criterion. High-strength concrete is generally used in high-rise structures; therefore, the extension of the time for compressive strength test results is reasonable since the lower portion of the structure will not attain full dead load for periods up to one year and longer.

4.8.2 Method of evaluation- To satisfy strength performance requirements, the average strength of concrete must be in excess of f'c', the design strength. The amount of excess strength depends on the expected variability of test results as expressed by a coefficient of variation or standard deviation and on the allowable proportion of low tests. Available information indicates that the standard deviation for high-strength concrete becomes uniform in the range of 500 to 700 psi (3.5 to 4.8 MPa), and therefore, the coefficient of variation will actually decrease as the average strength of the concrete increases. This, of course, may be the result of increased vigilance and quality assurance on the part of the producer. Thus, the method of quality control is closely related to the factors noted in Section 4.7. Assuming that the producer will devote a reasonable effort to proper quality assurance measures, the standard deviation method of evaluation appears to be a logical quality control procedure. Consider, for example, that good quality control may be expected on a job where an f'c' of 10,000 psi
(69 MPa) is required. A required average strength $f'_{c}$ of only 11,000 psi (76 MPa) is thus required with a standard deviation of 645 psi (4.4 MPa)

$$f'_{c} = f'_{c} + 134s$$

$$= 10,000 + 1.34 \times 645$$

$$= 10,864 \text{ psi (75 MPa)}$$ (4-la)

or

$$f'_{c} = f'_{c} + 2.33s - 500$$

$$= 10,000 + 2.33 \times 645 - 500$$

$$= 11,000 \text{ psi (76 MPa)}$$ (4-lb)

$s =$ standard deviation.

Of course, a close check of the field results and maintenance of records in the form of control charts or other means are necessary to maintain the desired control. Early-age control of concrete strength such as the accelerated curing and testing of compression test specimens according to ASTM C 684 is often used, especially where later-age (56 or 90 days) strength tests are the final acceptance criterion.

4.9 - Strength measurements

4.9.1 Conditions—since much of the interest in high-strength concrete is limited to strength only in compression, compressive strength measurements are of primary concern in the testing of high-strength concrete. Standard test methods of the American Society for Testing and Materials (ASTM) are followed except where changes are dictated by the peculiarities of the high-strength concrete. The potential strength and variability of the concrete can be established only by specimens made, used, and tested under standard conditions. Then standard control tests are necessary as a first step in the control and evaluation of the mixture. Curing concrete test specimens at the construction site and under job conditions is sometimes recommended since this is considered more representative of the curing applied to the structure. Tests of job-cured specimens may be highly desirable and are necessary when determining the time of form removal, particularly in cold weather, and when establishing the rate of strength development of structural members. They should never be used for quality control testing. Strength specimens of concrete made or cured under other than standard conditions provide additional information but are analyzed and reported separately. ASTM C 684 requires that a minimum of two cylinders be tested for each age and each test condition.

4.9.2 Specimen size and shape—ASTM standards specify a cylindrical specimen 6 in. (152 mm) in diameter and 12 in. (305 mm) long. This size specimen has evolved over a period of time, apparently from practical considerations. It is about the maximum weight one person can handle with reasonable effort and is large enough to be used for concrete containing 2 in. (50 mm) maximum size aggregate and smaller, which encompasses the majority of concrete being placed today. Designers generally assume 6 x 12 in. (152 x 305 mm) specimens as the standard for measured strengths. Recently some 4 x 8 in. (102 x 204 mm) cylinders have been used for determining compressive strength. However, 4 x 8 in. (102 x 204 mm) cylinders exhibit a higher strength and an increase in variability compared to the standard 6 x 12 in. (152 x 305 mm) cylinder. Regardless of the specimen size, as the compressive stress is transferred through the loading platen-specimen interface, a complex, triaxial distribution of stresses in the specimen end may develop which can radically alter the specimen failure mode and affect results.

4.9.3 Testing apparatus—Testing machine characteristics that may affect the measured compressive strength include calibration accuracy, longitudinal and lateral stiffness, stability, alignment of the machine components, type of platens, and the behavior of the platen spherical seating. Testing machines should meet the requirements of ASTM C 39 when used for testing compressive strength of cylindrical specimens. Overall testing machine design including longitudinal and lateral stiffness and machine stability will affect the behavior of the specimen at its maximum load. The type of platens and behavior of the spherical seating will affect the level of measured compressive strength.

Sigvaldason recommended a minimum lateral stiffness of $10 \times 10^{3}$ lb/in. (17.5 x $10^{6}$ N/m), and a longitudinal stiffness of $10 \times 10^{4}$ lb/in. (17.5 x $10^{6}$ N/m). He reported that a longitudinally “flexible” machine would contribute to an explosive failure of the specimen at the maximum stress, but that the actual stress achieved was insensitive to machine flexibility. However, he also noted that a machine which is longitudinally stiff but laterally flexible deleteriously influences the measured compressive strengths. Sigvaldason and Cole reported that use of proper platen size and design is critical if strengths are to be maximized and variations reduced. The upper platen must have a spherical bearing block seating and be able to rotate and achieve full contact with the specimen under the initial load and perform in a fixed mode when approaching the ultimate load. Cole demonstrated that a testing machine with a spherical bearing block seating (able to rotate under load) measured increasingly erroneous results for higher strength concretes, with reductions as high as 16 percent for 10,000 psi (69 MPa) cubes.

The diameters of the platen and spherical bearing socket are critically important. Ideally, the platen and spherical bearing block diameters should be approximately the same as the bearing surface of the specimen. Bearing surfaces larger than the specimen will be restrained (due to size effects) against lateral expansion will probably not expand as rapidly as the specimen, and will consequently create confining stresses in the specimen end. Bearing surfaces and spherical seating blocks smaller in diameter than those of the specimen may result in portions of the specimens remaining unloaded and bend-
ing of the platen around the socket with a consequent nonuniform distribution of stresses.

4.9.4 Type of mold-- The choice of mold materials, and specify construction of the mold regardless of the types of material used, can have a significant effect on measured compressive strengths. A given consolidation effort is more effective with rigidly constructed molds, and sealed waterproofed molds reduce leakage of mortar paste and inhibit the dehydration of the concrete. Blick\textsuperscript{1,15} compared high-strength specimens cast in steel and high-quality paper molds and reported that use of the rigid steel molds increased strengths approximately 13 percent but that use of either mold material did not consistently affect variability of the measured strengths. Hester\textsuperscript{4,7} evaluated a number of mold materials used under actual field conditions. Measured compressive strengths achieved with properly prepared specimens were compared. Specimens cast in steel molds achieved approximately 6 percent higher strengths but had a slightly higher coefficient of variation compared to specimens cast in tin molds. Specimens cast in steel molds achieved approximately 16 percent higher strength than specimens cast in plastic molds.

4.9.5 Specimen preparation - For many years concrete technologists have recognized the need to cap or grind the ends of cast concrete test specimens prior to testing for compressive strength. The detrimental effects of non-planeness, irregularities, and grease, etc., have been well documented.\textsuperscript{4,23} For high-strength concrete the strength of the cap, if used, is another consideration. Troxell,\textsuperscript{4,27} Werner,\textsuperscript{4,28} and other have compared the relative merits of sulfur mortars, gypsum plaster, high-alumina cements, and other capping materials. If the compressive strength or modulus of elasticity of the capping material is less than that of the specimen, loads applied through the cap will not be transmitted uniformly.

Sulfur mortar is the most widely used capping material. Most commercially available sulfur mortar capping compounds are combinations of sulfur with inert minerals and fillers and, when properly prepared, are economical, convenient to apply, and develop a relatively high strength in a short period of time. However, these materials are sensitive to the material formulations and handling practices. Kennedy\textsuperscript{4,29} and Werner investigated the effect of the thickness of sulfur mortar caps on compressive strengths of moderate strength concretes. Cap thicknesses in the range of 1/16 to 3/4 in. (1.5 to 3 mm) are desirable for use on high-strength concrete. However, caps consistently thinner than 3/4 in. (3 mm) are difficult to obtain. Kennedy\textsuperscript{4,26} and Hester\textsuperscript{4,23} note that the principal problems with thin caps are air voids at the specimen-cap interface and cracking of the specimen cap under load. Caps with a thickness of 3/4 in. (6 mm) are apparently satisfactory. Low-strength thick caps may creep laterally under load and therefore contribute to increased tensile stresses in the specimen ends and consequently substantially reduce measured compressive strengths for the concrete specimen. Gaynor\textsuperscript{4,30} and Saucier\textsuperscript{4,31} indicate that concrete strengths up to 10,000 psi (69 MPa) may be determined using high-strength capping materials, including sulfur mortar, which have strengths in the range of 7000 to 8000 psi (50 to 60 MPa), if the cap thickness is maintained at approximately 3/4 in. (6 mm). For expected compressive strengths above 10,000 psi (69 MPa), the ends are usually formed or ground to tolerance.\textsuperscript{4,29,4,30}

4.10- Cited References
(See also Chapter 10- References)


4.4. “Concrete Plant Standards of the Concrete Plant Manufacturers Bureau,” 7th Revision, Concrete Plant Manufacturers Bureau, Silver Spring, Jan. 1, 1983, 11 pp.

4.5. “Concrete Plant Mixer Standards of the Plant Mixers Manufacturers Division, Concrete Plant Manufacturers Bureau,” 5th Revision, Concrete Manufacturers Bureau, Silver Spring, July 18, 1977, 4 pp.


4.11. Davies, R.D., “Some Experiments on the Com-
paction of Concrete by Vibration,” Magazine of Concrete Research (London), V. 3, No. 8, Dec. 1951, pp. 71-78.


CHAPTER 5-PROPERTIES OF HIGH-STRENGTH CONCRETE

5.1-Introduction

Concrete properties such as stress-strain relationship, modulus of elasticity, tensile strength, shear strength, and bond strength are frequently expressed in terms of the uniaxial compressive strength of 6 x 12-m. (152 x 305-mm) cylinders. Generally, the expressions have been based on experimental data of concrete with compressive strengths less than 6000 psi (41 MPa). Various properties of high-strength concrete are reviewed in this chapter. The applicability of current and proposed expressions for predicting properties of high-strength concrete are examined.

5.2-Stress-strain behavior in uniaxial compression

Axial-stress versus strain curves for concrete of compressive strength up to 12,000 psi (83 MPa) are shown in Fig. 5.1. The shape of the ascending part of the stress-strain curve is more linear and steeper for high-strength concrete, and the strain at the maximum stress is slightly higher for high-strength concrete. The slope of the descending part becomes steeper for high-strength concrete. To obtain the descending part of the stress-strain curve, it is generally necessary to avoid the specimen-testing system interaction; this is more difficult to do for high-strength concrete. A simple method of obtaining a stable descending part of the stress-strain curve is described in References 5.3

![Fig. 5.1 -Complete compressive stress-strain curves](image-url)
and 5.7. Concrete cylinders were loaded in parallel with a hardened steel tube with a thickness such that the total load exerted by the testing machine was always increasing. This approach can be employed with most conventional testing machines. An alternate approach is to use a closed-loop testing machine. In a closed-loop testing machine, specimens can be loaded so as to maintain a constant rate of strain increase and avoid unstable failure.

High-strength concrete exhibits less internal microcracking than lower-strength concrete for a given imposed axial strain. As a result, the relative increase in lateral strains is less for high-strength concrete (Fig. 5.2). The lower relative lateral expansion during the inelastic range may mean that the effects of triaxial stresses will be proportionally different for high-strength concrete. For example, the influence of hoop reinforcement is observed to be different for high-strength concrete. It was reported that the effectiveness of spiral reinforcement is less for high-strength concrete.

![Fig. 5.2-- Axial stress versus axial strain and lateral strain for plain normal weight concrete](image)

**5.3-Modulus of elasticity**

In 1934, Thoman and Raede reported values for the modulus of elasticity determined as the slope of the tangent to the stress-strain curve in uniaxial compression at 25 percent of maximum stress from 4.2 x 10^6 to 5.2 x 10^6 psi (29 to 36 GPa) for concretes having compressive strengths ranging from 10,000 (69 MPa) to 11,000 psi (76 MPa). Many other investigators have reported values for the modulus of elasticity of high-strength concretes of the order of 4.5 to 6.5 x 10^6 psi (31 to 45 GPa) depending mostly on the method of determining the modulus. A comparison of experimentally determined values for the modulus of elasticity with those predicted by the expression given in ACI 318, Section 8.5 for lower-strength concrete, based on a dry unit weight of 145 lb/ft^3 (2346 kg/m^3) is given in Fig. 5.3. The ACI 318 expression overestimates the modulus of elasticity for concretes with compressive strengths over 6000 psi (41 MPa) for the data given in Fig. 5.3.

A correlation between the modulus of elasticity $E_c$ and the compressive strength $f'_c$ for normal weight concretes (see Fig. 5.3) was reported in Reference 5.19 as

$$E_c = 40,000 \sqrt{f'_c} + 1.0 \times 10^9 \text{ psi}$$

for 3000 psi $< f'_c < 12,000$ psi

$$E_c = 3320 \sqrt{f'_c} + 6900 \text{ MPa}$$

for 21 MPa $< f'_c < 83$ MPa

(5-1)

Other empirical equations for predicting elastic modulus have been proposed. Deviation from predicted values are highly dependent on the properties and proportions of the coarse aggregate. For example, higher values than predicted by Eq. (5-1) were reported by Russell, Saucier, and Pfeiffer.

**5.4-Poisson’s ratio**

Experimental data on values of Poisson’s ratio for high-strength concrete are very limited. Shideleker and Carrasquillo et al. reported values for Poisson’s ratio of lightweight-aggregate high-strength concrete having uniaxial compressive strengths up to 10,570 psi (73 MPa) at 28 days to be 0.20 regardless of compressive strength, age, and moisture content. Values determined by the dynamic method were slightly higher.

On the other hand, Perenchio and Klieger reported values for Poisson’s ratio of normal weight high-strength concretes with compressive strengths ranging from 8000 to 11,600 psi (55 to 80 MPa) between 0.20 and 0.28. They concluded that Poisson’s ratio tends to decrease with increasing water-cement ratio. Kaplan found values for Poisson’s ratio of concrete determined using dynamic measurements to be from 0.23 to 0.32 regardless of compressive strength, coarse aggregate, and test age for concretes having compressive strengths ranging from 2500 to 11,500 psi (17 to 79 MPa).

Based on the available information, Poisson’s ratio of high-strength concrete in the elastic range seems comparable to the expected range of values for lower-strength concretes.

**5.5-Modulus of rupture**

The values reported by various investigators for the modulus of rupture of both lightweight and normal weight high-strength concretes fall in the range of 7.5 to 12 $\sqrt{f'_c}$ where both the modulus of rupture and the compressive strength are expressed in psi. The following equation was recommended for the prediction of the tensile strength of normal weight concrete, as measured by the modulus of rupture $f'_r$ from the compressive strength as shown in Fig. 5.4

$$f'_r = 11.7 \sqrt{f'_c} \text{ psi}$$

for 3000 psi $< f'_c < 12,000$ psi

$$f'_r = 0.94 \sqrt{f'_c} \text{ MPa}$$

for 21 MPa $< f'_c < 83$ MPa

(5-2)
ACI 318, $E_c = 33 w_c^{1.5} \sqrt{f_c'}$ psi

$E_c \left( \frac{145}{w} \right)^{1.5} \times 10^6$ psi

$E_c = (40,000 \sqrt{f_c'} + 1.0 \times 10^6) \left( \frac{w_c}{145} \right)^{1.5}$ psi

$\sqrt{f_c'}$, psi

$\sqrt{f_c'}$, M Pa

Fig. 5.3—Modulus of elasticity versus concrete strength$^{5,19}$
5.6-Tensile splitting strength

Dewar studied the relationship between the indirect tensile strength (cylinder splitting strength) and the compressive strength of concretes having compressive strengths of up to 12,105 psi (83.79 MPa) at 28 days. He concluded that at low strengths, the indirect tensile strength may be as high as 10 percent of the compressive strength but at higher strengths it may reduce to 5 percent. He observed that the tensile splitting strength was about 8 percent higher for crushed-rock-aggregate concrete than for gravel-aggregate concrete. In addition, he found that the indirect tensile strength was about 70 percent of the flexural strength at 28 days. Carrasquillo, Nilson, and Slate reported that the splitting strength did not vary much from the usual range shown in Fig. 5.5, although as the compressive strength increases, the values for the splitting strength fall in the upper limit of the expected range. The following equation for the prediction of the tensile splitting strength of normal weight concrete was recommended:

$$f_{sp}' = 7.4 \sqrt{f_c} \text{ psi}$$

for 3000 psi < $f_c$ < 12,000 psi

$$f_{sp}' = 0.59 \sqrt{f_c} \text{ MPa}$$

for 21 MPa < $f_c'$ < 83 MPa

(5.3)

5.7-Fatigue strength

The available data on the fatigue behavior of high-strength concrete is very limited. Bennett and Muir studied the fatigue strength in axial compression of high-strength concrete with a 4-in. (102-mm) cube compressive strength of up to 11,155 psi (76.9 MPa) and found that after one million cycles, the strength of specimens subjected to repeated load varied between 66 and 71 percent of the static strength for a minimum stress level of 1250 psi (8.6 MPa). The lower values were found for the higher-strength concretes and for concrete made with the smaller-size coarse aggregate, but the actual magnitude of the difference was small. To the extent that is known, the fatigue strength of high-strength concrete is the same as that for concretes of lower strengths.

5.8-Unit Weight

The measured values of the unit weight of high-strength concrete are slightly higher than lower-strength concrete made with the same materials.

5.9-Thermal properties

The thermal properties of high-strength concretes fall within the approximate range for lower-strength concretes. Quantities that have been measured are specific heat, diffusivity, thermal conductivity, and coefficient of thermal expansion.

5.10-Heat evolution due to hydration

The temperature rise within concrete due to hydration depends on the cement content, water-cement ratio, size of the member, ambient temperature, environment, etc. Freedman has concluded from data of Saucier et al. in Fig. 5.6 that the heat rise of high-strength concretes will be approximately 11 to 15 F/100 lb of cement/yd$^3$ (6 to 8 °C per 59 kg/m$^3$ of cement). Values for temperature rise of the order of 100 F (56 C) in high-strength concrete members containing 846 lb of cement/yd$^3$ (502 kg/m$^3$) were measured in a building in Chicago as shown in Fig. 5.7.

5.11-Strength gain with age

High-strength concrete shows a higher rate of strength gain at early ages as compared to lower-strength concrete but at later ages the difference is not
strength concrete and 0.7 to 0.75 for lower-strength concrete, while Carrasquillo, Nilson, and Slate\textsuperscript{5,2,5,9} found typical ratios of 7-day to 95-day strength of 0.60 for low-strength, 0.65 for medium-strength, and 0.73 for high-strength concrete. It seems likely that the higher rate of strength development of high-strength concrete at early ages is caused by (1) an increase in the internal curing temperature in the concrete cylinders due to a higher heat of hydration and (2) shorter distance between hydrated particles in high-strength concrete due to low water-cement ratio.

5.12--Freeze/thaw resistance

Information about air content requirement for high-strength concrete to produce adequate durability is contradictory. For example, Saucier, Tynes, and Smith\textsuperscript{5,21} concluded from accelerated laboratory freeze-thaw tests that, if high-strength concrete is to be frozen under wet conditions, air-entrained concrete should be considered despite the loss of strength due to air entrainment. In contrast, Perenchio and Klieger\textsuperscript{5,24} obtained excellent resistance to freezing and thawing of all of the high-strength concretes used in their study, whether air-entrained or non-air-entrained. They attributed this to the greatly reduced freezable water contents and the increased tensile strength of high-strength concrete.

5.13--Shrinkage

Little information is available on the shrinkage behavior of high-strength concrete. A relatively high initial rate of shrinkage has been reported,\textsuperscript{5,26,5,30} but after drying for 180 days there is little difference between the shrinkage of high-strength and lower-strength concrete made with dolomite or limestone. Reducing the curing period from 28 to 7 days caused a slight increase in the shrinkage.\textsuperscript{5,26} Shrinkage was unaffected by changes in water-cement ratio\textsuperscript{5,15} but is approximately proportional to the percentage of water by volume in the concrete. Other laboratory studies\textsuperscript{5,31} and field studies\textsuperscript{5,16,5,22,5,28} have shown that shrinkage of high-strength concrete is similar to that of lower-strength concrete. Nagataki and Yonekur\textsuperscript{a} reported that the shrinkage of high-strength concrete containing high-range water reducers was less than for lower-strength concrete.

5.14--Creep

Parrott\textsuperscript{5,26} reported that the total strain observed in sealed high-strength concrete under a sustained loading of 30 percent of the ultimate strength was the same as that of lower-strength concrete when expressed as a ratio of the short-term strain. Under drying conditions, this ratio was 25 percent lower than that of lower-strength concrete. The total long-term strains of drying and sealed high-strength concrete were 15 and 65 percent higher, respectively, than for a corresponding lower-strength concrete at a similar relative stress level. Ngab et al.\textsuperscript{5,31} found little difference between the creep of high-strength concrete under drying and sealed conditions. The creep
of high-strength concrete made with high-range water reducers is reported\(^5.32\) to be decreased significantly. The maximum specific creep was less for high-strength concrete than for lower-strength concrete loaded at the same age.\(^5.16,5.20,5.31\) An example is shown in Fig. 5.9.\(^5.28\) However, high-strength concretes are subjected to higher stresses. Therefore, the total creep will be about the same for any strength found \(^5.22\) concrete. No problems due to creep were in columns cast with high-strength concrete. As is found with lower-strength concrete, creep decreases as the age at loading increases,\(^5.31\) specific creep increases with increased water-cement ratio,\(^5.24\) and there is a linear relationship with the applied stress.\(^5.31\) This linearity extends to a higher stress-strain ratio than for lower-strength concrete.

Some additional information on properties of high-strength concrete can be obtained from References 5.33 to 5.42.

\[\begin{array}{c}
\text{Creep Coefficient, } C_c \\
\text{Time After Loading, days}
\end{array}\]

\[\begin{array}{c}
1.00 \\
0.75 \\
0.50 \\
0.25
\end{array}\]

\[\begin{array}{c}
\text{Unsealed} \\
\text{Sealed}
\end{array}\]

\[f'_c = 4800 \text{ psi}
\]

\[f'_c = 5900 \text{ psi}
\]

\[f'_c = 9000 \text{ psi}
\]

\[f'_c = 9100 \text{ psi}
\]

\[1000 \text{ psi} = 6.895 \text{ M Po}
\]

\[f'_c / f''_c = 0.45 \text{ in All Cases}
\]

\[\text{Age at Loading} = 2 \text{ Days}
\]

\[\text{After 28 Days Curing}
\]

\[\text{Fig. 5.9-- Relationship between creep coefficient and time for sealed and unsealed concrete specimens \(^5.31\)}

### 5.15-Cited references

(See also Chapter 10-References)

5.1 Wischers, Gerd, “Applications and Effects of Compressive Loads on Concrete,” Betontechnische Berichte 1978, Betone Verlag GmbH, Dusseldorf, 1979, pp. 31-56. (in German)


CHAPTER 6-STRUCTURAL DESIGN CONSIDERATIONS

6.1—Introduction

High-strength concretes have some characteristics and engineering properties that are different from those of lower-strength concretes. Internal changes resulting from short-term and sustained loads and environmental factors are known to be different. Directly related to these internal differences are distinctions in mechanical properties that must be recognized by design engineers in predicting the performance and safety of structures. These distinctions are increasingly important as strengths increase. Tests of unreinforced high-strength concrete have shown, for example, that such material in many cases may be closely characterized as linearly elastic up to stress levels approaching the maximum stress. Thereafter, the stress-strain curve of high-strength concrete decreases at a much greater rate than lower-strength concretes.

Extensive experimentation at several research centers has provided a fundamental understanding of the behavior of high-strength concrete. While substantial information is now available on many aspects, some final recommendations must await the results of current and future work.

In this chapter, the emphasis will be placed on design of members and structures. Where recommendations are made, they are based on the best current experimental information and in most cases must be considered tentative.

6.2-Axially-loaded columns

Few columns in practice are subjected to truly axial loads. Bending moments, due to eccentric application of load or associated with rigid frame action, are usually superimposed on axial loads. ACI 318-83 requirements for design and ACI 318R-83 reflect this. However, it is useful to look first at the behavior of columns carrying axial load only.

6.2.1 Strength contribution of steel and concrete-The attribute of main interest is the ultimate strength. Present design practice, in calculating the nominal strength of an axially loaded member, is to assume a direct addition law summing the strength of the concrete and that of the steel. The justification for this is seen in Fig. 6.1, which superimposes typical stress-strain curves in compression for three concretes with that for reinforcing steel having 60,000 psi (414 MPa) yield strength (the last curve is drawn to a different vertical scale for convenience). The usual assumption is made that steel and concrete strains are identical at any load stage.

For lower-strength concrete, when the concrete reaches the range of significant nonlinearity (about 0.001 strain), the steel is still in the elastic range and consequently starts to pick up a larger share of the load. When the strain is close to 0.002, the slope of the concrete curve is nearly zero and it can be thought of as deforming plastically, with little or no increase in stress.

\[
P = 0.85 f'_c A_c + f_y A_s
\]

where \( f'_c \) = cylinder compressive strength of the concrete
\( f_y \) = yield strength of steel
\( A_c \) = area of concrete section
\( A_s \) = area of steel

The factor 0.85 is used to account for the observed difference in strength of concrete in columns compared with concrete of the same mix in standard compression-test cylinders.

A similar analysis holds for high-strength concrete columns, except the steel will yield before the concrete reaches its peak strength. However, the steel will continue to yield at essentially constant stress until the concrete is fully stressed. Prediction of strength may therefore still be based on Eq. (6-1). Experimental documentation also supports use of the factor 0.85 for high-strength concrete.

6.2.2 Effects of confinement steel-Lateral steel in columns, preferably in the form of continuous spirals, has two beneficial effects on column behavior: (a) it greatly increases the strength of the core concrete inside the spiral by confining the core against lateral expansion under load, and (b) it increases the axial strain capacity of the concrete, permitting a more gradual and ductile failure, i.e., a tougher column.

The basis for design of spiral steel under the 1977 and later versions of ACI 318 is that the strengthening effect of the spiral must be at least equal to the column strength lost when the concrete shell outside of the spiral spalls off under load. The ACI 318 equation for mini-
has shown that

\[ \rho_s = 0.45 \left( \frac{A_s}{A_c} - 1 \right) \frac{f_c'}{f_y} \]  

(6-2)

where \( \rho_s \) = ratio of volume of spiral reinforcement to volume of concrete core

\( A_s \) = gross area of concrete section

\( A_c \) = area of concrete core

\( f_c' \) = cylinder compressive strength of concrete

\( f_y \) = yield strength of spiral steel

The increase in compressive strength of columns provided by spiral steel is based on an experimentally derived relationship for strength gain

\[ \bar{f}_c - f_c'' = 4.0f_2' \]  

(6-3a)

where \( \bar{f}_c \) = compressive strength of spirally reinforced concrete column

\( f_c'' \) = compressive strength of unconfined concrete column

\( f_2' \) = concrete confinement stress produced by spiral

This relationship can be shown to lead directly to Eq. (6-2). The concrete confinement stress produced by spiral \( f_2' \) is calculated on the basis that the spiral steel has yielded, using the familiar hoop tension equation.

\[ 2A_{sp}f_y = f_2'd_s \]

or

\[ f_2' = \frac{2A_{sp}f_y}{d_s} \]

where \( A_{sp} \) = area of spiral steel

\( d_s \) = diameter of concrete core

s = pitch of spiral

and other terms are as already defined.

Recent work by Ahmad and Shah\(^6\) has shown that spiral reinforcement is less effective for columns of higher-strength concrete and for lightweight concrete columns. They found also that the stress in the steel spiral at peak load for high-strength concrete columns and lightweight concrete columns is often significantly less than the yield strength assumed in the development of Eq. (6-2).

These conclusions are consistent with results of experimental research at Cornell University\(^6\). In the Cornell research, an “effective” confinement stress \( f_2 (1 - s/dc) \) was used in evaluating results, where \( f_2 \) is the confinement stress in the concrete, calculated using the actual stress in the spiral steel, often less than \( f_y \). The term \( (1 - s/dc) \) reflects the reduction in effectiveness of spirals associated with increasing spacing of the spiral wires.\(^6\) Thus an improved version of Eq. (6-3a) is

\[ \bar{f}_c - f_c'' = 4.0f_2 (1 - s/dc) \]  

(6-3b)

Fig. 6.2 shows the results of the Cornell tests on columns using different strength concretes. It is clear that the strength gain predicted by Eq. (6-3b) is valid for normal weight concrete of all strengths for confinement stress up to at least 3000 psi (21 MPa). A similar plot based on Eq. (6-3a) shows a somewhat unconservative prediction for higher confinement stresses, but it can be shown that typical confinement stresses for practical column spirals are seldom more than about 1000 psi (7 MPa). For this range Eq. (6-3a) gives good results. From the strength viewpoint, the present ACI 318 equation for minimum spiral steel ratio can be used safely for high-strength normal weight columns as well as for lower-strength concrete columns.

Fig. 6.2 also shows that a spiral has much less confining effect in lightweight concrete columns. The lightweight concrete tends to crush under the spirals at heavy loads, relieving the confining pressure.\(^6\) For lightweight spirally reinforced columns, Martinez has suggested that Eq. (6-3a) be replaced by

\[ \bar{f}_c - f_c'' = 1.8f_2' \]  

(6-4a)

and Eq. (6-3b) be replaced by

\[ \bar{f}_c - f_c'' = 1.8f_2' (1 - s/dc) \]  

(6-4b)

This important difference in behavior means that Eq. (6-2) found in ACI 318 must be reexamined. It appears that lightweight concrete columns would require about 2.5 times more spiral steel than corresponding normal weight columns to satisfy strength requirements after the cover spalls off, a requirement that is not reflected in ACI 318. Whether or not such heavy spirals are practical may be questioned.

There is not yet general agreement on the effectiveness of spiral steel for improving the ductility of high-strength concrete columns, that is, for increasing the strain limit and flattening the negative slope of the stress-strain curve past the point of peak stress. A paper by Ahmad and Shah\(^6\) indicates that confining spirals are about as effective in flattening the negative slope of the stress-strain curve for high-strength concrete columns as for lower-strength concrete columns. However, the Cornell work\(^6\) showed significant differences. Fig. 6.3 shows experimental stress-strain curves for different strengths of normal weight concrete columns with varying spiral reinforcement. Three groups of curves are identified by the three concrete strength levels studied. Each of these groups consists of three sets of curves corresponding to three different amounts of lateral reinforcement. Indicated in each set of curves with a short horizontal line is the average unconfined column strength corresponding to that particular set of confined columns.
Referring to Fig. 6.3, the curves for high-strength concrete columns NC167 that had an effective confinement stress of 767 psi (5 MPa) are compared with the curves for lower-strength concrete columns NC163 with an effective confinement stress of 800 psi (6 MPa). Different behavior for comparable confinement stress is evident. Not only is the strain at peak stress much less for high-strength concrete, but the stress falls off sharply just past the peak value. This last fact is seen to be true even for columns NC169 with a very high confinement stress of 2500 psi (17 MPa) (probably not attainable in practical columns).

Based on the available evidence, one may conclude that normal density high-strength concrete columns with spiral steel show strength gain due to the spirals that is predicted well by present equations, but that their properties past peak stress may be deficient compared with lower-strength columns. The design of lightweight concrete columns with spiral steel should be approached very carefully.

Another interesting and important observation relating to spirally reinforced columns generally is that the level of confinement stress corresponding to spirals designed by ACI 318 is rather low for all columns. The confinement stress becomes significantly lower for larger diameter columns, assuming that the cover requirements remain constant. This follows directly from Eq. (6-2). For larger columns, the ratio $A_s/A_c$ becomes much smaller; consequently the required spiral steel ratio becomes smaller and the effective confinement stress becomes...
proportionately smaller. Confinement stress produced by spirals designed to ACI 318 for lower-and high-strength concrete, for 15 and 50 in. column core diameters are compared in Table 6.1.

**Table 6.1-Confinement stress produced by spirals designed by ACI 318**

<table>
<thead>
<tr>
<th>d, in. (mm)</th>
<th>A/A*</th>
<th>ρ*</th>
<th>f° (1 - s/d), psi (MPa)</th>
<th>s, in. (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15(38)</td>
<td>1.44</td>
<td>0.0099</td>
<td>238 (1.64)</td>
<td>2.96 (75)</td>
</tr>
<tr>
<td>50(127)</td>
<td>1.12</td>
<td>0.0028</td>
<td>83 (0.57)</td>
<td>3.17 (81)</td>
</tr>
<tr>
<td>50(127)</td>
<td>1.12</td>
<td>0.0028</td>
<td>83 (0.57)</td>
<td>3.17 (81)</td>
</tr>
<tr>
<td>5(1.27)</td>
<td>0.0330</td>
<td>825(5.69)</td>
<td>2.50(64)</td>
<td></td>
</tr>
<tr>
<td>5(1.27)</td>
<td>0.0093</td>
<td>263(1.81)</td>
<td>2.67(68)</td>
<td></td>
</tr>
</tbody>
</table>

*Ratio of volume of spiral reinforcement to total volume of core (out-to-out of spirals).

Tests show that for lower-strength concrete even the reduction in confinement stress from 238 to 83 psi (1.64 to 0.57 MPa) obtained under ACI 318 will produce a column with very large strain capacity without significant loss of resistance. For high-strength concrete, the reduction of confinement stress from 825 to 263 psi (5.69 to 1.81 MPa) produces a column with virtually no post-peak strain capacity. Even the higher confinement stress of 825 psi (5.69 MPa) produces a column with the undesirable characteristic of a sharp drop-off of resistance immediately after peak stress.6,13

While some experimental data are available at this time for high-strength concrete columns using lateral ties rather than spirals,6,18,6,19 more work must be done for such members.

### 6.2.3 Repeated loading
High-strength concrete is relatively free of internal microcracking, even close to ultimate load, when loaded monotonically.6,1 The other hand, high-strength concrete is reported to be more brittle than lower-strength concrete,6,2 lacking much of the ductility that accompanies progressive crack growth. Some experimental research indicates that fatigue strength is essentially independent of compressive strength.6,20 Recent research indicates that failure of concrete subject to repeated loading can be approximately predicted by the concept of the envelope curve, directly related to the short-term monotonic stress-strain curve.6,21 For high-strength concrete, each load application causes relatively less incremental damage. However, the number of cycles to failure may not be necessarily larger because of the greater negative slope of the post-peak envelope curve.

While important work has been done6,20,26,6,22,6,23 it is clear that additional research is needed on all aspects of high-strength concrete, with and without confinement steel, subject to various repeated load regimens, before design recommendations can be made.

### 6.2.4 Sustained loading
In most structures, concrete is subjected to sustained loads. The time-dependent strains associated with these stresses have a profound effect on the structural behavior. Such strains are directly related to long-term deflection, losses in prestress force, and cracking. Column strength may be reduced due to sustained loading of high intensity. It may also be increased because of the capability of a concrete structure to adjust itself to local high over-stresses through creep.

Creep may be described either in terms of the creep coefficient

\[
C_c = \frac{\text{creep strain}}{\text{initial elastic strain}} \quad (6-5)
\]

or by the coefficient of specific creep (unit creep coefficient)

\[
\delta_c = \text{creep strain per unit stress} \quad (6-6)
\]

The two can be related through the modulus of elasticity

\[
C_c = E_c \delta_c \quad (6-7)
\]

There is general agreement that creep of high-strength concrete is significantly less than that of a lower-strength concrete.6,7,6,24,6,27 The most recent information, for concretes with strength up to about 10,000 psi (69-MPa), indicates that high-strength concrete has a specific creep only about 20 percent that of lower-strength concrete and a creep coefficient about 30 percent as high.6,27

As a result, for axially loaded high-strength concrete columns, creep shortening at a given stress level will be less than that of lower-strength columns, a fact of possible significance in high-rise concrete structures.6,30 In addition, the distribution of load between concrete and steel of high-strength concrete columns will be less subject to change with the passage of time. Elastic distribution of stresses may be more nearly maintained. Loss of stress in a prestressed member due to creep shortening will be much less at a given concrete stress level, but this advantage may be largely canceled if higher sustained load stresses are permitted.

### 6.3-Beams and slabs
The material properties described in Chapter 5 and in Section 6.2 may effect the characteristic behavior of high-strength concrete beams.6,31-6,34 In some cases, improvements are seen; in other cases less satisfactory behavior will result. In many ways, high-strength beams may behave according to essentially the same rules that have been used to describe behavior of beams made of lower-strength concrete. However, some questions remain to be answered.

#### 6.3.1 Compressive stress distribution
The compressive stress distribution in beams is directly related to the shape of the stress-strain curve in uniaxial compression. Consequently, for high-strength concrete, which displays differences in that shape, as shown in Fig. 6.1, it is reasonable to expect differences in flexural compressive...
stress distribution, particularly at loads approaching ultimate.

In present U.S. practice as in ACI 318 and ACI 318R, proportioning of beam sections is generally based on conditions at a hypothetical state of incipient collapse at factored loads. Fig. 6.4(a) shows the generally parabolic shape of the compressive stress distribution in a beam made of lower-strength concrete. The nominal resisting moment may be calculated knowing the internal forces \( T \) and \( C \) and the internal lever arm between them. The actual shape of the compressive stress distribution at incipient failure, always highly variable even within a given range of concrete strengths, may be considered irrelevant if one knows (a) the magnitude of the compressive resultant \( C \), and (b) the level in the beam at which it acts. These may be established in terms of three parameters characteristic of a given stress distribution [see Fig. 6.4(a)].

\[
\begin{align*}
k_1 &= \text{ratio of average to maximum compressive stress in beam} \\
k_2 &= \text{ratio of depth to compressive resultant to neutral axis depth} \\
k_3 &= \text{ratio of maximum stress in beam to maximum stress in corresponding axially loaded cylinder}
\end{align*}
\]

For ordinary design purposes, it is convenient to work with an equivalent rectangular compressive stress distribution, shown in Fig. 6.4(b), with magnitude of compressive resultant and line of action the same as before. Such an equivalent distribution is specifically referenced and permitted in ACI 318 and its Commentary, ACI 318R. With the uniform value of concrete compression assumed equal to 0.85\( f'_c \), a single parameter \( \beta_j \) is sufficient to define both magnitude and line of action.

For high-strength concrete, the stress-strain curve is more linear than parabolic. Therefore, there is reason to suspect that the stress block parameters may be different. Experimental research has confirmed that differences do exist, and alternatives to the rectangular stress block have been proposed, such as in Fig. 6.4(c). However, differences in calculated strength values for beams and eccentric columns depend on steel ratios and other factors.

ACI 318R suggests, based on an equivalent rectangular stress block, that the nominal flexural strength of a singly reinforced beam that is under-reinforced can be calculated by

\[
M_n = A_f f_y d \left( 1 - 0.59 \frac{f_y}{f'_c} \right)
\]

where \( M_n \) = nominal moment strength at section, in-lb
\( A_f \) = area of tension reinforcement, in.\(^2\)
\( f_y \) = specified yield strength of reinforcement, psi
\( d \) = distance from extreme compression fiber to centroid of tension reinforcement, in.
\( \rho \) = ratio of tension reinforcement
\( f'_c \) = specified compressive strength of concrete, psi

The coefficient 0.59 can be shown to be equivalent to \( k_2/k_1k_3 \). The experimental variation of \( k_2/k_1k_3 \) with concrete compressive strength based on research at several centers is shown in Fig. 6.5. While a detailed study of the separate \( k \) values indicates that significant differences in the separate values exist depending on concrete strength, it is clear from Fig. 6.5 that the differences are compensative and that the combined coefficient is well-represented by the constant value 0.59. This statement is reinforced by the results shown in Fig. 6.6, which compares flexural strength predictions obtained using the usual rectangular stress block, a triangular stress block, and a distribution based on experimentally derived stress-strain curves with test data for beams of varying reinforcement ratios and concrete strengths to 11,000 psi (76 MPa). Test values were best predicted using actual stress-strain curves, but either the rectangular or triangular distributions gave acceptable lower bounds to the experimental and theoretical values.

Based on these and similar studies, it appears that, for under-reinforced beams, the present ACI 318 methods can be used without change, at least for concrete strengths up to 12,000 psi (83 MPa). For over-reinforced
beams, which are not allowed by ACI 318, or for members combining axial compression and bending, important differences may occur.

6.2.3 Limiting compressive strain—While high-strength concrete reaches its peak stress at a compressive strain slightly higher than that for lower-strength concrete, the ultimate strain is lower for high-strength concrete, both in uniaxial compression tests and in beam tests. It has been suggested that this result apparently is due to energy release from the testing equipment. Fig. 6.7 shows the variation of concrete strain at failure at the extreme compression face of singly reinforced concrete beams or eccentrically loaded columns without lateral confinement steel. The constant value of strain at extreme concrete compression fiber of 0.003 prescribed by ACI 318 is seen to represent satisfactorily the experimental results for high-strength as well as lower-strength concrete, although it is not as conservative for high-strength concrete.

6.3.3 Influence of confinement steel and compression Steel—Considering the more limited strain capacity of high-strength concrete in compression, it is necessary to evaluate the ductility of beams made of high-strength concrete. Deflection ductility index \( \mu \) will be defined here as

\[
\mu = \frac{\Delta_s}{\Delta_y}
\]

(6.9)

where

- \( \Delta_s \) = beam deflection at failure load
- \( \Delta_y \) = beam deflection at the load producing yielding of tensile steel

Tests by Pastor et al. of beams made of relatively high-strength concrete are summarized in Table 6.2 (Series A) and Table 6.3 (Series B). Beams of Series A were singly reinforced with no compression steel and no confinement steel. The series includes beams with concrete strengths from 3700 to 9265 psi (26 to 64 MPa). For the high-strength beams, tensile steel ratio varied from 0.29 \( \rho_b \) to 1.11 \( \rho_b \), where \( \rho_b \) = reinforcement ratio for balanced strain conditions.

The results show a lower ductility for the beams with
the higher concrete strengths. Based on these results, the second series summarized in Table 6.3 was performed. These beams included varying amounts of compression steel (50 to 100 percent of tensile steel area) and lateral confinement steel in the form of closed hoops at spacing of 3, 6, and 12 in. (7.6, 15.2, and 30.5 mm). All beams were of high-strength concrete and comparable to Beam A4 of the first series, which had no ties and no compression steel.

Table 6.2-Deflection ductility index for Series A beams

<table>
<thead>
<tr>
<th>Beam</th>
<th>$f'_c$ (psi)</th>
<th>$p/p_*$</th>
<th>Ductility index $\mu = \Delta/\Delta_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>3700</td>
<td>0.51</td>
<td>3.54</td>
</tr>
<tr>
<td>A2</td>
<td>6500</td>
<td>0.52</td>
<td>2.84</td>
</tr>
<tr>
<td>A3</td>
<td>8535</td>
<td>0.59</td>
<td>2.35</td>
</tr>
<tr>
<td>A4</td>
<td>8535</td>
<td>0.64</td>
<td>1.75</td>
</tr>
<tr>
<td>A5</td>
<td>8524</td>
<td>0.87</td>
<td>1.14</td>
</tr>
<tr>
<td>A6</td>
<td>8755</td>
<td>1.11</td>
<td>1.07</td>
</tr>
</tbody>
</table>

*Ratio of tension reinforcement divided by reinforcement ratio producing balanced strain conditions.

Table 6.3-Deflection ductility index for Series B beams

<table>
<thead>
<tr>
<th>Beam</th>
<th>$f'_c$ (psi)</th>
<th>$p/p_*$</th>
<th>Spacing of No. 2 ties A'/A</th>
<th>Ductility index $\mu = \Delta/\Delta_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>8534</td>
<td>0.57</td>
<td>$1/2$ 12 (305)</td>
<td>2.36</td>
</tr>
<tr>
<td>B2</td>
<td>8605</td>
<td>0.55</td>
<td>$1$ 12 (305)</td>
<td>2.64</td>
</tr>
<tr>
<td>B3</td>
<td>8578</td>
<td>0.57</td>
<td>$1/2$ 6 (152)</td>
<td>4.88</td>
</tr>
<tr>
<td>B4</td>
<td>8478</td>
<td>0.59</td>
<td>6 (152)</td>
<td>8.32</td>
</tr>
<tr>
<td>B5</td>
<td>8516</td>
<td>0.56</td>
<td>$1/2$ 3 (76)</td>
<td>5.61</td>
</tr>
<tr>
<td>B6</td>
<td>8466</td>
<td>0.58</td>
<td>3 (76)</td>
<td>6.14</td>
</tr>
</tbody>
</table>

From Beams B1 and B2, compared with Beam A4, it can be concluded that ties at 12 in. (30.5 mm) increased the ductility index, but not significantly. Ductility index increased markedly when the tie spacing was reduced to 6 in. (15.2 mm) in Beams B3 and B4, but showed no upward trend when the spacing was further reduced to 3 in. (7.6 mm).

A comparison between Beams B3 and B4 indicates a beneficial effect in adding more compression steel, although this trend is not clearly reflected in a comparison of Beams B5 and B6.

**6.3.4 Minimum tensile steel ratios**—ACI 318 sets an upper limit on the tensile steel ratio for beams at 75 percent of the balanced ratio to insure that failure, should it occur, will be a gradual, yielding type. A lower limit of tensile steel ratio is set to guard against sudden failure of very lightly reinforced beams upon concrete cracking, when the tension formerly carried by the concrete is transferred to the steel reinforcement.

The present ACI 318 expression for minimum steel ratio

$$\rho_{min} = \frac{200}{f'_{y}} \text{ for } f'_{y} \text{ in psi}$$

$$\rho_{min} = 0.38 \frac{f'_{y}}{f_{y}} \text{ for } f_{y} \text{ in MPa} \quad (6.10)$$

is derived on the basis that the resisting moment of the cracked section should be at least as great as the moment that caused the member to crack, based on the modulus of rupture. Since the latter is known to be greater for high-strength concrete than for lower-strength concrete, it is evident that the strength of concrete should be included in a revised version of Eq. (6.10). With modulus of rupture taken at 7.5 $\sqrt{f'_{c}}$ (0.62 $\sqrt{f'_{c}}$), it can be shown that

$$\rho_{min} = \frac{2.7 \sqrt{f'_{c}}}{f_{y}} \geq 200/(1.38 f_{y}) \quad (6.11)$$

would be an appropriate equation for all concrete strengths from 3000 to 12,000 psi (21 to 83 MPa).

**6.3.5 Shear and diagonal tension**—In current US. practice, design for shear is based on conditions at factored loads. The total shear resistance is made up of two parts: $V_c$ provided by the stirrups and $V_r$, nominally the “concrete contribution.” The nominal concrete contribution includes, in an undefined way, the contributions of the still uncracked concrete at the head of a hypothetical diagonal crack, the resistance provided by aggregate interlock along the diagonal crack face, and the dowel resistance provided by the main reinforcing steel.

High-strength concrete loaded in uniaxial compression fractures suddenly and, in so doing, may form a failure surface that is smooth and nearly a plane. This is in contrast to the rugged failure surface characteristic of lower-strength concrete. In beams controlled by shear strength, the state of stress is biaxial, combining diagonal compression in the direction from the load point to the support with diagonal tension in the perpendicular direction. Diagonal tension cracks in high-strength concrete beams can be expected to have a smooth surface, likely to be deficient in aggregate interlock.

Tests confirm that aggregate interlock decreases as concrete strength increases. Thus, a shear strength deficiency may be produced which is not accounted for by present design equations. Data from Frantz at the University of Connecticut have indicated that the calculated concrete contribution $V_c$ is adequate for high-strength concrete. Data by Nilson at Cornell University indicates that current design methods are not conservative for high-strength concrete. Experimental research by Ahmad et al. indicates that the shear strength contribution of the concrete is conservatively predicted by ACI 318-83 Eq. (11-3) for shear-span ratios of 2.5 or less, but for higher ratios, more typical in ordinary construction, and for relatively low steel ratios, the ACI equation may be unconservative. It was further
shown by their research that the more complex ACI 318-83 Eq. (11-6) is unconservative for high-strength concrete beams with low steel ratios. Recent research by Russell and Roller\textsuperscript{6,37} indicates that, for beams with high flexural steel ratios, the current ACI Code equations are safe. The beneficial effects of high-strength concrete for prestressed beams was demonstrated, using an analysis based on truss models, by Kaufman and Ramirez.\textsuperscript{6,34} Higher strength concrete increases the strength of the diagonal truss members, resulting in increased efficiency of the web reinforcement through the mobilization of more stirrups as well as increased load-carrying capacity of the struts themselves. Currently, no research data are available regarding the minimum requirement of web reinforcement to prevent brittle failure resulting from the formation of a critical diagonal crack.

6.3.6 Bond, anchorage, and development length—Present ACI 318 methods of design for development length and anchorage of tensile steel are based on tests, generally using concretes with compressive strengths not greater than about 4000 psi (28 MPa). Although some information has recently become available for high-strength concrete, not enough data have been obtained to permit recommendations.

6.3.7 Cracking—The modulus of rupture, which is the appropriate measure of concrete tensile strength for use in predicting flexural cracking load, has been reported in Chapter 5 to be 11.7 $\sqrt{f_c}$ for normal weight concretes with strengths in the range from 3000 to 12,000 psi (21 to 83 MPa). It thus appears that the ACI 318 value of 7.5 $\sqrt{f_c}$ is too low. However, for curing conditions such as seven days moist curing followed by air drying, a value of 7.5 $\sqrt{f_c}$ is probably fairly close for the full strength range. It may, therefore, be recommended with no change. The assumption of a modulus of rupture lower than the actual value for a flexible member is neither conservative nor unconservative but simply results in an inaccurate prediction of cracking load. This will result in inaccurate estimation of both elastic and creep deflections.

The direct tensile strength is seldom measured but is of interest in studying web-shear cracking in prestressed concrete members, for one example. Both modulus of rupture and tensile splitting strength of high-strength concrete, not enough data have been obtained to permit recommendations.

6.3.8 Elastic deflections—The main uncertainties in predicting elastic deflections of reinforced concrete beams are (a) elastic modulus $E_c$; (b) modulus of rupture $f_c'$; and (c) effective moment of inertia, which depends on the extent of cracking of the beam.

For the elastic modulus, the following equation of Chapter 5 may be used unless actual values of modulus are known.

$$E_c = 40,000 \sqrt{f_c} + 1.0 \times 10^6 \text{ psi}$$

$$E_c = 3320 \sqrt{f_c} + 6900 \text{ MPa} \quad (6-12)$$

Eq. (6.12) should be modified by the correction factor $(w/c/145)1.5$ [for SI units $(w/c/2300)1.5$ for concrete densities other than 145 lb/ft$^3$ (2320 kg/m$^3$)].\textsuperscript{6,2,6,13,6,46}

The modulus of rupture has been discussed in Section 6.3.7. For prediction of deflections a value of 7.5 $\sqrt{f_c}$ may be used to calculate the flexural cracking moment of the beam. The equation for effective moment of inertia $I_e$ included in the ACI 318 is

$$I_e = \left( \frac{M_{cr}}{M_a} \right)^3 I_g + \left[ 1 - \left( \frac{M_{cr}}{M_a} \right)^3 \right] I_{cr} \quad (6-13)$$

where

- $M_{cr}$ = cracking moment
- $M_a$ = maximum moment
- $I_g$ = gross moment of inertia of section
- $I_{cr}$ = moment of inertia of cracked transformed section

This provides a basis for beam deflection calculation that appears valid for high-strength concrete as well as normal concrete beams, based on information currently available.\textsuperscript{6,34,6,47,6,48}

6.3.9 Time-dependent deflections—Time-dependent deflections of beams due to creep and shrinkage are presently calculated by applying multipliers to computed elastic deflections. This procedure is generally valid for high-strength concrete members, but experimental data indicates that the multipliers may be significantly less because of the lower creep coefficient typical of high-strength concrete. According to ACI 318, additional long-term deflections are obtained using the following multiplier

$$\xi = \frac{\xi}{1 + 50 \rho'}$$

where

- $\rho'$ = reinforcement ratio for nonprestressed compression reinforcement
- $\xi$ = time-dependent factor

The time-dependent factor is given by Fig. 6.8, taken from ACI 318R.

Research in progress,\textsuperscript{6,47,6,48} providing an indication of long-term multipliers and their variation with time, is summarized in Fig. 6.9. Results are currently available up to about 1 year loading age, and clear trends are evident, as follows:
6.3.10 Repeated Loading—With reference to Section 6.2.3, it appears that high-strength concrete, because of its relative freedom from internal microcracking at service loads, would be more resistant to repeated loading consisting of a large number of cycles at relatively low stress ranges such as in bridges. If ductility is an important consideration, as is the case in seismic resistant design, it would be important to include lateral confinement steel in the form of closed hoop stirrups as well as compression reinforcement. While the subject has been thoroughly studied for lower-strength concrete, little information on high-strength concrete beams subject to repeated loads is available at this time.

6.3.11 Prestressed concrete beams—Characteristics of high-strength concrete, discussed previously in this chapter in the context of axially loaded members and reinforced concrete beams, affect the behavior of prestressed concrete beams in corresponding ways. Special mention must be made, however, of the effects of a very low creep coefficient.

At the same concrete stress levels, time-dependent deflection of high-strength beams will be less. On the other hand, low concrete creep may have little effect on prestressed beam deflections because upward creep deflection due to prestress is, in many cases, canceled by downward creep deflection due to sustained loads. This results in only very small net deflections associated with all sustained loads.

For a given level of concrete stress, loss of prestress force due to creep could be expected to be much smaller for prestressed beams using high-strength concrete. Higher sustained concrete stress would negate this advantage.
6.4 Eccentric columns

6.4.1 Compressive stress distribution—It was pointed out in discussing beams in Section 6.3.1 that the shape of the compressive stress distribution in high-strength concrete beams is apt to be different from that in lower-strength concrete beams, reflecting the different shape of the compressive stress-strain curve as shown in Fig. 6.1. For under-reinforced concrete beams, with strength controlled by the yield strength of the reinforcement, the actual shape of the compressive stress block used in calculation of the nominal flexural strength is of little importance so long as the internal lever arm to the compressive resultant is close to the true value. The conventional rectangular stress block and equations for determining nominal flexural strength based on the rectangular stress block will normally be satisfactory. Over-reinforced beams are not permitted according to ACI 318, and so one concludes that present procedures will produce adequate results for all beams designed under provisions of ACI 318, whether lower- or high-strength concrete is used.

In the case of combined bending and axial load, i.e., eccentric columns, members failing in flexural compression cannot be avoided. For members with relatively low eccentricity, failure will be initiated by the concrete reaching its limiting strain, while the steel on the far side of the column may be well below tensile yielding or may remain in compression at the failure load. For such cases, a more accurate representation of the concrete compressive stress block could be important.

6.4.2 Interaction diagram for strength of short columns—Limited analytical studies have been made of eccentric columns comparing the predictions of the current ACI 318 Commentary approach based on the equivalent rectangular stress block, with a trapezoidal concrete stress distribution. The general shape of the trapezoid would vary, ranging from nearly rectangular for lower-strength concrete to nearly triangular for very-high-strength as discussed in Section 6.3.1. Fig. 6.10 shows a comparison of the strength interaction diagram relating axial load capacity \( P_n \) and flexural capacity \( M_n \) for a 14 in. column made of 12,000 psi (83 MPa) strength concrete. Reinforcement is provided by four No. 11 corner bars having yield strength \( f_y = 60,000 \) psi (414 MPa). Strength under combined axial load and bending was computed first using the conventional rectangular stress block (solid line), then using a variable-proportioned trapezoid (dashed line).

For relatively large eccentricities, when moment dominates and failure is initiated by tensile reinforcement yielding, the two curves are almost indistinguishable. For intermediate to small eccentricities, ACI 318 results in larger values for both moment and axial force at a given eccentricity at failure than those obtained by the more exact calculation. Differences of up to 15 percent in the interaction diagram relating moment to axial load have been found based on comparative calculations.

ACI 318 procedures in corporate an assumed concrete

strained limit in compression of 0.003. It has been shown in Section 6.3.2 that this is less conservative for high-strength concrete than for lower-strength concrete. In the presence of effective lateral confinement, such as provided by continuous spirals in normal weight concrete columns, the effective strain limit is larger than this value, and strain compatibility analysis can be based on 0.003 strain. However, there is no apparent justification for increasing limiting strain assumptions above present values.

6.4.3 Slenderness effects—The moment magnification method for dealing with slenderness effects in reducing the strength of reinforced concrete columns appears to be generally valid for high-strength concrete. An exception may be in the equations for calculating effective flexural rigidity. Two alternative equations are given in ACI 318 for flexural rigidity, both of which include factors to account for the effect of concrete creep in an approximate way. The validity of these equations for high-strength concrete may at least be questioned, recognizing the significantly lower creep coefficient for high-strength concrete. No experimental information is available at this time. In addition, calculations should incorporate estimates of \( E_c \) as given by Eq. (6-12).

6.5 Summary

6.5.1 Review—A brief summary has been given of the special characteristics of high-strength concrete as they bear upon the behavior and design of reinforced concrete members and structures.

For axially loaded columns, the direct addition of concrete and steel strength contributions is generally valid, as for lower-strength concrete members. Lateral steel plays a particularly important role in that it is necessary to improve ductility and toughness. Of special concern is
the sharp drop-off of load after peak stress and the apparent diminished effectiveness of lateral steel in increasing ductility compared with lower-strength concrete columns. Further studies are needed. High-strength concrete columns will exhibit less shortening under load than lower-strength concrete columns because of the higher elastic modulus and lower creep coefficient.

For beams, use of the conventional equivalent rectangular stress block appears to give satisfactory results for under-reinforced members required by ACI 318 procedures. The compressive strain limit is less than found for lower-strength concrete but still may be taken at 0.003. Confinement steel and compressive steel should be used in designing concrete beams where ductility is important as for seismic resistant structures. Changes have been recommended for ACI 318 values for minimum tensile steel ratio to reflect the influence of concrete strength, as well as in the modulus of elasticity to be used in deflection calculations. Significant changes should also be considered in the calculation of long-term beam deflections to reflect the much lower creep coefficient and reduced effectiveness of compression steel in the case of high-strength concrete beams.

The calculation of eccentric column strength may be influenced by the shape of the compressive stress block used, particularly for columns with relatively small eccentricity with neutral axis at failure close to an edge. Limited trial calculations comparing rectangular stress block with trapezoidal stress block indicate only small differences. In determining slenderness effects, special consideration should be given to the lower creep coefficient for high-strength concrete, as it affects the effective flexural rigidity used in the calculations, and to improved values of modulus of elasticity.

6.5.2 Research needs-The material of Chapter 6 should be considered to be subject to revision based on future research results. Areas in which information is lacking include shear, diagonal tension, torsion, bond, anchorage, development length, and the effects of repeated loading. Research programs are now in progress in several centers that are aimed at filling some of these gaps. In this way, the research base will be expanded so that the many advantages of high-strength concrete may be used safely and with confidence based on thorough documentation of material properties and behavioral characteristics of members.

6.6-Cited references
(See also Chapter 10--References)


6.11. Nilson, A.H., “Design Implications of Current Research on High-Strength Concrete,” High-Strength Concrete, SP-87, American Concrete Institute, Detroit, 1985, pp. 85-118.


6.17. Slate, F.O., Nilson, A.H., and Martinez, S.,


CHAPTER 7--ECONOMIC CONSIDERATIONS

7.1—Introduction

As earlier chapters have demonstrated, high-strength concrete is a state-of-the-art material, and like most state-of-the-art materials, it commands a premium price. In some instances, the benefits are well worth the additional effort and expense; in others they are not. Before the cost/benefit trade-offs in specific applications are discussed, the economic considerations regarding the use of high-strength concrete generally will be examined.

In many areas and for many uses, the benefits of high-strength concrete more than compensate for the increased costs of raw materials and quality control. Basically, high-strength concrete will carry a compression load at less cost than any lower-strength concrete. Chicago-based structural engineers William Schmidt and Edward S. Hoffman compiled charts indicating the cost of supporting 100,000 lb (445 kN) of service load comes to $5.02 per story with 6000 psi (41 MPa) concrete, $4.21 with 7500 psi (52 MPa), and drops to $3.65 with 9000 psi (62 MPa) concrete (which the authors report they had no difficulty obtaining in the Chicago area). "While the figures reflect 1975 costs, the ratio should remain similar. The reason for these economies is that, although the concrete itself is more costly than lower-strength mixtures, the cost differential is offset by significant reduction in the given member size. This capability is particularly attractive for use in columns. Since column size is so important for architectural and rental reasons, the ability to limit the sixes for taller structures often allows the use of a concrete solution in lieu of one of structural steel.

In 1976, Architectural Record noted that "... a 30 x 30-in. column of 6,000 psi concrete might require an amount of reinforcing steel equal to 4 percent of the column area for a given load, whereas the same column in 9,000 psi would require only 1 percent steel-the minimum allowed by code."

7.2—Cost studies

The Material Service Corporation conducted a pricing study that dramatically demonstrated the cost advantage of replacing percentages of steel with high-strength concrete in short tied columns. This 1983 study was made for a column supporting a design load (1.4D + 1.7L) of 1000 kips (4.45 MN) and based on the following prices:

- Reinforcing steel $760/ton in place
- 7000 psi concrete $80/yd in place
- 9000 psi concrete $85/yd in place
- 11,000 psi concrete $104/yd in place
- 14,000 psi concrete $129/yd in place
- Formwork $280/yd in place

As Fig. 7.1 shows, using high-strength concrete with a minimum of steel is the most economical solution.

Fig. 7.1--Cost of columns

7.3—Case histories

Two examples might help translate this savings into actual dollars.

7.3.1 Case history No. 1-In 1968, Philadelphia’s first high-rise office building using 6000 psi (41 MPa) concrete was designed. To meet the span requirements [approximately 30 ft (9m) square bays] while avoiding unacceptable oversized columns on the lower floors, the columns of the first three floors were built of structural steel. However, a comparison study made by the designing engineers for 8000 psi (55 MPa) concrete showed that

7.3.1.1 With the same column sixes as the original unacceptable sixes, a 60 percent reduction in reinforcing steel with the 8000 psi (55 MPa) concrete would have been made. This would also have resulted in 24 fewer splices per column, a side benefit in labor and time cost savings.

7.3.1.2 With the same amount of reinforcing used
as in the original column, the column size could have been reduced from 36 x 46 in. (915 x 1170 mm) to 30 x 30 in. (760 x 760 mm). This size would have been acceptable to the architect and owner and would have eliminated the need for an additional trade-structural steel-on the job.

Rough calculations show that 8000 psi (55 MPa) concrete for the lower-floor columns with a stepped strength reduction, as the building reached the upper floors, to 3000 psi (21 MPa) concrete at the top would have resulted in a column size that met the demands of the architect, owner, and rental agent. This would have saved close to $530,000 in 1968 dollars.

7.3.2 Case history No. 2-- The economies of high-strength concrete were more dramatically demonstrated in the construction of New York City’s first building using 8000 psi (55 MPa) concrete, The Palace Hotel built in 1979. The building was originally conceived using structural steel for the lower floors, designated for ballroom and restaurant functions, with a reinforced concrete superstructure for the hotel facilities. However, the engineers were able to convert the entire design, except for two columns on the lowest four levels, to reinforced concrete through the use of 8000 psi (55 MPa) concrete. These ballroom and restaurant areas required large column spacing. The common limitations of 6000 psi (41 MPa) concrete would have made the columns prohibitively large and uneconomical. A presentation to the New York City Building Department about the values of high-strength concrete, together with the proposed controls to insure quality, resulted in its acceptance for use in New York.

Concrete with compressive strengths of 8000 and 7000 psi (55 and 48 MPa) was used in the columns of the building only. Lightweight concrete with compressive strength of 3500 psi (24 MPa) was used for floor slabs, and 5000 to 6000 psi (34 to 41 MPa) concrete was used in wall construction. On the lower five levels of the hotel, column sizes were reduced by approximately 25 percent. Approximately 10 percent less reinforcing steel was used because of the strength of the concrete. In addition, No. 11 reinforcing bars remained a viable size, avoiding the need for mechanical connections between the reinforcing bars, thus considerably reducing the floor-framing time requirements. Further economies were realized by minimizing changes in column sizes and reducing column reinforcement on the upper floors.

The ability to reduce the amount of costly reinforcing steel without sacrificing strength is an attractive benefit to owners, builders, and engineers, but the use of high-strength concrete in building columns has a corollary economic benefit. It enables the lower floors of high-rise buildings to maintain an acceptable column size, while at the same time increasing the number of possible stories.

This is a case of a relatively new material meeting the needs of market economics. The Chicago Committee study noted “The potential number of stories in high-rise buildings is limited by the required large columns if they were to be built with ordinary low-strength concrete. Real estate properties in prime locations had to be developed with maximum rental floor area. Architectural layout of apartment or condominium units demanded flexibility, which is restricted by large columns. High-strength concrete satisfied this condition by allowing column sizes to be reduced to a minimum.”

7.4-Other studies

In Ontario, Canada, the Richmond-Adelaide Center’s use of high-strength concrete columns enabled the architect to increase the use of the underground parking garage by approximately 30 percent. In times when all building construction is difficult to capitalize, a material that both reduces construction costs and substantially increases the amount of revenue-gathering space within a building can be a tremendous factor in the decision to build.

7.5-Selection of materials

The economic consequences of requiring fly ash may vary. On The Royal Bank Plaza Project in Ontario, Canada, a 43-story building constructed from 1973-1976 (one of the first to use fly ash in high-strength concrete), all of the various strength concrete mixtures on the project were converted to local fly ash. This resulted in a saving of approximately $100,000 over the contract and produced concretes with extremely good fresh and hardened properties.

The Scotia Plaza-A 68-story building in Toronto, Canada, constructed in 1988—is one of the first buildings to employ the use of silica fume in concrete as an element in increasing strength. Strengths of up to 10,000 psi have been achieved. Two Union Square in Seattle employs 19,000-psi concrete containing silica fume—the highest strength used to date in a conventional building.

7.6-Quality control

While selection of materials will influence costs, another factor, and one more exclusively the result of the use of high-strength concrete, is the cost of the increased testing, quality control, and inspection that the use of high-strength concrete requires. The quality and consistency of the concrete is crucial, and additional steps must be taken to insure that quality and consistency.

In the Royal Bank Plaza Project, a number of precautions were necessary. The supplier had to have a quality control person at the site to control both the scheduling of trucks and the consistency of the concrete at the time it was delivered. For this central plant project, the supplier agreed that there would be no water added to the trucks after they had come onto the site and that any minor adjustments would be made prior to sending the truck to the site. Regular visits were made to the batch plant to check batching procedures and to obtain the test samples. Furthermore, a full-time technician was employed to carry out sampling and testing on site. This was
found to be an essential feature of quality control.

On a later project, Richmond-Adelaide Center, Phase II, Ontario, Canada, 1977-1979, by the same engineers, not only did the supplier maintain full-time inspection on the site to insure that the delivered material met requirements but the engineer employed by the owner also maintained full-time inspection and regularly inspected the batch plant. Often this type of stringent quality control is required by regulation. For The Palace Hotel, the New York City Building Department stipulated that at least two suppliers of concrete prequalify the concrete mixtures to strengths up to 8000 psi (55 MPa). The prequalification was to be performed by an independent testing laboratory, and a full-time professional engineer would be required to continuously inspect the progress of the work, performing no other work during the construction. For hot weather concreting, the engineers required mixing water limited to no more than 50 °F (10 °C) and the truck drums to be hosed down if standing in direct sunlight. Further, all trucks were limited to 10 yd³ (7.6 m³) loads, despite capacities of 16 yd³ (12.2 m³). While the professional inspection does add to the cost, the continuing education of the suppliers and concrete subcontractors in the areas of quality control should ultimately create better concretes of all strengths and result in better and more economical use of materials.

7.7-Areas of application

In general, the economic advantages of high-strength concrete are most readily realized when the concrete is used in the columns of high-rise buildings. In this application, engineers may take full advantage of its increased compressive strength: reducing the amount of steel, reducing column size to increase usable floor space, or allowing additional stories without detracting from lower floors. These benefits overshadow the increased quality control costs and possible higher cost of raw materials discussed earlier. Yet the use of high-strength concrete has also spread to other applications, primarily slabs, beams, and long-span bridges. The economic considerations of these uses should also be examined.

Parking garages, bridge decks, and other installations requiring improved density, lower permeability, and increased resistance to freeze-thaw and corrosion have become prime candidates for consideration of the use of high-strength materials.

The primary advantage of high-strength concrete in slabs is the resulting reduction in dead load. However, as Schmidt and Hoffman point out, significant economies can be achieved only by reducing the thickness that is required for stiffness; the additional reinforcement required may offset the concrete savings. Used for rectangular beams, T-beams, and one-way slabs, high-strength concrete yields reduced section width or thickness and allows for longer spans, but (as with slabs) less expensive lightweight concrete continues to perform this job satisfactorily. Presently, there is no economic justification, under normal circumstances, for the use of a premium material such as high-strength concrete for slabs or beams.

Long-span bridges are another area where the qualities of high-strength concrete are proving themselves economically attractive. High-strength concrete’s comparatively greater compressive strength per unit weight and unit volume allows lighter, more slender bridge piers. This provides improved horizontal clearances. In addition, the increased stiffness of high-strength concrete is advantageous when deflections or stability govern the bridge design. Increased tensile strength of high-strength concrete is helpful in service load design in prestressed concrete.

In bidding to build a cable-stayed bridge across the Ohio River, a concrete-deck proposal beat steel by 29 percent—roughly $10 million. The two-lane crossing between Huntington, West Virginia, and Proctorville, Ohio, includes the first major asymmetrical stayed-girder structure in the United States. The bridge has a main span of 900 feet over one pier. The three bids to construct the bridge using concrete ranged from $23.5 million to $29.7 million, all well below the lowest steel bid ($33.3 million). The designer, Arvid Grant Associates, specified box girders only 5 ft (1.5 m) deep cast of 8000 psi (55 MPa) high-strength concrete. 7.8

7.8-Conclusion

The economic benefits of high-strength concrete are just now becoming fully apparent. Certainly as the use of high-strength concrete increases, additional and possibly even greater benefits will be realized. In any case, those projects that have led the way in the use of high-strength concrete have clearly demonstrated its economic advantages. For now, it allows the profession to engineer most cost effectively and space effectively. In the future, those considerations may tip the balance on whether certain projects are constructed at all.

7.9-Cited references

(See also Chapter 10—References)


7.4. Private correspondence from J. Moreno of Material Service Corp. to Irwin G. Cantor, May 12, 1983.


CHAPTER 8—APPLICATIONS

8.1—Introduction

Some specific applications of high-strength concrete are described in this chapter. Separate sections describe applications in buildings, bridges, and special structures. The applications are not all-inclusive but demonstrate a range of applications of high-strength concrete. Some potential applications for high-strength concrete are also discussed.

Table 8.1-- Buildings with high-strength concrete

<table>
<thead>
<tr>
<th>Building</th>
<th>Location</th>
<th>Year*</th>
<th>Total Stories</th>
<th>Maximum design concrete strength, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>S.E. Financial center</td>
<td>Miami</td>
<td>1982</td>
<td>53</td>
<td>7000</td>
</tr>
<tr>
<td>Petrocanada Building</td>
<td>Calgary</td>
<td>1982</td>
<td>34</td>
<td>7250</td>
</tr>
<tr>
<td>Lake Point Tower</td>
<td>Chicago</td>
<td>1965</td>
<td>70</td>
<td>7500</td>
</tr>
<tr>
<td>1130 S. Michigan Ave.</td>
<td>Chicago</td>
<td></td>
<td></td>
<td>7500</td>
</tr>
<tr>
<td>Texas Commerce Tower</td>
<td>Houston</td>
<td>1981</td>
<td>75</td>
<td>7500</td>
</tr>
<tr>
<td>Helmsley Palace Hotel</td>
<td>New York</td>
<td>1978</td>
<td>53</td>
<td>8000</td>
</tr>
<tr>
<td>Trump Tower</td>
<td>New York</td>
<td>1981</td>
<td>68</td>
<td>8000</td>
</tr>
<tr>
<td>City Center Project</td>
<td>Minneapolis</td>
<td>1981</td>
<td>52</td>
<td>8000</td>
</tr>
<tr>
<td>Collins Place</td>
<td>Melbourne</td>
<td>1981</td>
<td>44</td>
<td>8000</td>
</tr>
<tr>
<td>Larimer Place Condominium</td>
<td>Denver</td>
<td>1980</td>
<td>31</td>
<td>8000</td>
</tr>
<tr>
<td>499 Park Avenue</td>
<td>New York</td>
<td>1975</td>
<td>43</td>
<td>8800</td>
</tr>
<tr>
<td>Royal Bank Plaza</td>
<td>Toronto</td>
<td>1978</td>
<td>33</td>
<td>8800</td>
</tr>
<tr>
<td>Richmond-Adelaide Center</td>
<td>Chicago</td>
<td>1972</td>
<td>50</td>
<td>9000</td>
</tr>
<tr>
<td>Midcontinental Plaza</td>
<td>Chicago</td>
<td>1975</td>
<td>79</td>
<td>9000</td>
</tr>
<tr>
<td>Water Tower Place</td>
<td>Chicago</td>
<td>1976</td>
<td>56</td>
<td>9000†</td>
</tr>
<tr>
<td>River Plaza</td>
<td>Chicago</td>
<td>1982</td>
<td>40</td>
<td>9000‡</td>
</tr>
<tr>
<td>Chicago Mercantile Exchange</td>
<td>Chicago</td>
<td>1983</td>
<td>76</td>
<td>9500</td>
</tr>
<tr>
<td>Columbia Center</td>
<td>Seattle</td>
<td>1983</td>
<td>72</td>
<td>10,000</td>
</tr>
<tr>
<td>Interfirst Plaza</td>
<td>Dallas</td>
<td>1983</td>
<td>44</td>
<td>11,000</td>
</tr>
<tr>
<td>Eugene Terrace</td>
<td>Chicago</td>
<td>1988</td>
<td>70</td>
<td>12,000‡</td>
</tr>
<tr>
<td>311 S. Wacker Drive</td>
<td>Chicago</td>
<td>1986</td>
<td>15</td>
<td>14,000</td>
</tr>
<tr>
<td>900 N. Michigan Annex</td>
<td>Chicago</td>
<td>1987</td>
<td>62</td>
<td>14,000†</td>
</tr>
<tr>
<td>Two Union Square</td>
<td>Seattle</td>
<td>1988</td>
<td>30</td>
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<tr>
<td>225 W. Wacker Drive</td>
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<td>1988</td>
<td>15</td>
<td>14,000</td>
</tr>
<tr>
<td>Scotia Plaza</td>
<td>Toronto</td>
<td>1988</td>
<td>68</td>
<td>10,000</td>
</tr>
</tbody>
</table>

* Year in which high-strength concrete was cast.
† Two experimental columns of 11,000 psi strength were included.
‡ Two experimental columns of 14,000 psi strength were included.
§ 9,000 psi also used in flow slabs of lower level.
* 19,000 psi indirectly specified to achieve a high modulus of elasticity.
Table 8.2-- Bridges with high-strength concrete

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Location</th>
<th>Year</th>
<th>Maximum span, ft</th>
<th>Maximum design concrete strength, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Willsows Bridge</td>
<td>Toronto</td>
<td>1967</td>
<td>158</td>
<td>6,000</td>
</tr>
<tr>
<td>Houston Ship Canal</td>
<td>Texas</td>
<td>1981</td>
<td>750</td>
<td>6,000</td>
</tr>
<tr>
<td>San Diego to Coronado</td>
<td>California</td>
<td>1969</td>
<td>140</td>
<td>6,000 1.*</td>
</tr>
<tr>
<td>Linn Cove Viaduct</td>
<td>North Carolina</td>
<td>1979</td>
<td>180</td>
<td>6,000</td>
</tr>
<tr>
<td>Pasco-Kennewick Intercty</td>
<td>Washington</td>
<td>1978</td>
<td>98</td>
<td>6,000</td>
</tr>
<tr>
<td>Cowenman River Bridges</td>
<td>Washington</td>
<td>1978</td>
<td>146</td>
<td>7,000</td>
</tr>
<tr>
<td>Huntington to Proctorville</td>
<td>W. Va. to Ohio</td>
<td>1984</td>
<td>900</td>
<td>8,000</td>
</tr>
<tr>
<td>Annicus Bridge</td>
<td>British Columbia</td>
<td>1986</td>
<td>1526</td>
<td>8,000</td>
</tr>
<tr>
<td>Nitta Highway Bridge</td>
<td>Japan</td>
<td>1968</td>
<td>98</td>
<td>8,500</td>
</tr>
<tr>
<td>Kaminoshima Highway Bridge</td>
<td>Japan</td>
<td>1970</td>
<td>282</td>
<td>8,500</td>
</tr>
<tr>
<td>Tower Road</td>
<td>Washington</td>
<td>1987</td>
<td>161</td>
<td>9,000</td>
</tr>
<tr>
<td>Fukamitsu Highway Bridge</td>
<td>Japan</td>
<td>1974</td>
<td>85</td>
<td>10,000</td>
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<tr>
<td>Ootonabe Railway Bridge</td>
<td>Japan</td>
<td>1973</td>
<td>79</td>
<td>11,400</td>
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<tr>
<td>Akkagawa Railway Bridge</td>
<td>Japan</td>
<td>1976</td>
<td>150</td>
<td>11,400</td>
</tr>
</tbody>
</table>

* Lightweight concrete
Metric equivalent: 100 psi = 0.6895 MPa

The effect of using high-strength concrete in four different solid-section girders has been described by Carpenter.8.8 For integral deck bulb tees, span capability for closely spaced girders increased with increase in concrete strength. For wider spaced girders, capability increased when concrete strength was increased up to 8000 psi (55 MPa). Above 8000 psi (55 MPa) compressive strength, span capability did not increase because sufficient prestress forces could not be provided. Similar results were obtained for other cross sections.

For post-tensioned box girder bridges, Carpenter reported that high-strength concrete can be used to increase span capability. However, for higher-strength concretes, maximum available prestress force again limited maximum spans. For segmental box girder bridges, he showed that high-strength concrete is feasible in regions where member thickness is controlled by stress. However, where thickness is controlled by other factors, high-strength concrete may not be beneficial.

Some actual bridges in which the use of high-strength concrete has been reported are listed in Table 8.2. Perhaps the most significant application in the United States is the Huntington, West Virginia, to Proctorville, Ohio, crossing for which a compressive strength of 8000 psi (55 MPa) was specified. The bridge consists of an asymmetrical stayed-girder superstructure with a main span of 900 ft (274 m).

The use of concrete with compressive strengths up to 11,000 psi (76 MPa) in railway bridges in Japan has also been reported. Nagatak8.11 reports that strengths of 11,400 psi (79 MPa) can be easily obtained in the field in Japan.

8.4-Special applications

In 1948, concrete with a specified compressive strength of 9000 psi (62 MPa) was used for precast panels for a powerhouse at Fort Peck Dam, Montana. High-strength concrete was specified to provide an extremely dense concrete that would withstand the harsh exposure. Actual compressive strengths of concrete were reported to be considerably higher than 9000 psi (62 MPa).

The use of 10,000 psi (69 MPa) concrete for prestressed concrete poles produced by spinning has been described by Skrastins. High-strength concrete was selected to reduce the size of the poles.

Copen has indicated that the use of 10,000 psi (69 MPa) concrete in thin arch dams would usually result in greater economy through reduced volume of concrete. High-strength concrete would tend to reduce deflections in a dam and may improve strength of construction joints and permit earlier removal of formwork. Disadvantages of high-strength concrete listed by Copen include development of stress concentrations, particularly in the foundation for the dam; tendency toward more cracking in concrete; increased temperature control problems; and complications involved with openings through the dam and railways over the dam.

The application of high-strength concrete in two grandstand roofs has been described by Bobrowski. Lightweight concrete with a density of 118 lb/ft³ (1.84 Mg/m³) and a minimum cube strength of 7500 psi (52 MPa) at 28 days was used in the roof beams at Doncaster racecourse, England. Roof beams at Leopardstown racecourse, Ireland, had 28 day cube compressive strengths between 7200 and 8850 psi (50 and 61 MPa) and an average density of 115 lb/ft³ (1.84 Mg/m³).

Anderson has reported the use of high-strength concrete in piles for marine foundations in northwestern United States. Measured 28 day compressive strengths ranged between 7900 and 9900 psi (55 to 68 MPa). High-strength concretes with compressive strengths up to 9400 psi (65 MPa) have also been used for decks of dock structures in the northwestern United States.

In 1984, the Glomar Beaufort Sea I, placed in the arctic, This exploratory drilling structure contains about 12,000 yd³ (9200 m³) of high-strength lightweight concrete.
Table 8.3-- Corrosion resistance data for selected high-strength concrete (data from reference 8.29)

<table>
<thead>
<tr>
<th>Strength (psi)</th>
<th>Microsilica† (wt. %)</th>
<th>Chloride Permeability* (Cloulombs)</th>
<th>Electrical Resistivity (ohm.cm)</th>
<th>18 month Corrosion Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>5,160</td>
<td>0</td>
<td>3,663</td>
<td>8</td>
<td>-456</td>
</tr>
<tr>
<td>7,360</td>
<td>15</td>
<td>198</td>
<td>95</td>
<td>-26</td>
</tr>
<tr>
<td>8,580</td>
<td>15</td>
<td>98</td>
<td>118</td>
<td>-53</td>
</tr>
<tr>
<td>9,290</td>
<td>8</td>
<td>132</td>
<td>74</td>
<td>-3</td>
</tr>
<tr>
<td>12,120</td>
<td>15</td>
<td>75</td>
<td>161</td>
<td>+53</td>
</tr>
</tbody>
</table>

† By weight of portland cement.
* Measured by AASHTO-T-227 rapid chloride permeability test at 28 days age.
$ Corrosion potential measured with respect to a copper sulfate reference electrode.
$ $R_p$ is the polarization current; its reciprocal is a measure of the rate of corrosion.

Concrete with unit weights of about 112 lb/ft³ (1794 kg/m³) with 56-day compressive strength of 9000 psi (62 MPa) and about 6500 yd³ (5000 m³) of high-strength normal-weight concrete with unit weights of about 145 lb/ft³ (2323 kg/m³) and 56-day compressive strengths of about 10,000 psi (69 MPa).

Field placements of high-strength, low-permeability, and chemical-resistant concretes for industrial manufacturing applications were reported by Wolsiefer.8.20 Special applications have included several modular bank vaults placed at slumps of 9 in. (230 mm) with measured compressive strengths of 12,000 psi (83 MPa) at 45 days.

The protection of reinforcing steel from corrosion can be expected to be enhanced when high-strength concrete is used. The resultant low porosity should increase the electrical resistivity and reduce the rate at which oxygen reaches the steel, both of which will reduce corrosion rates. Moreover, the ease with which chloride ions from deicing salts can reach the steel and initiate corrosion is also reduced.

Although there are many studies evaluating the corrosion of steel embedded in regular strength concrete, no systematic studies in the influence of concrete strength appear to have been reported. Published data for high-strength concrete can be extracted from studies investigating other factors, particularly the influence of silica fume. The conclusions obtained with regular concretes are also applicable to high-strength concrete; namely, there is an increase in electrical resistivity8.28,8.29 and a reduction in chloride permeability8.29,8.30 with increased strength. Data linking these parameters with laboratory corrosion data are given in Table 8.3. The corrosion behavior of a very high-strength mortar has also been reported.8.31 Useful discussions regarding the factors affecting the corrosion of steel in concretes with silica fume are to be found in references.8.31,8.32,8.33

8.5-- Potential applications

Most applications of high-strength concrete have used the strength property of the material. However, high-strength concrete may possess other characteristics that could be used advantageously in concrete structures.

LeMessurier8.21 proposed the use of high-strength concrete to satisfy the need for a high modulus of elasticity. Similarly, high-strength concrete can be used in slabs to allow early removal of formwork and avoid reshoring.8.22 This takes advantage of both the high modulus of elasticity and lower creep of high-strength concrete. Anderson8.19 had suggested that the low creep of high-strength concrete should be taken into account when considering prestress losses. Since most of the prestress loss is attributable to creep and shrinkage, prestress losses for high-strength concrete members should be less than for lower-strength concrete members.

Rabbat and Russell8.23 have reported that the maximum span capability of solid-section girders can be increased by 15 percent when the concrete compressive strength is increased from 5000 to 7000 psi (34 to 48 MPa). Finally, Manning8.24 has suggested that the relationship between high-strength concrete and high-quality concrete may make high-strength concrete attractive not for its strength but for its long-term service performance.

More recently, high-strength concrete has been specified for applications in warehouses, foundries, parking garages, bridge deck overlays, dam spillways, and heavy duty industrial floors. In these applications, high-strength concrete is being used to provide a concrete with improved resistance to chemical attack, better abrasion resistance, improved freeze-thaw durability, and reduced permeability.

8.6--Cited references

(See also Chapter 10-References)


8.4. Pickard, Scott S., "Ruptured Composite Tube


8.6. Venema, T.P., and Regnier, H.J., “Placement, Batching, and Tests of High Strength Concrete for Minneapolis City Center Project,” submitted to ACI for publication.


CHAPTER 9-- SUMMARY

The objective of this report was to present state-of-the-art information on concrete with strengths in excess of about 6000 psi (41 MPa) but not including concrete made using exotic materials or techniques. This section of the report presents a summary of the material contained in the previous chapters.

All materials for use in high-strength concrete must be carefully selected using all available techniques to insure uniform success. Items to be considered in selecting materials include cement characteristics, aggregate size, aggregate strength, particle shape and texture, and the effects of set-controlling admixtures, water reducers, silica fume, and pozzolans. Trial mixtures are essential to insure that required concrete strengths will be obtained and that all constituent materials are compatible.

Mix proportions for high-strength concrete generally have been based on achieving a required compressive strength at a specified age. Depending on the appropriate application, a specified age other than 28 days has been used. Factors included in selecting concrete mix proportions have included availability of materials, desired workability, and effects of temperature rise. All materials must be optimized in concrete mix proportioning to achieve maximum strength. High-strength concrete mixes have usually used high cement contents, low water-cement ratios, normal weight aggregate, and chemical and pozzolanic admixtures. Required strength, specified age, material characteristics, and type of application have strongly influenced mix design. High-strength concrete mix proportioning has been found to be a more critical process than the proportioning of lower-strength concrete mixes. Laboratory trial batches have been required in order to generate necessary data on mix design. In many cases, laboratory mixes have been followed by field production trial batches.

Batching, mixing, transporting, placing, and control procedures for high-strength concrete are not essentially different from procedures used for low-strength concretes. However, special attention is required to insure a high-strength uniform material. Special consideration should be given to minimizing the length of time between concrete batching and final placement in the forms. Delay in concrete placement can result in a subsequent loss of long-term strength or difficulties in concrete placement. Special attention should also be paid to the testing of high-strength concrete cylinders since any deficiency will result in an apparent lower strength than that actually achieved by the concrete. Items deserving specific attention include manufacture, curing, and capping of control specimens for compressive strength measurements; characteristics of testing machines; type of mold used to produce specimens; and age of testing. In many instances, strength measurements at early ages have been made even though the compressive strength has not been specified until 56 or 90 days.

Some research data have indicated that the modulus of elasticity of high-strength concrete is lower than would have been predicted from data on lower-strength concretes. However, values of Poisson’s ratio appear to be in the expected range, based on lower-strength concretes. The modulus of rupture for high-strength concretes is higher than would have been anticipated. However, the tensile splitting strength values appear to be consistent with lower-strength concretes. Unit weight, specific heat, diffusivity, thermal conductivity, and coefficient of thermal expansion have been found to fall generally within the usual range for lower-strength concretes. High-strength concrete has shown a higher rate of strength gain at early ages as compared to lower-strength concrete, but at later ages the difference is not significant. Information on creep and shrinkage of high-strength concrete has indicated that the shrinkage is similar to that for lower-strength concrete. However, specific creep is much less for high-strength concretes than for lower-strength concretes.

In the area of structural design, it has been found that axially loaded columns with high-strength concrete can be designed in the same way as lower-strength columns. It has also been identified that high-strength concrete columns exhibit less shortening under load than lower-strength columns because of the higher modulus of elasticity and lower creep coefficients. For beams, use of the conventional equivalent rectangular stress block appears to give satisfactory results for under-reinforced concrete members. The compressive strain limit of 0.003 appears to be acceptable. However, changes have been recommended for present code values for minimum tensile steel ratio, modulus of rupture, modulus of elasticity, shear strength, and development length. Changes are also needed in the area of calculating long-term beam deflections.

The economic advantages of using high-strength concrete in the columns of high-rise buildings have been clearly demonstrated by applications in many cities. The ability to reduce the amount of reinforcing steel in columns without sacrificing strength and to keep, the columns to an acceptable size has been an economic benefit to owners of high-rise buildings. Consequently, concrete with compressive strengths in excess of 6000 psi (41 MPa) has been used in the columns of high-rise buildings in cities throughout North America. Studies have also indicated advantages in the use of high-strength concrete in long-span concrete bridges. However, this application has yet to be fully implemented. There have also been applications where high-compressive-strength concrete has been needed to satisfy special local requirements. These include dams, prestressed concrete structures, grandstand roofs, marine foundations, parking garages, bridge deck overlays, heavy duty industrial floors, and industrial manufacturing applications.

Although high-strength concrete is often considered a relatively new material, it is becoming accepted in more parts of North America as shown by the many examples of its usage. At the same time, material producers are
responding to the demands for the material and are learning production techniques. As with many developments of new materials, research data supporting the growth has also increased. However, the need for additional research has been documented in ACI 363.1R. Some research projects are underway to satisfy these needs. However, further work is needed to fully use the advantages of high-strength concrete and to affirm its capabilities. This report has documented existing knowledge of high-strength concrete so that the direction for future development may be ascertained.

CHAPTER 10-REFERENCES

10.1-Recommended references

The documents of the various standards-producing organizations referred to in this document are listed below with their serial designation.

American Association of State Highway and Transportation Officials
T-26 Quality of Water to be Used in Concrete

American Concrete Institute
116R Cement and Concrete Terminology
201.1R Guide for Making a Condition Survey of Concrete in Service
211.1 Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete
212.2R Guide for Use of Admixtures in Concrete
214 Recommended Practice for Evaluation of Strength Test Results of Concrete
304 Guide for Measuring, Mixing, Transporting, and Placing Concrete
304.4R Placing Concrete with Belt Conveyors
305R Hot Weather Concreting
308 Standard Practice for Curing Concrete
309 Guide for Consolidation of Concrete
318 Building Code Requirements for Reinforced Concrete
318R Commentary on Building Code Requirements for Reinforced Concrete

American Society for Testing and Materials
C 31 Standard Method of Making and Curing Concrete Test Specimens in the Field
C 33 Standard Specification for Concrete Aggregates
C 39 Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens
C 94 Standard Specification for Ready-Mixed Concrete
C 109 Standard Test Method for Compressive Strength of Hydraulic Cement, Mortars (using 2 in. or 50 mm cube specimens)
C 143 Standard Test Method for Slump of Portland Cement Concrete
C 150 Standard Specification for Portland Cement
C 192 Standard Method of Making and Curing Concrete Test Specimens in the Laboratory
C 260 Standard Specification for Air-Entraining Admixtures for Concrete
C 311 Standard Methods of Sampling and Testing Fly Ash or Natural Pozzolans for Use as a Mineral Admixture in Portland Cement Concrete
C 494 Standard Specification for Chemical Admixtures for Concrete
C 595 Standard Specification for Blended Hydraulic Cements
C 618 Standard Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete
C 684 Standard Method of Making, Accelerated Curing, and Testing of Concrete Compression Test Specimens
C 917 Standard Method for Evaluation of Cement Strength Uniformity from a Single Source
E 329 Standard Recommended Practice for Inspection and Testing Agencies for Concrete, Steel, and Bituminous Materials as Used in Construction

Canadian Standards Association
A 266.5-M1981 Guidelines for the Use of Supersuspending Admixtures in Concrete

Concrete Plant Manufacturers Bureau
Concrete Plant Manufacturers Standards of the Plant Mixer Manufacturers Division

The above publications may be obtained from the following organizations:

American Association of State Highway and Transportation Officials
333 N Capitol St. N.W.
Suite 225
Washington, D.C. 20001

American Concrete Institute
P.O. Box 19150
Detroit, MI 48219

American Society for Testing and Materials
1916 Race Street
Philadelphia, PA 19103

Canadian Standards Association
178 Rexdale Blvd.
Rexdale, Ont.
Canada M9W 1R3
Concrete Plant Manufacturers Bureau
900 spring St.
Silver Spring, Md. 20910

10.2-- Cited references
Cited references are provided at the end of each chapter.

10.3-- Bibliography
The purpose of this bibliography is to call attention to literature on high-strength concrete in addition to that listed at the ends of chapters in this report. The entries are organized alphabetically by author. Anonymous references are listed alphabetically according to their titles.


10.9 Bennett, E.W., “Fatigue in Concrete,” Concrete (London), May 1974, pp. 43-45.


10.24. Chernobaev, V.I., “Investigation of the Carrying Capacity of High Strength Concrete Flexible Columns (Issledovanie Nesushchih sposobnostei Gibrikh Kolonn...


10.28. Collepardi, Mario, and Corradi, Mario, “Influence of Naphthalene-Sulfonated Polymer Based Superplasticizers on the Strength of Ordinary and Lightweight Concretes,” Superplasticizers in Concrete, SP-62, American Concrete Institute, Detroit, 1979, pp. 315-336.


10.47. Funakoshi, M., and Okamoto, T., “The Shear Strength of Prestressed Beams for which Very High Strength Concrete is Employed,” Transactions, Japan Concrete Institute, Tokyo, V. 2, 1980, pp. 271-278.


10.60. Hattori, Kenichi, “Experiences with Mighty Superplasticizer in Japan,” Superplasticizers in Concrete, SP-62, American Concrete Institute, Detroit, 1979, pp. 37-66.


10.84. Lobanov, A.T., et al., “Practice of Prefabrication of High-Strength Concrete Columns for Buildings (Opyt Izgotovleniya Kolonn Iz Vysokoprochnyky Betonov Dlya Zhilykh Domov),” Beton i Zhelezobeton (Moscow), No.


10.93. Mather, Bryant, “Tests of High-Range Water-Reducing Admixtures,” Superplasticizers in Concrete, SP-62, American Concrete Institute, Detroit, 1979, pp. 157-166.


10.113. Nishi, H.; Ohshio, A.; and Fukuzawa, K.,


10.139. Shah, S.P.; Gokoz, Ulker; and Ansari, Farhad, “Experimental Technique for Obtaining Complete Stress-Strain Curves for High Strength Concrete,” Cement, Concrete, and Aggregates, V. 3, No. 1, Summer 1981, pp. 21-27.


10.143. “Soviet Concrete Not So Tough?,” Engineering News-Record, V. 166, No. 22, June 1, 1961, p. 47.
10.147. Superplasticizers in Concrete, SP-62, American Concrete Institute, Detroit, 1979, 436 pp.