ACI 363R-10

# Report on High-Strength Concrete

Reported by ACI Committee 363

American Concrete Institute®

First Printing
March 2010

American Concrete Institute®
*Advancing concrete knowledge*

## Report on High-Strength Concrete



**Order information:** ACI documents are available in print, by download, on CD-ROM, through electronic subscription, or reprint and may be obtained by contacting ACI.

Most ACI standards and committee reports are gathered together in the annually revised *ACI Manual of Concrete Practice* (MCP).

**American Concrete Institute**
**38800 Country Club Drive**
**Farmington Hills, MI 48331**
**U.S.A.**
**Phone: 248-848-3700**
**Fax: 248-848-3701**

**www.concrete.org**

ISBN 978-0-87031-254-0

# Report on High-Strength Concrete

## Reported by ACI Committee 363


Michael A. Caldarone
Chair

John J. Myers
Secretary

| | | | |
|---|---|---|---|
| Ronald G. Burg | William M. Hale | Jaime Morenco | Robert C. Sinn |
| James E. Cook | Jerry S. Haught | Charles K. Nmai | Peter G. Snow |
| Daniel Cusson | Tarif M. Jaber | Clifford R. Ohlwiler | Konstantin Sobolev |
| Per Fidjestøl | Daniel C. Jansen | Michael F. Pistilli | Houssam A. Toutanji |
| Seamus F. Freyne | Anthony N. Kojundic | William F. Price | Dean J. White II |
| Brian C. Gerber | Federico Lopez Flores | Henry G. Russell | John T. Wolsiefer Sr. |
| Shawn P. Gross | Mark D. Luther | Michael T. Russell | Paul Zia |
| Neil P. Guptill | Barney T. Martin Jr. | Ava Shypula | |



*This report summarizes currently available information about high-strength concrete (HSC). Topics discussed include selection of materials, concrete mixture proportions, ordering, batching, mixing, transporting, placing, quality control, concrete properties, structural design, economic considerations, and applications.*



**Keywords:** concrete properties; economic considerations; high-strength concrete; material selection; mixture proportions; structural applications; structural design; quality control.




**CONTENTS**




ACI 363R-10 supersedes ACI 363R-92 and was adopted and published March 2010.

# CHAPTER 1—INTRODUCTION

## 1.1—Historical background

The use and definition of high-strength concrete (HSC) has seen a gradual and continuous development over many years. In the 1950s, concrete with a compressive strength of 5000 psi (34 MPa) was considered high strength. In the 1960s, concrete with compressive strengths of 6000 and 7500 psi (41 and 52 MPa) were produced commercially. In the early 1970s, 9000 psi (62 MPa) concrete was produced. Today, compressive strengths approaching 20,000 psi (138 MPa) have been used in cast-in-place buildings. Laboratory researchers using special materials and processes have achieved "concretes" with compressive strengths in excess of 116,000 psi (800 MPa) (Schmidt and Fehling 2004). As materials technology and production processes evolve, it is likely the maximum compressive strength of concrete will continue to increase and HSC will be used in more applications.

Demand for and use of HSC for tall buildings began in the 1970s, primarily in the U.S.A. Water Tower Place in Chicago, IL, which was completed in 1976 with a height of 859 ft (260 m) and used 9000 psi (62 MPa) specified compressive strength concrete in the columns and shear walls. The 311 South Wacker building in Chicago, completed in 1990 with a height of 961 ft (293 m), used 12,000 psi (83 MPa) specified compressive strength concrete for the columns. In their time, both buildings held the record for the world's tallest concrete building. Two Union Square in Seattle, WA, completed in 1989, holds the record for the highest specified compressive strength concrete used in a building at 19,000 psi (131 MPa).

High-strength concrete is widely available throughout the world, and its use continues to spread, particularly in the Far East and Middle East. All of the tallest buildings constructed in the past 10 years have some structural contribution from HSC in vertical column and wall elements. The world's tallest building, at 1670 ft (509 m), is Taipei 101 in Taiwan, completed in 2004. The structural system uses a mix of steel and concrete elements, with specified concrete compressive strengths up to 10,000 psi (69 MPa) in composite columns. Petronas Towers 1 and 2, completed in 1998 in Kuala Lumpur, Malaysia, used concrete with specified cube strengths up to 11,600 psi (80 MPa) in columns and shear walls. At the time of this report, these towers are the second and third tallest buildings in the world, both at 1483 ft (452 m). The world's tallest building constructed entirely with a reinforced concrete structural system is the CITIC Plaza building in Guangzhou, People's Republic of China, with a height of 1283 ft (391 m). Trump World Tower in New York City, reportedly the world's tallest residential building at 861 ft (262 m) and completed in 2001, is constructed using a concrete system alone with columns having specified compressive strengths up to 12,000 psi (83 MPa). In 2005, construction began on Burj Dubai tower in Dubai, UAE. With a height exceeding 1969 ft (600 m), this all-concrete residential structure, scheduled for completion in 2009, will use concrete with specified cube strengths up to 11,600 psi (80 MPa).

The use of HSC in bridges began in the U.S. in the mid-1990s through a series of demonstration projects. The highest specified concrete compressive strength is 14,700 psi (101 MPa) for prestressed concrete girders of the North Concho River Overpass in San Angelo, TX. High-strength concrete has also been used in long-span box-girder bridges and cable-stayed bridges. There are also some very significant applications of HSC in offshore structures. These include projects such as the Glomar Beaufort Sea I drilling structure, the Heidrun floating platform in the North Sea, and the Hibernia offshore concrete platform in Newfoundland, Canada. In many offshore cases, HSC is specified because of the harsh environments in which these structures are located (Kopczynski 2008).

## 1.2—Definition of high-strength concrete

In 2001, Committee 363 adopted the following definition of HSC:

**concrete, high-strength**—concrete that has a specified compressive strength for design of 8000 psi (55 MPa) or greater.

When the original version of this report was produced in 1992, ACI Committee 363 adopted the following definition of HSC:

**concrete, high-strength**—concrete that has a specified compressive strength for design of 6000 psi (41 MPa) or greater.

The new value of 8000 psi (55 MPa) was selected because it represented a strength level at which special care is required for production and testing of the concrete and at which special structural design requirements may be needed. As technology progresses and the use of concrete with even higher compressive strengths evolves, it is likely that the definition of high-strength concrete will continue to be revised.

Although 8000 psi (55 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 8000 psi (55 MPa) represent a process that starts with the lower-strength concretes and continues into higher-strength concretes. Many empirical equations used to predict concrete properties or to design structural members are based on tests using concrete with compressive strengths of 8000 to 10,000 psi (55 to 69 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 empirical relationships from lower-strength to higher-strength concretes. If necessary, tests should be made to develop relationships for the materials or applications in question.

The committee also recognized that the definition of HSC varies on a geographical basis. In regions where concrete with a compressive strength of 9000 psi (62 MPa) is already being produced commercially, HSC might range from 12,000 to 15,000 psi (83 to 103 MPa) compressive strength. 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 mixture proportioning, batching, mixing, transporting, placing, curing, and quality-control procedures are applicable across a wide range of concrete strengths. The committee agreed, however, that material properties and structural design considerations given in this report should be concerned with concretes having high compressive strengths. The committee has tried to cover both aspects in developing this report.

## 1.3—Scope of report

Because the definition of HSC has changed over the years, the following scope was adopted by Committee 363 for this report: "The immediate concern of Committee 363 shall be concretes with specified compressive strengths for design of 8000 psi (55 MPa) or greater, but for the present time, considerations shall not include concrete made using exotic materials or techniques." The word "exotic" was included so that the committee would not be concerned with concretes such as polymer-impregnated concrete, epoxy concrete, ultra-high-performance concrete; concrete with artificial, normal, and heavyweight aggregates; and reactive powder concrete. In addition to focusing on concretes made with nonexotic materials or techniques, the committee also attempted to focus on concretes that were commercially viable rather than concretes that have only been produced in the laboratory.

# CHAPTER 2—NOTATION, DEFINITIONS, AND ACRONYMS

## 2.1—Notation

$A_b$ = area of a single spliced bar (or wire), in.$^2$ (mm$^2$)

$A_{cp}$ = area enclosed by outside perimeter of concrete cross section, in.$^2$ (mm$^2$)

$A_g$ = gross area of concrete section, in.$^2$ (mm$^2$). For a hollow section, $A_g$ is the area of concrete only and does not include the area of the void(s)

$A_s$ = area of nonprestressed longitudinal tension reinforcement, in.$^2$ (mm$^2$)

$A_{sp}$ = area of transverse reinforcement crossing the potential plane of splitting through the reinforcement being developed, in.$^2$ (mm$^2$)

$A_{st}$ = total area of nonprestressed longitudinal reinforcement, in.$^2$ (mm$^2$)

$A_{tr}$ = total cross-sectional area of all transverse reinforcement with spacing $s$ that crosses the potential plane of splitting through the reinforcement being developed, in.$^2$ (mm$^2$)

$A_{Vmin}$ = minimum area of shear reinforcement within spacing $s$, in.$^2$ (mm$^2$)
$B$ = width of compression face of member, in. (mm)
$b$ = width of the cross section, in. (mm)
$b_w$ = web width, or diameter of circular section, in. (mm)
$C_c$ = creep coefficient
$D$ = distance from extreme compression fiber to centroid of longitudinal reinforcement, in. (mm)
$d$ = distance from extreme compression fiber to centroid of tension reinforcement, in. (mm)
$E_c$ = modulus of elasticity of concrete, psi (MPa)
$f_2'$ = concrete confinement stress produced by spiral, psi (MPa)
$f_c'$ = specified compressive strength of the concrete, psi (MPa)
$\bar{f}_c$ = compressive strength of spirally reinforced concrete column, psi (MPa)
$f_c''$ = compressive strength of unconfined concrete column, psi (MPa)
$f_{cr}'$ = required average compressive strength of concrete used as the basis for selection of concrete proportions, psi (MPa)
$f_r$ = modulus of rupture of concrete, psi (MPa)
$f_{sp}$ = splitting cylinder strength of concrete, psi (MPa)
$f_y$ = specified yield strength of reinforcement, psi (MPa)
$I_{cr}$ = moment of inertia of cracked transformed to concrete, in.$^4$ (mm$^4$)
$I_g$ = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, in.$^4$ (mm$^4$)
$k_1$ = ratio of average to maximum compressive stress in beam
$k_2$ = ratio of depth to compressive resultant to neutral axis depth
$k_3$ = ratio of maximum stress in beam to maximum stress in corresponding axially loaded cylinder
$M_a$ = maximum moment in member due to service loads at stage deflection is computed, in.-lb (N·mm)
$M_{cr}$ = cracking moment, in.-lb (N·mm)
$M_n$ = nominal flexural strength at section, in.-lb (N·mm)
$M_u$ = factored moment at section, in.-lb (N·mm)
$n$ = number of spliced bars ($n$ = 1 for a single bar)
$s_s$ = sample standard deviation, psi (MPa)
$T_{cr}$ = cracking torsional moment, in.-lb (N·mm)
$V_c$ = nominal shear strength provided by concrete, lb (N)
$V_u$ = factored shear force at section, lb (N)
$w_c$ = unit weight of normalweight concrete or equilibrium density of lightweight concrete, lb/ft$^3$ (kg/m$^3$)
$w/cm$ = water-cementitious material ratio
$\alpha_1$ = stress block parameter as defined in Fig. 7.2
$\beta_1$ = factor relating depth of equivalent rectangular compressive stress block to neutral axis depth
$\delta_c$ = specific creep (unit creep coefficient)
$\Delta_u$ = beam deflection at failure load, in. (mm)
$\Delta_y$ = beam deflection at the load producing yielding of tensile steel, in. (mm)
$\varepsilon_{initial}$ = initial strain upon application of load, in./in. (mm/mm)
$\varepsilon_{creep}$ = additional time-dependent strain due to creep, in./in. (mm/mm)
$\lambda_\Delta$ = multiplier for additional deflection due to long-term effects
$\mu$ = ductility index
$\xi$ = time-dependent factor for sustained load taken from ACI 318
$\sigma_{initial}$ = initial stress due to sustained load, psi (MPa)
$\rho'$ = reinforcement ratio for non-prestressed compression reinforcement; ratio of $A_s'$ to $bd$
$\rho_{cp}$ = outside perimeter of concrete cross section
$\rho_{min}$ = minimum reinforcement ratio; ratio of $A'_{s\,min}$ to $bd$
$\rho_s$ = ratio of volume of spiral reinforcement to total volume of concrete core confined by the spiral (measured out-to-out of spirals)
$\psi_u$ = cross-section curvature at failure load
$\psi_y$ = cross-section curvature at the load producing yielding of tensile steel
$\omega$ = tension reinforcement index

## 2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource, "ACI Concrete Terminology" (http://terminology.concrete.org) (American Concrete Institute 2009). Definitions provided here complement that resource.

**admixture**—a material other than water, aggregates, hydraulic cement, and fiber reinforcement, used as an ingredient of a cementitious mixture to modify its freshly mixed, setting, or hardened properties and that is added to the batch before or during its mixing.

**admixture, air-entraining**—an admixture that causes the development of a system of microscopic air bubbles in concrete, mortar, or cement paste during mixing, usually to increase its workability and resistance to freezing and thawing.

**admixture, water-reducing (high-range)**—a water-reducing admixture capable of producing large water reduction or great flowability without causing undue set retardation or entrainment of air in mortar or concrete.

**aggregate**—granular material, such as sand, gravel, crushed stone, crushed hydraulic-cement concrete, or iron blast-furnace slag, used with a hydraulic cementing medium to produce either concrete or mortar.

**concrete, high-strength**—concrete that has a specified compressive strength for design of 8000 psi (55 MPa) or greater.

**creep**—time-dependent deformation due to sustained load.

**heat of hydration**—heat evolved by chemical reactions with water, such as that evolved during the setting and hardening of portland cement, or the difference between the heat of solution of dry cement and that of partially hydrated cement.

**materials, cementitious**—pozzolans and hydraulic cements used in concrete and masonry construction.

**modulus of elasticity**—the ratio of normal stress to corresponding strain for tensile or compressive stress below the proportional limit of the material; also referred to as elastic modulus, Young's modulus, and Young's modulus of elasticity; denoted by the symbol $E$.

**modulus of rupture**—a measure of the load-carrying capacity of a beam and sometimes referred to as rupture modulus or rupture strength; it is calculated for apparent tensile stress in the extreme fiber of a transverse test specimen under the load that produces rupture.

**permeability to water, coefficient of**—the rate of discharge of water under laminar flow conditions through a unit cross-sectional area of a porous medium under a unit hydraulic gradient and standard temperature conditions, usually 70°F (20°C).

**ratio, Poisson's**—the absolute value of the ratio of transverse (lateral) strain to the corresponding axial (longitudinal) strain resulting from uniformly distributed axial stress below the proportional limit of the material; the value will average approximately 0.2 for concrete and 0.25 for most metals.

**resistance, abrasion**—ability of a surface to resist being worn away by rubbing and friction.

**resistance, fire**—the property of a material or assembly to withstand fire or give protection from it; as applied to elements of buildings, it is characterized by the ability to confine a fire or, when exposed to fire, to continue to perform a given structural function, or both.

**scaling**—local flaking or peeling away of the near-surface portion of hardened concrete or mortar; also peeling or flaking of a layer from metal.

**shrinkage**—decrease in either length or volume. Note: may be restricted to the effects of moisture content or chemical changes.

**strength, fatigue**—the greatest stress that can be sustained for a given number of stress cycles without failure.

**strength, splitting tensile**—tensile strength of concrete determined by a splitting tensile test.

**quality assurance**—actions taken by an organization to provide and document assurance that what is being done and what is being provided are in accordance with the contract documents and standards of good practice for the work.

**quality control**—actions taken by an organization to provide control and documentation over what is being done and what is being provided so that the applicable standard of good practice and the contract documents for the work are followed.

**water-cement ratio**—the ratio of the mass of water, exclusive only of that absorbed by the aggregates, to the mass of portland cement in concrete, mortar, or grout, stated as a decimal and abbreviated as *w/c*. (See also **water-cementitious material ratio**.)

**water-cementitious material ratio**—the ratio of the mass of water, exclusive only of that absorbed by the aggregate, to the mass of cementitious material (hydraulic) in concrete, mortar, or grout, stated as a decimal and abbreviated as *w/cm*. (See also **water-cement ratio**.)

## 2.3—Acronyms

CCHRB Chicago Committee on High-Rise Buildings
CSH calcium silicate hydrate
CTE coefficient of thermal expansion
FHWA Federal Highway Administration
HRM high-reactivity metakaolin
HRWRA high-range water-reducing admixture
HSC high-strength concrete
MRWRA mid-range water-reducing admixture
SCM supplementary cementitious material

# CHAPTER 3—SELECTION OF MATERIAL

## 3.1—Introduction

Producing high-strength concrete (HSC) that consistently meets requirements for workability and strength development places stringent requirements on material selection compared with conventional concretes. Quality materials are needed, and specifications require enforcement. High-strength concrete has been produced using a wide range of constituent materials. Trial batching, in both the laboratory and field, is necessary to assess the quality and suitability of constituent materials in HSC. This chapter cites the state of knowledge regarding material selection and provides a baseline for the subsequent discussion of mixture proportions in Chapter 4.

## 3.2—Cementitious materials

**3.2.1** *Portland cement*—Portland cement is by far the most widely used type of cement in the manufacture of hydraulic-cement concrete, and HSC is no exception. The choice of portland cement for HSC is extremely important (Freedman 1971; Hester 1977). Portland cement for use in HSC should be selected based on performance needs. For example, unless high early strength is required, such as in prestressed concrete, there is no need to use high-early-strength portland cement, such as ASTM C150/C150M Type III. Furthermore, because of the significant variations in properties that are permitted in cement specifications within a given cement type, different brands of cement will have different strength development characteristics. Differences in compressive strength among mixtures containing different cements are more pronounced at an age of 1 day than at 56 days (Myers and Carrasquillo 1998). Also, cement characteristics will generally have a larger influence on compressive strength than modulus of elasticity (Freyne et al. 2004).

Initially, manufacturers' mill certificates for the previous 6 to 12 months should be obtained from potential suppliers. This will give an indication of strength characteristics from ASTM C109/C109M mortar cube tests, and more importantly, it will provide an indication of cement uniformity. The cement supplier should be required to report uniformity in accordance with ASTM C917. Variations in chemical and physical properties over time should be tightly controlled. For example, in the case of a portland cement, if the tricalcium silicate content varies by more than 4%, the ignition loss by more than 0.5%, or the fineness by more than 171 $ft^2/lb$ (35 $m^2/kg$) (Blaine), then objectionable variability in strength performance may result (Hester 1977). Sulfur trioxide ($SO_3$) levels should not vary by more than ±0.20 percentage points from that in the cement used for the mixture development process.

Although mortar cube tests can be a good indicator of potential strength, mortar cube test results alone should not be the sole basis for selecting cement for use in concrete, particularly in HSC. A reliable estimate of cement performance in HSC can be achieved by assessing the cements' normal consistency and setting times along with cube strength (ASTM C191; ASTM C109/C109M). Concrete tests, however, should be run on trial batches of concrete made with proposed aggregates, supplementary cementitious materials (SCMs), and chemical admixtures, and evaluated under simulated job conditions. Unless the objective is only to achieve high early strength, in most cases, strengths should be determined through at least 56 days. The effect of cementitious material characteristics on water demand is more pronounced in HSCs because of higher cementitious materials contents and low water-cementitious material ratios (*w/cm*).

The type and amount of cementitious materials in a HSC mixture can have a significant effect on temperature development within the concrete. For example, the Chicago Committee on High-Rise Buildings (CCHRB 1997) reported that the temperature in the 4 ft (1.2 m) square columns used in Water Tower Place, which had a cement content of 846 lb/yd$^3$ (502 kg/m$^3$), rose to 150 from 75°F (66 from 24°C) during hydration. The heat was dissipated within 6 days without harmful effects. When temperature rise is expected to be a problem, however, slower-reacting, low-heat-of-hydration materials, such as Type II portland cement, SCMs such as slag or Class F fly ash, or blended hydraulic cements incorporating slag or Class F fly ash can be used provided they meet strength and heat of hydration requirements. Additional practices that can alleviate problems associated with temperature rise and related hot weather conditions are discussed in ACI 305R.

A further consideration is optimization of the cement-admixture system. Optimization in terms of the balance of cement and admixtures is the level at which the cement, cementitious admixtures, and chemical admixtures are minimized from a cost perspective. The exact effect of a water-reducing chemical admixture on water requirement, for example, will depend on cement characteristics. Strength development depends on both the characteristics of the cementitious materials and the *w/cm* (ACI 211.4R).

**3.2.2** *Supplementary cementitious materials*—In the past, fly ash, silica fume, and natural pozzolans were frequently called mineral admixtures. In North America today, these materials and others, such as slag cement, are now covered under the term "supplementary cementitious materials" (SCMs). Supplementary cementitious materials for use in concrete are materials that have mineral oxides similar to those found in portland cement, but in different proportions and possibly different mineral phases. Supplementary cementitious materials are widely used in the production of HSC because their presence alters the mineral constituents in the binding (paste) system to allow attainment of high strengths.

Supplementary cementitious materials consisting of certain pozzolans or slags are extremely well-suited for use in HSC. Supplementary cementitious materials can be predominantly hydraulic, pozzolanic, or possess properties of both a hydraulic and pozzolanic material. Similar to portland cement, hydraulic SCMs set and harden when in contact with water. Pozzolans are siliceous or siliceous and aluminous materials that, by themselves, possess little or no cementitious value. In finely divided form and in the presence of moisture, however, they will chemically react with calcium hydroxide released by cement hydration to form additional calcium silicate hydrate (CSH) gel, the glue that binds aggregate particles together. In addition to the pozzolanic effect, some SCMs improve the particle packing of the binder system (Brewe and Myers 2005).

With a good understanding of their individual properties and an understanding of how these materials interact with the other mixture constituents (ACI 232.2R; ACI 233R; ACI 234R), appropriate use of SCMs can significantly improve strength in concrete, particularly HSC. In fact, without their use, achieving extremely high strength levels that are routinely available in many construction markets would be significantly more difficult, if not impossible. In many cases, workability, pumpability, finishability, durability, and economy can also be improved through the proper use of these materials.

It is important that all cementitious materials be tested for acceptance and uniformity, and carefully investigated for strength-producing properties and compatibility with the other materials in the mixture, particularly chemical admixtures, before they are used in the work.

**3.2.2.1** *Fly ash*—Specifications for fly ash are covered in ASTM C618. There are two fly ash classifications: Class F and Class C. Class F fly ash is normally produced from burning anthracite or bituminous coal and has strong pozzolanic properties, but little or no hydraulic properties. Class C fly ash is normally produced from burning lignite or sub-bituminous coal, and in addition to having pozzolanic properties, has some hydraulic properties. The major difference between these two classes of fly ash is the amounts of silicon dioxide (silica) and calcium oxide they contain. Class C fly ash, having an abundance of both silica and calcium oxide, is capable of producing CSH when it alone comes into contact with water. Class F fly ash, though high in silica, lacks a sufficient quantity of calcium oxide to produce CSH when it alone comes into contact with water. Class C fly ash is more reactive than Class F fly ash. In general, Class F fly ash has been used predominantly in the eastern and western regions of the U.S. and Canada, and Class C fly ash has been used mostly in the Midwestern and South Central regions of the U.S. (ACI 232.2R).

In addition to its chemical and physical properties and how it interacts with admixtures and other cementitious materials in the mixture, the optimum quantity of fly ash in a HSC depends to a large extent on the target strength level and the age at which strength is desired. For example, the optimum quantity of a Class C fly ash in conventional concrete having a specified compressive strength of 4000 psi (28 MPa) at 28 days and containing 450 lb/yd$^3$ (225 kg/m$^3$) of cementitious material might be 25% (by mass) of the cementitious material content. In a concrete having a specified compressive strength of 10,000 psi (69 MPa) at 56 days and containing 900 lb/yd$^3$ (450 kg/m$^3$) of cementitious material, the optimum quantity of the same fly ash might be 40% or more (Caldarone 2008).

Methods for sampling and testing fly ash are given in ASTM C311 and C618. Variations in chemical or physical properties, although within the tolerances of these specifications, may cause appreciable variations in HSC properties. Such variations can only be minimized by changes in the coal burning and fly ash collection process employed at the power plant.

**3.2.2.2** *Silica fume*—Silica fume has been used in structural concrete and repair applications where high strength, low permeability, or high abrasion resistance are advantageous. Major advancements in the areas of high-strength and high-performance concrete have been largely possible through the use of silica fume. 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 high amorphous silicon dioxide content and consists of very fine spherical particles, is collected from the gases escaping the furnaces. Specifications for silica fume are covered in standards, such as ASTM C1240 and EN 13263.

Silica fume is composed mostly of amorphous silica particles, and its specific gravity is expected to be approximately 2.20, the most commonly accepted value for amorphous silica (Malhotra et al. 2000). ELKEM (1980) reported the specific surface area of silica fume is on the order of 88,000 to 107,500 $ft^2/lb$ (18,000 to 22,000 $m^2/kg$) when measured by nitrogen adsorption techniques. Nebesar and Carette (1986) reported an average value of 97,700 $ft^2/lb$ (20,000 $m^2/kg$). Particle-size distribution of typical silica fume shows most particles are smaller than 1 micrometer (1 μm), with the majority being on the order of 0.1 to 0.3 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 $lb/ft^3$ (160 to 320 $kg/m^3$). Silica fume for commercial applications is available in either densified or slurry form. Silica fume in slurry form, however, is not readily available in some markets. Silica fume is generally dark gray to black in color.

Silica fume, because of its extreme fineness and high silica content, is highly reactive and effective pozzolanic material. In addition to the pozzolanic reaction, the fine particle size of silica fume also helps to increase paste density by filling voids between the cement grains, thereby improving particle packing and pore size distribution (Brewe and Myers 2005). Because of its extreme fineness, the increased water demand resulting from its use is quite high; therefore, using a high-range water-reducing admixture (HRWRA) is usually required. Silica fume contents typically range from 5 to 10% of the cementitious materials content. The use of silica fume to produce high-strength concrete increased dramatically, starting in the 1980s, with much success. Laboratory and field experience indicates that concrete incorporating silica fume exhibits reduced bleeding but 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 surface drying. An in-depth discussion of silica fume for use in concrete can be found in ACI 234R and the *Silica Fume User's Manual* (Holland 2005).

**3.2.2.3** *High-reactivity metakaolin*—High-reactivity metakaolin (HRM) is a reactive alumino-silicate pozzolan formed by calcining purified kaolin (china) clay at a specific temperature range. Unlike most other SCMs, such as fly ash, slag cement, and silica fume, which are by-products of major industry, HRM is a specifically manufactured material. It is nearly white in color, and usually supplied in powder form. Specifications for HRM are covered under ASTM C618, Class N.

High-reactivity metakaolin is a highly reactive pozzolan suitable for applications where high strength or low permeability is required in structural or repair materials. High-reactivity metakaolin particles are significantly smaller than most cement particles, but are not as fine as silica fume. The average particle size of a HRM produced for concrete applications is approximately 2 μm, or approximately 20 times the average particle size of silica fume. Because of its larger particle size, the increased water demand associated with HRM is not quite as high as it is with silica fume (Caldarone et al. 1994); however, measures to preclude surface drying and plastic cracking may still need to be employed due to a reduction in bleeding rate. HRM contents typically range from 5 to 15% (by mass) of the cementitious materials content used. The specific gravity of HRM is approximately 2.5 (Caldarone et al. 1994).

**3.2.2.4** *Slag cement*—Slag cement is produced only in certain areas of the U.S. and Canada, but is generally available in many North American markets. Specifications and classifications for this material are covered in ASTM C989. Slag appropriate for use in concrete is the nonmetallic product developed in a molten condition simultaneously with iron in a blast furnace. Iron blast-furnace slag essentially consists of silicates and alumino-silicates of calcium and other bases.

When properly quenched and processed, iron blast-furnace slag acts hydraulically in concrete and can be used as a partial replacement for portland cement. According to ACI 233R, most slag cement is batched as a separate constituent at the concrete production plant. Blended hydraulic cements are also produced consisting of slag cement and portland cement produced through intergrinding or intermixing processes. It is the committee's experience that slag cement contents typically range from 30 to 50% (by mass) of the cementitious material content, though higher contents are frequently used for special applications, such as in mass concrete where minimal heat of hydration is desired. The use of HSCs consisting of ternary combinations of portland cement, slag cement, and pozzolans, such as fly ash and silica fume, is also common.

**3.2.3** *Evaluation and selection*—Cementitious materials, like any material in a HSC mixture, should be evaluated using laboratory trial batches to establish optimum desirable qualities. Materials representative of those that will be employed in the actual construction should be used. Care should be taken to ensure that the materials evaluated are representative, come from the same source, and are handled in the same manner as those for the proposed work. For example, if a certain silica fume is to be supplied in bulk form, the material should not be evaluated using a sample that has gone through a bagging process. This general method applies to all constituent materials, including portland cement.

Generally, several trial batches are made using varying cementitious materials contents and chemical admixture dosages to establish curves that can be used to select the optimum amount of cementitious material and admixture required to achieve desired results. Optimum performance results may be characterized in terms of any single or multiple mechanical properties, material properties, or both. For HSC, compressive strength is often an optimum performance property.

## 3.3—Admixtures

**3.3.1** *General*—Admixtures, particularly chemical admixtures, are widely used in the production of HSC. Chemical admixtures are generally produced using lignosulfonates, hydroxylated carboxylic acids, carbohydrates, melamine and naphthalene condensates, and organic and inorganic accelerators in various formulations. Air-entraining admixtures are generally surfactants that will develop an air-void system appropriate for enhanced durability. Chemical admixtures are most commonly used for water reduction and set time alteration, and can additionally be used for purposes such as corrosion inhibition, viscosity modification, and shrinkage control. 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 can be achieved with the proper selection and use of chemical admixtures. Reliable performance on previous work and compatibility with the proposed cementitious materials and between chemical admixtures should be considered during the selection process. Specifications for chemical admixtures and air-entraining admixtures are covered under ACI 212.3R, ASTM C494/C494M and C260.

**3.3.2** *Chemical admixtures*

**3.3.2.1** *Retarding chemical admixtures (ASTM C494/C494M, Types B and D)*—High-strength concrete mixtures incorporate higher cementitious materials contents than conventional-strength concrete. Retarding chemical admixtures are highly beneficial in controlling early hydration, particularly as it relates to strength (ACI 212.3R). With all else being equal, increased hydration time results in increased long-term strength. Retarding chemical admixtures are also beneficial in improving workability. Adding water to retemper a HSC mixture and maintain or recover workability will result in a marked strength reduction. Structural design frequently requires heavy reinforcing steel and complicated forming with difficult placement of concrete. A retarding admixture can control the rate of hardening in the forms to eliminate cold joints and provide more flexibility in placement schedules. The dosage of a retarding admixture can be adjusted to give the desirable rate of hardening under anticipated temperature conditions.

Retarding admixtures frequently provide a strength increase proportional to the dosage rate, although the selected dosage rate is significantly affected by ambient temperatures conditions (ACI 212.3R). Mixture proportions can be tailored to ambient conditions with a range of retarding admixture dosages corresponding to the anticipated temperature conditions. During summer months, an increase in retarder dosage can effectively mitigate temperature-induced strength reduction. During winter months, dosage rates are often decreased to prevent objectionably long setting times. Transition periods between summer and winter conditions may be handled with a corresponding adjustment in the retarding admixture dosage.

When the retarding effect of the admixture has diminished, normal or slightly faster rates of heat liberation usually occur. Depending on the type and dosage of retarding admixture used, early hydration can be effectively controlled while maintaining favorable 24-hour strengths. Extended retardation or cool temperatures may adversely affect early strengths.

**3.3.2.2** *Normal-setting chemical admixtures (ASTM C494/C494M, Type A)*—Type A water-reducing chemical admixtures, commonly called normal-setting or conventional chemical admixtures, can provide strength increases while having minimal effect on rates of hardening. Their selection should be based on strength performance. Dosages increased above the manufacturer's recommended amounts generally increase strengths, but may extend setting times.

**3.3.2.3** *High-range water-reducing chemical admixtures (ASTM C494/C494M, Types F and G)*—One potential advantage of HRWRAs is decreasing the *w/cm* and providing high-strength performance, particularly at early (24-hour) ages (Mindess et al. 2003). Matching the chemical admixture to cementitious materials both in type and dosage rate is important. Slump loss characteristics of the concrete will determine whether the HRWRA should be introduced at the plant, at the site, or at both locations. With the advent of newer-generation products, however, sufficient slump retention can be achieved through plant addition in most cases (ACI 212.3R).

High-range water-reducing admixtures may serve the purpose of increasing strength through a reduction in the *w/cm* while maintaining equal slump, increasing slump while maintaining equal *w/cm*, or a combination thereof. The method of addition should distribute the admixture uniformly throughout the concrete. Adequate mixing is critical to achieve uniformity in performance. Problems resulting from nonuniform admixture distribution or batch-to-batch dosage variations include inconsistent slump, rate of hardening, and strength development. Proper training of site personnel is essential to the successful use of a HRWRA at the project site.

**3.3.2.4** *Accelerating chemical admixtures (ASTM C494/C494M, Types C and E)*—Accelerating admixtures are not normally used in HSC unless early form removal or early strength development is absolutely critical. High-strength concrete mixtures can usually be proportioned to 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 to long-term strength development.

**3.3.2.5** *Air-entraining admixtures (ASTM C260)*—The use of air entrainment is recommended to enhance durability when concrete will be subjected to freezing and thawing while critically saturated or in the presence of deicers. Critical saturation is when the moisture content within the capillaries or pores exceeds 91.7%. To reach critical saturation, concrete requires direct contact with moisture for long periods. Exterior exposure alone does not justify the use of air entrainment in HSC. Periodic precipitation, such as rain or snow against a vertical surface alone, does not constitute conditions conducive to critical saturation. In 1982, Gustaferro et al. (1983) inspected 20 out of 50 concrete bridges built on the Illinois Tollway in 1957. They observed minimal freezing-and-thawing damage in the non-air-entrained, precast, prestressed concrete bridge beams. Even though the bridges were geographically located in a severe freezing-and-thawing region and subjected to deicer chemicals from the adjacent roadway, a non-air-entrained mixture was selected because tollway engineers were concerned that air-entrained HSC could not be economically achieved on a daily basis. Entrained air can significantly reduce the strength of high-strength mixtures and increase potential for strength variability as air contents in the concrete vary; therefore, extreme caution should be exercised with respect to its use. Even though many state departments of transportation require entrained air in prestressed precast HSC bridge girders, air entrainment in HSC should be avoided unless absolutely necessary. Refer to Sections 4.6 and 6.12.

**3.3.2.6** *Chemical admixture combinations*—Combining HRWRAs with water-reducing or retarding chemical admixtures has become common practice to achieve optimum performance at lowest cost. With optimized combinations, improvements in strength development and control of setting times and workability are possible. When using a combination of admixtures, they should be dispensed individually as approved by the manufacturer(s). Air-entraining admixtures, if used, should never directly contact chemical admixtures during the batching process.

## 3.4—Aggregates

**3.4.1** *General*—Production of HSC requires purposeful selection of quality aggregates. Both fine and coarse aggregates used for HSC should, as a minimum, meet the requirements of ASTM C33/C33M; however, there are several exceptions that discussed in this section that have been found to be beneficial for HSC.

**3.4.2** *Fine aggregate*—Fine aggregates with a rounded particle shape and smooth texture have been found to require less mixing water in concrete; for this reason, they are preferable in HSC (Wills 1967; Gaynor and Meininger 1983). The optimum gradation of fine aggregate for HSC is determined more by its effect on water requirement than on physical packing. Blick (1973) reported that sand with a fineness modulus below 2.50 gave the concrete a sticky consistency, making it difficult to compact. Sand with an fineness modulus of approximately 3.0 gave the best workability and compressive strength. Also, refer to Section 4.7.1.

High-strength concretes typically contain such high contents of fine cementitious materials that the grading of the fine aggregates used is less critical compared with conventional concrete. However, the fine aggregate may be used to enhance the particle packing aspects of the mixture design. It is sometimes helpful, however, to increase the fineness modulus. A National Crushed Stone Association report (1975) made several recommendations in the interest of reducing the water requirement. The amounts passing the No. 50 (300 μm) and No. 100 (150 μm) sieves should be kept low, but within the requirements of ASTM C33/C33M, and mica or clay contaminants should be avoided. In the same study, it was reported that 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."

**3.4.3** *Coarse aggregate*—Coarse aggregate mineralogical characteristics, grading, shape, surface texture, elastic modulus (stiffness), and cleanliness can influence concrete properties. Many varieties of coarse aggregates have proved suitable for high-strength concrete production, but some aggregates are more suitable than others. No simple guidance on the selection of coarse aggregate is available (Neville 1996). Coarse aggregate may have a more pronounced effect in high-strength concrete than in conventional concrete (Mokhtarzadeh and French 2000a). In conventional concrete, compressive strength is typically limited by the cement paste capacity or by the capacity of the bond between coarse aggregate and cement paste. In high-strength concrete, where the cement paste and coarse aggregate and cement paste bond are enhanced by design of a low *w/cm* and use of SCMs, ultimate strength potential may be limited by the intrinsic strength of the coarse aggregate itself (deLarrard and Belloc 1997; Aïtcin and Neville 1993; Cetin and Carrasquillo 1998; Sengul et al. 2002).

Coarse aggregates occupy the largest volume of any of the constituent materials in concrete. In HSC, coarse aggregate volumes typically range between 50 and 70%. The optimum amount depends on the maximum size of coarse aggregate and the fineness modulus of the fine aggregate. As the maximum size of coarse aggregate increases, the optimum amount of coarse aggregate in concrete also increases. As the fineness modulus of the fine aggregate increases, the optimum amount of coarse aggregate in concrete decreases (Freyne 2000).

Past studies (Blick 1973; Perenchio 1973) have shown that for optimum compressive strength with high cementitious material contents and low *w/cm*, the maximum size of coarse aggregate should be kept to a minimum, at 1/2 or 3/8 in. (13 or 10 mm). Maximum sizes of 3/4 and 1 in. (19 and 25 mm) have also been used successfully (Cook 1982).

Maximum aggregate sizes of 1/2 in. (13 mm), or smaller sizes of coarse aggregate and crushed coarse aggregate, are recommended for use in HSC. Smaller sizes of coarse aggregate have greater surface area for a given aggregate content, which improves coarse aggregate and cement paste bond and enhances ultimate strength potential. The crushing process eliminates potential zones of weakness within the parent rock with the effect that smaller particles are likely to be stronger than larger ones (deLarrard and Belloc 1997). 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. Coarse aggregate with a rough surface texture is generally more suitable for use in HSC than coarse aggregate with a smooth surface texture because of the superior bond that it provides (Mokhtarzadeh et al. 1995; Neville 1997).

Optimum strength in an HSC mixture can most often be achieved through the use of smaller-sized aggregates. The governing factor for selecting HSC for a structure, however, may be a property other than strength. For example, in a tall building, modulus of elasticity may be the primary reason that HSC is specified. In such cases, a larger-sized aggregate, though yielding lower strength, may provide a higher modulus of elasticity.

Studies have shown that crushed stone produces higher strengths than rounded gravel (Perenchio 1973; Walker and Bloem 1960; Harris 1969). The likely reason for this is the greater mechanical bond that can develop with angular particles. Accentuated angularity, however, is to be avoided because of the attendant high water requirement and reduced workability. Aggregate should be clean, cubical, angular, 100% crushed aggregate with a minimum of flat and elongated particles. Refer to Section 4.7.2.

**3.4.3.1** *Paste-aggregate homogeneity*—Neville (1996) reported that designing HSC to act more like a homogeneous material can enhance ultimate strength potential. This can be achieved by increasing the similarity between the elastic moduli of coarse aggregate and cement paste. Having like elastic moduli will reduce stress at the paste-aggregate interface. Using a coarse aggregate with greater stiffness has been found to increase the elastic modulus of concrete, but it is sometimes detrimental to ultimate strength potential (Cetin and Carrasquillo 1998; Myers 1999; Tadros et al. 1999).

**3.4.4** *Intrinsic aggregate strength*—High-strength concrete often uses higher-strength and higher-quality aggregates to generate the targeted compressive strength level. Using normal-strength or low-quality aggregates will result in fracture of the aggregate before fully developing strength potential of the paste matrix or bond strength of the aggregate-paste transition zone. Developing a paste matrix and selecting an aggregate type that has a compatible relative strength and stiffness will yield high-compressive-strength concrete as further discussed in Section 6.3.

### 3.5—Water

The requirements for mixing water quality for HSC are no more stringent than those for conventional concrete. Specifications for standard and optional compositional and performance requirements for water used as mixing water in hydraulic cement concrete are covered in ASTM C1602/C1602M. Potable water is permitted to be used as mixing water in concrete without testing for conformance to the requirements of ASTM C1602/C1602M.

As a result of environmental regulations that prevent the discharge of runoff water from production facility properties, use of nonpotable water or water from concrete production operations is increasing. Nonpotable water includes water containing quantities of substances that discolor it, make it smell, or have objectionable taste. Water from concrete production operations includes wash water from mixers or water that was part of a concrete mixture that was reclaimed from a concrete recycling process, water collected in a basin as a result of storm water runoff at a concrete production facility, or water that contains quantities of concrete ingredients. Water from these sources should not be used to produce HSC unless it has been shown that their use will not adversely affect the properties of the concrete.

## CHAPTER 4—CONCRETE MIXTURE PROPORTIONS

### 4.1—Introduction

Concrete mixture proportions for HSC have varied widely. Factors influencing mixture proportions include the strength level required, test age, material characteristics, and type of application. In addition, economics, structural requirements, manufacturing practicality, anticipated curing environment, and even the time of year have affected the selection of mixture proportions. Much information on proportioning concrete mixtures is available in ACI 211.1, which deals specifically with proportioning HSC containing fly ash.

High-strength concrete mixture proportioning is a more critical process than proportioning normal-strength concrete mixtures. Frequently, the use of SCMs and chemical admixtures, and the attainment of a low *w/cm* are considered essential in high-strength mixture proportioning. Many trial batches are often required to generate the data that enable optimum mixture proportions to be identified.

### 4.2—Strength required

**4.2.1** *ACI 318*—As with most structural concretes, HSC is usually specified in terms of its compressive strength. ACI 318 specifies concrete strength requirements. Structural concrete is normally proportioned so that the average compressive strength test results exceed specified strength $f_c'$ by an amount sufficiently high to minimize the frequency of test results below the specified compressive strength (refer to ACI 214R).

An average value can be calculated for any set of measurement data. The fraction of individual test values that deviate from the average is usually quantified by the standard deviation. The standard deviation of test results can be valuable in predicting future variability.

Many factors can influence the variability of compressive strength test results, including variations in testing equipment and procedures, constituent materials, production facilities, delivery equipment, inspection agencies, and environmental conditions. All factors that may affect the variability of measured strength should be considered when selecting mixture proportions and establishing the acceptable standard deviation for strength results. Carrasquillo (1994) identified principal factors affecting compressive strengths of normal- and high-strength concretes, including specimen moisture condition, specimen size, and end conditions. Burg et al. (1999) investigated the effect of end conditions, curing methods, specimen size, and testing machine properties for HSC. Refer to ACI 363.2R for additional information on quality control and testing of HSC.

High-strength concrete is more sensitive to variations in mixture proportions and testing than normal-strength concrete, and is recognized to be more challenging to evaluate accurately than lower-strength concretes. A high variability 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 computing a standard deviation, the concrete producer should use the most realistic test record.

ACI 318 recognizes that some test results are likely to be lower than the specified strength. Acceptance criteria are designed to limit the frequency of tests allowed to fall below the specified strength. ACI 318-05, Section 5.6.3.3 considers the strength level of an individual class of concrete satisfactory if both of the following requirements are met:

a) Every arithmetic average of any three consecutive strength tests (average of two cylinders) equals or exceeds $f_c'$; and

b) No individual strength test (average of two cylinders) falls below $f_c'$ by more than 500 psi (3.4 MPa) when $f_c'$ is 5000 psi (34 MPa) or less, or by more than $0.10f_c'$ when $f_c'$ is more than 5000 psi (34 MPa).

When $f_c'$ exceeds 5000 psi (34 MPa), and when strength data are available to establish a standard deviation $s_s$, the required average strength $f_{cr}'$ used as the basis for selection of mixture proportions should be based on the larger value computed from the following equations (ACI 318-05, Table 5.3.2.1):

$$f_{cr}' = f_c' + 1.34s_s \qquad \text{(SAE units, psi)} \tag{4-1}$$

$$f_{cr}' = 0.90f_c' + 2.33s_s \qquad \text{(SAE units, psi)} \tag{4-2}$$

When strength data are not available to establish a standard deviation, the required average strength $f_{cr}'$, used as the basis for selection of concrete proportions when $f_c'$ exceeds 5000 psi (34 MPa), should be based on the following equation (ACI 318-05, Table 5.3.2.2):

$$f_{cr}' = 1.10f_c' + 700 \qquad \text{(SAE units, psi)} \tag{4-3}$$

ACI 318 allows mixtures to be proportioned based on field experience or by laboratory trial batches. When the concrete producer chooses to select HSC mixture proportions based upon laboratory trial batches, mixture performance under field conditions should also be confirmed before proceeding with the work.

**4.2.2** *ACI 214R*—Once sufficient test data have been generated from the project, a reevaluation of mixture proportions based on actual test results is required. Refer to ACI 214R for methods of monitoring strength test results during production. Analyses affecting reproportioning of mixtures based upon test histories are described in Chapter 5.

**4.2.3** *Other requirements*—In some situations, considerations other than compressive strength may influence mixture proportions. A detailed discussion of the mechanical properties of HSC, including flexural strength, tensile strength, modulus of elasticity, shrinkage, and creep is given in Chapter 6. Chapter 6 also presents a discussion on material properties that influence HSC.

## 4.3—Test age

Selection of mixture proportions can be influenced by the testing age or early-age strength requirements. Testing age depends upon construction requirements. Testing age is usually the age at which acceptance criteria are established, for example, at 56 or 90 days. Testing, however, can be conducted before the age of acceptance testing, or after that age, depending on the type of information desired.

**4.3.1** *Early age*—Pretensioned concrete operations may require very high strengths in 12 to 24 hours. Special applications for early use of machinery foundations, pavement traffic lanes, or slipformed concrete have required high strength at early ages. Post-tensioned concrete is often stressed at ages of 2 to 3 days or more, and requires high strength at later ages. Generally, once the effect of set-retarding admixtures have subsided, early-age strength development can be significant. The optimum materials and mixture proportions selected, however, may vary for different test ages. For example, mixtures with Type III cement have been used for high early strength, compared with Types I, II, or V cement for high later-age strength. Early-age strengths may be more variable due to the influence of curing temperature and the early age strength development characteristics of the specific cement, SCM, or chemical admixture. Therefore, mixture proportions should be evaluated for a higher required average strength. The effects of SCMs and chemical admixtures on early-age strength are addressed further in Section 4.8 (Leming et al. 1993a; Zia et al. 1993a,b; Ahmad and Zia 1997).

**4.3.2** *Twenty-eight days*—A common test age for compressive strength of normal-strength concrete is 28 days. Performance of structures has been empirically correlated with the strength of moist-cured concrete cylinders, usually 6 x 12 in. (150 x 300 mm) or 4 x 8 in. (100 x 200 mm) prepared according to ASTM C31/C31M and C192/C192M. This has produced good results for normal-strength concretes not requiring early strength or early evaluation.

**4.3.3** *Later age*—High-strength concretes made with SCMs may gain considerable strength at later ages and, therefore, are typically evaluated at later ages, such as 56 or 90 days, when construction requirements allow the concrete more time to develop strength before loads are imposed. High-strength concrete has been placed frequently in columns or shear walls of high-rise buildings. Therefore, it has been desirable to take advantage of long-term strength gains so that efficient use of construction materials is achieved. This has often been justified in applications such as

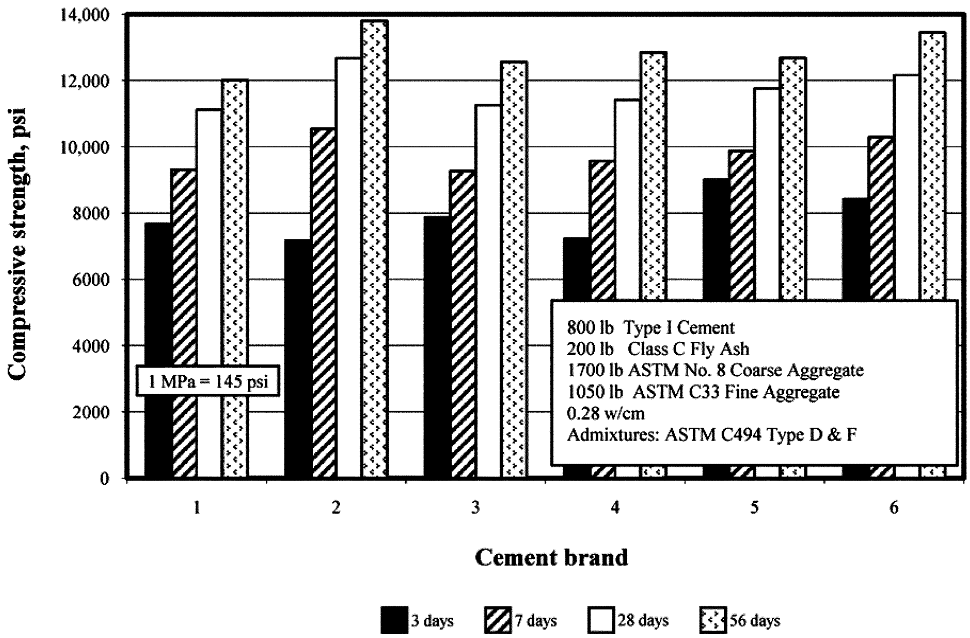


*Fig. 4.1—Effects of various brands of cement on concrete compressive strength.*

high-rise buildings where full loading may not occur until significantly later ages.

In cases where later-age acceptance criteria are specified, it may be advantageous for the concrete supplier to develop early-age or accelerated strength test data to estimate later-age strengths, refer to ASTM C684 and C918/C918M. In such cases, correlation data should be developed for the materials and proportions to be used in the work. These tests may not always accurately estimate later-age strengths, but they can provide an early identification of lower-strength trends before a long history of noncompliance is realized.

Extra test cylinders should be prepared and held for testing at ages later than the specified acceptance age. In cases where the specified compressive strength is not achieved, subsequent testing of later-age or "hold" cylinders may justify acceptance of the concrete in question.

**4.3.4** *Test age in relationship to curing*—When selecting mixture proportions, the type of curing anticipated should be considered along with the test age, especially when designing for high early strengths. Concrete gains strength as a function of maturity, which is defined as a function of curing time and curing temperature. This is particularly important for steam-cured precast concrete.

## 4.4—Water-cementitious material ratio

**4.4.1** *Nature of* w/cm *in high-strength concrete*—When SCMs such as pozzolans or slag cement are used in concrete, a *w/cm* by mass has been considered in place of the traditional *w/c* by mass.

The relationship between the *w/cm* and compressive strength, which has been identified in lower-strength concretes, is applicable to higher-strength concretes as well. Higher cementitious materials contents and lower water contents have produced higher strengths. In many cases, however, using larger amounts of cementitious material increases water demand. Depending on properties of the cementitious materials used, increasing the cementitious material content beyond a certain point has not always resulted in increased compressive strength. Other factors that may limit maximum contents of cementitious materials are discussed in Section 5.5.3. The use of HRWRAs has enabled concrete to be placed at flowing and self-consolidating consistencies with lower *w/cm*. HRWRAs are discussed in Section 4.8.2.2.

Water-cementitious material ratios by mass for HSCs have ranged typically from 0.25 to 0.40. The quantity of water contained in liquid admixtures, particularly HRWRAs, should always be included in determining the *w/cm*.

As the *w/cm* changes, the density of the concrete also changes. By incrementally decreasing the *w/cm*, less cementitious material is available to hydrate (Mindess et al. 2003). As long as decreasing the *w/cm* increases density, strength should also increase. Any unhydrated cementitious material will merely act as mineral filler.

**4.4.2** *Estimating compressive strength*—The compressive strength that a concrete will develop at a given *w/cm* depends on the cementitious materials, aggregates, and admixtures employed.

Principal causes of variations in compressive strengths at a given *w/cm* include the strength-producing capabilities of the cement and the hydraulic or pozzolanic activity of SCMs, if used. Figure 4.1 shows the effects of various brands of Type I portland cement on compressive strength.

Specific information pertaining to the range of values of compressive strengths of portland cements is published in ASTM C917. Depending on their chemical and physical properties, fly ashes and natural pozzolans may vary in their pozzolanic activity index from 75 to 110% or more of the portland cement control. The pozzolanic activity index for fly ash and natural pozzolans is specified in ASTM C618. Similarly, the strength activity indexes for silica fume and various grades of slag cement are given in ASTM C1240 and C989, respectively. Proprietary pozzolans containing silica fume have been reported to have activity indexes in excess of 200% (Gaynor 1980).

The water requirement of the particular pozzolan employed can vary significantly, and generally increases with increasing fineness of the pozzolan. For example, as a result of the nearly spherical shape of fly ash particles, the water requirement for concrete containing fly ash is usually lower than for concrete made only with portland cement, which helps in lowering the *w/cm*.

Perenchio and Klieger (1978) reported variations in compressive strength at given *w/cm* in laboratory-prepared concretes, depending on the aggregates used. In addition, these laboratory results differed from results achieved in actual production with materials from the same area. In total, three aggregate sources were used in their study. Maximum aggregate size was 3/8 in. (10 mm) for the Elgin and Dresser aggregates and 1/2 in. (13 mm) for the Romeoville limestone used. Examples of strengths reported at given *w/cm* are presented in Fig. 4.2. Trial batches with materials actually to be used in the work were found to be necessary. Generally, laboratory trial batches have produced strengths higher than are achievable in production, as seen in Fig. 4.3.

## 4.5—Cementitious material content

The quantity of cementitious material proportioned in a HSC mixture is best determined by making trial batches. The required content of cementitious material in a HSC mixture is usually governed by the required *w/cm*. Typical cementitious materials contents in HSC test programs have ranged from 650 to 1000 lb/yd$^3$ (386 to 593 kg/m$^3$). In evaluating optimum cementitious materials contents, trial mixtures usually are proportioned to equal consistencies. This can be achieved either by allowing the admixture dosage to vary and keeping the water content fixed, or by allowing the water content to vary and keeping the admixture dosage fixed.

**4.5.1** *Cement strength*—The strength for any given cement or cementitious materials content will vary with the water demand of the mixture and the strength-producing characteristics of the particular combination of cementitious material, as shown in Fig. 4.1. Figure 4.1 illustrates a variation in compressive strength on the order of 10% when comparing different cement brands. Strength-producing characteristics of cements at a given age can vary depending on the mixture proportions and compatibility with other materials, particularly SCMs and chemical admixtures. The relative strength performance of cement can differ depending on the strength level of the concrete, as shown in Fig. 4.4. High-strength concrete is more sensitive to cement brand than normal-strength concrete. This may be attributed to the varied interaction of the cement and the mixture constituent chemical and mineral admixtures. For example, cement that exhibits one level of relative strength performance in a 4000 psi (27 MPa) concrete mixture may perform quite differently in a 10,000 psi (69 MPa) mixture.

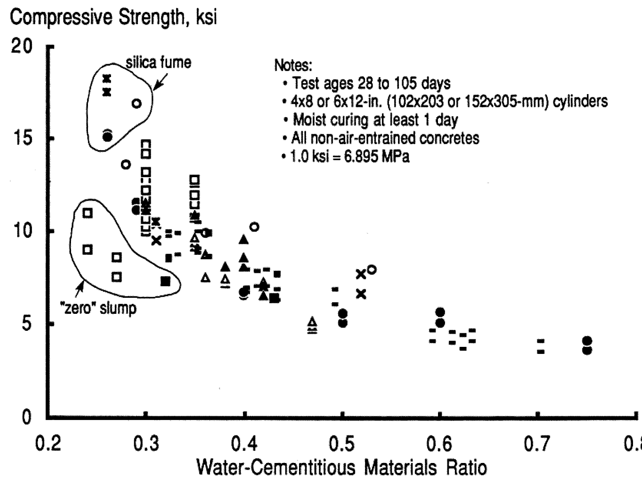


*Fig. 4.2—Strength versus* w/cm *of various mixtures (adapted from Fiorato [1989]).*

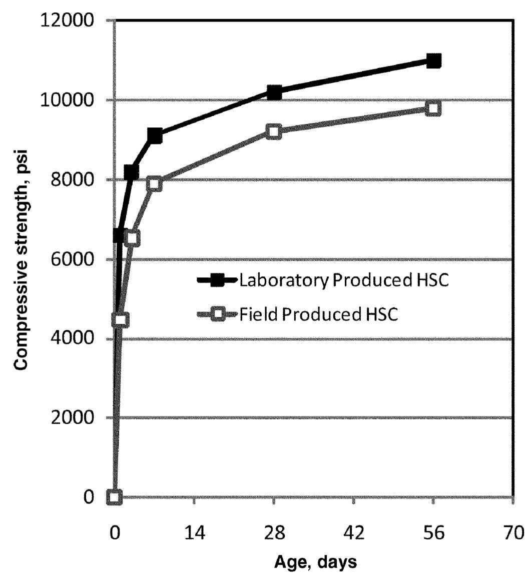


*Fig. 4.3—Laboratory-molded concrete strengths versus ready mixed field-molded concrete strengths for 9000 psi (62 MPa) concrete (adapted from Myers [1999]).*

Concrete strength depends on 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" (Neville 1981; Leming et al. 1993b).

Although mortar cube tests (ASTM C109/C109M) can be extremely useful in monitoring the strength uniformity of cement over time, the performance of cement in a mortar cube can be quite different than its strength performance in concrete. Therefore, strength characteristics of various cements and combinations of cement and SCMs should be evaluated in concrete rather than mortar.

**4.5.2** *Optimization*—A principal consideration in establishing the desired cementitious material content is the determination of material combinations that will produce optimum strengths. Ideally, evaluations of each potential source of cementitious materials, aggregates, and chemical admixtures in varying quantities would indicate the optimum

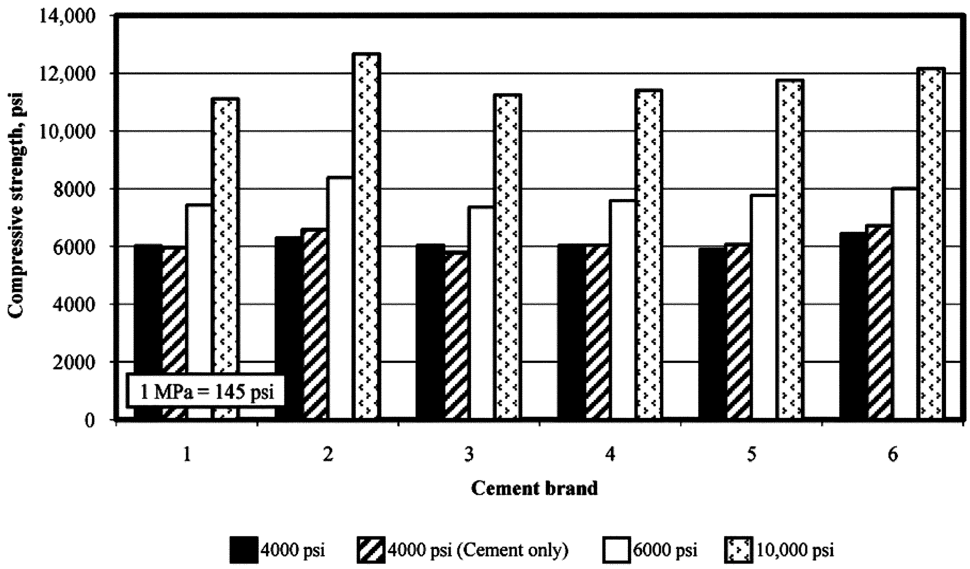


*Fig. 4.4—Effects of various brands of cement on concrete compressive strength using different mixture proportions. (Mixture proportions shown in Tables 4.1(a) and (b).)*

**Table 4.1(a)—Laboratory mixtures used in Fig. 4.4 study (U.S. Customary units)**

| | Mixture no. | | | |
|---|---|---|---|---|
| | 1 | 2 | 3 | 4 |
| Specified strength, psi | 4000 | 4000 | 6000 | 10,000 |
| Type I cement, lb/yd$^3$ | 423 | 564 | 588 | 800 |
| Class C fly ash, lb/yd$^3$ | 80 | 0 | 125 | 200 |
| ASTM C33 fine aggregate, lb/yd$^3$ | 1500 | 1450 | 1320 | 1050 |
| 3/4 in. coarse aggregate, lb/yd$^3$ | 1750 | 1750 | 1750 | 0 |
| 3/8 in. coarse aggregate, lb/yd$^3$ | 0 | 0 | 0 | 1700 |
| Type A WR, oz/yd$^3$ | 12.7 | 0 | 17.6 | 0 |
| Type D WR, oz/yd$^3$ | 0 | 0 | 0 | 32 |
| Type F HRWR | 0 | 0 | 0 | Varied |
| Water, lb/yd$^3$ | Varied | Varied | Varied | 280 |
| *w/cm* | Varied | Varied | Varied | 0.28 |
| Target slump, in. | 5 | 5 | 5 | 8 |

**Table 4.1(b)—Laboratory mixtures used in Fig. 4.4 study (SI units)**

| | Mixture no. | | | |
|---|---|---|---|---|
| | 1 | 2 | 3 | 4 |
| Specified strength, MPa | 28 | 28 | 41 | 69 |
| Type I cement, kg/m$^3$ | 251 | 335 | 349 | 475 |
| Class C fly ash, kg/m$^3$ | 47 | 0 | 74 | 119 |
| ASTM C33 fine aggregate, kg/m$^3$ | 890 | 860 | 783 | 623 |
| 19 mm coarse aggregate, kg/m$^3$ | 1038 | 1038 | 1038 | 0 |
| 9.5 mm coarse aggregate, kg/m$^3$ | 0 | 0 | 0 | 1009 |
| Type A WR, mL/m$^3$ | 492 | 0 | 681 | 0 |
| Type D WR, mL/m$^3$ | 0 | 0 | 0 | 1239 |
| Type F HRWR | 0 | 0 | 0 | Varied |
| Water, kg/m$^3$ | Varied | Varied | Varied | 166 |
| *w/cm* | Varied | Varied | Varied | 0.28 |
| Target slump, mm | 125 | 125 | 125 | 200 |

cementitious materials content and optimum combination of constituent materials. Testing costs and time requirements have usually limited the completeness of evaluation programs, but particular attention has been given to evaluation of the type and brand of cement to be used with the type and source of SCMs.

The strength efficiency of cementitious material combinations will vary for different nominal maximum-size aggregates at different strength levels. Higher cementitious material efficiencies are achieved at high strength levels with smaller maximum aggregate sizes. Figure 4.5 illustrates this principle. For example, a nominal maximum aggregate size of approximately 3/8 in. (10 mm) yields the highest cement efficiency for a 7000 psi (48 MPa) mixture.

Incorporating SCMs and chemical admixtures can significantly increase concrete strength (Myers and Carrasquillo 2000). Today, HSCs with specified compressive strengths up to 16,000 psi (110 MPa) at 56 days have been produced successfully using crushed aggregate having a nominal maximum size of 3/8 in. (10 mm) with corresponding cementitious efficiency values of 17 psi/lb/yd$^3$ (0.29 MPa/kg/m$^3$).

**4.5.3** *Limiting factors*—There are several factors that may limit the maximum quantity of cementitious material that may be desirable in a high-strength mixture. Concrete strength may decrease if the cementitious materials content exceeds optimum value. The maximum desirable content of cementitious material may vary considerably depending upon the efficiency of dispersing agents, such as MRWRAs or HRWRAs, in promoting deflocculation of cementitious particles.

Extremely low *w/cm* or high cementitious material contents can have a significant effect on the rheology of the concrete mixture. Stickiness and loss of workability may increase as higher amounts of cementitious materials are incorporated into the mixture. Combinations of constituent materials should be evaluated for their effect on the ability to place, consolidate, and finish the mixture. As discussed in

Section 4.9.3, combinations of chemical admixtures such as MRWRAs and HRWRAs may reduce stickiness and improve workability. To date, no standard test methods are available to evaluate finishing characteristics.

The maximum temperature permitted in the concrete element may limit the quantity or type of cementitious material. It may be helpful to use materials that are capable of reducing the initial temperature and, subsequently, the peak temperature, such as ice, chilled water, and liquid nitrogen. Furthermore, the temperature rise and, subsequently, the peak temperature, can be reduced by using slag cement and pozzolans.

Mixtures with high cementitious materials contents may frequently have higher water demands, particularly if the cementitious material is composed of extremely finely divided particles, such as silica fume. Under some circumstances, it may be preferable to reduce the amount of cementitious material in the mixture and to rely more upon careful selection of aggregates and aggregate proportions.

The amount of early stiffening (loss of workability) can vary depending on the type and quantity of cementitious materials and chemical admixtures used. In some cases, loss of workability has been attributed to poor constituent material compatibility. As the use of retempering water can result in significant strength loss, it should not be permitted as a remedy to loss of workability.

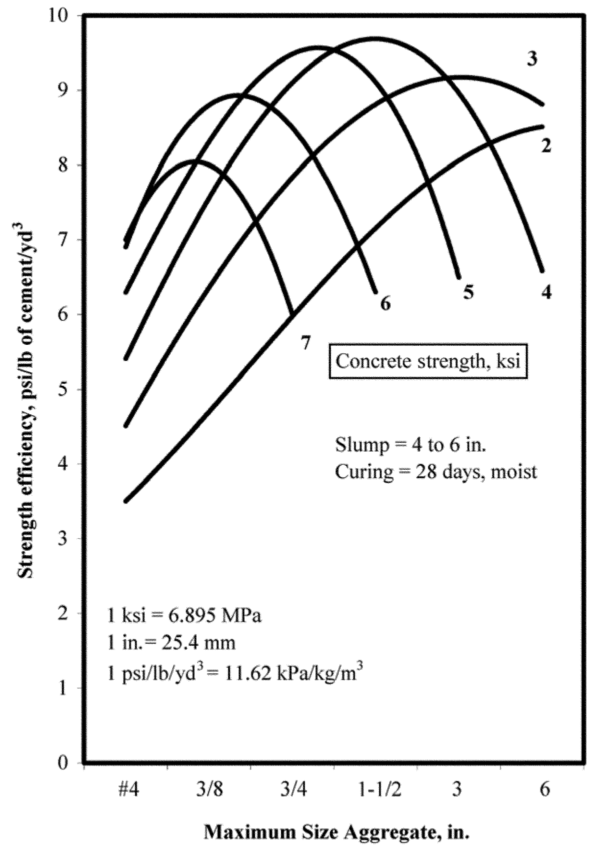


*Fig. 4.5—Maximum-size aggregate for strength efficiency envelope (adapted from Cordon and Gillespie [1963]).*

## 4.6—Air entrainment

**4.6.1** *Resistance to freezing and thawing*—There are advantages and disadvantages associated with the use of air entrainment. The primary advantage of having entrained air is the protection it provides in the event that the moisture content within the capillaries or pores exceeds critical saturation. As the water within concrete freezes, it expands approximately 9% by volume. Without a system of tiny, uniformly distributed air bubbles throughout the mortar fraction, this expansion can produce hydraulic and osmotic pressures within the capillaries and pores of the paste and aggregate that will damage the concrete.

To reach critical saturation, concrete has to be in direct contact with moisture for long periods. Obviously, horizontal members are significantly more susceptible to critical saturation than vertical members. Periodic precipitation, such as rain or snow against a vertical surface alone, does not constitute conditions conducive to saturation. Because of the significantly detrimental effects it can have on strength, air entrainment should be used in HSC only when absolutely necessary.

Additional discussions on the freezing-and-thawing resistance of HSC are provided in Chapter 6.

**4.6.2** *Effect on strength*—The primary disadvantage of air entrainment is its negative effect on strength. To achieve equal strength, air-entrained concrete generally requires a lower *w/cm* and, therefore, a higher quantity of cementitious material than non-air-entrained concrete. The quantity of cementitious material needed to attain equal strength varies depending on the strength class of the concrete. For example, it is the committee's experience that a 4000 psi (27 MPa) air-entrained concrete mixture may require only an additional 50 lb/yd$^3$ (30 kg/m$^3$) of cementitious material than a non-air-entrained mixture to attain equal strength, whereas a 6000 psi (41 MPa) air-entrained mixture might require an additional 150 lb/yd$^3$ (90 kg/m$^3$) of cementitious material than its non-air-entrained counterpart. The specific difference depends on the characteristics of the local constituent materials. Beyond 6000 psi (41 MPa), however, the decrease in strength due to the inclusion of entrained air becomes so large that it is usually necessary to include SCMs such as silica fume or high-reactivity metakaolin.

The decrease in strength for each incremental increase in air content becomes larger as the specified strength $f_c'$ of the concrete increases. For example, in a 4000 psi (27 MPa) concrete mixture, an air content increase from 5 to 7% may reduce compressive strength by 200 to 400 psi (1.4 to 2.8 MPa), or 5 to 10%. In a 10,000 psi (69 MPa) mixture, the same air content increase may reduce strength by 2000 to 3000 psi (6.9 to 13.8 MPa), or 20 to 30%. The effect of increasing air content on the compressive strength of various concretes is demonstrated in Fig. 4.6(a) (Gaynor 1968). Ekenel et al. (2004) observed a similar trend for HSC mixtures, although the scatter of data appear to be more sensitive to the mixture constituents and SCMs used (Fig. 4.6(b)).

Because normal fluctuations in air content will have a significantly more dramatic effect on strength of HSC, higher variations in strength should be expected. As a result, the required average strength $f_{cr}'$ of air-entrained HSC is expected to be higher than non-air-entrained HSC.

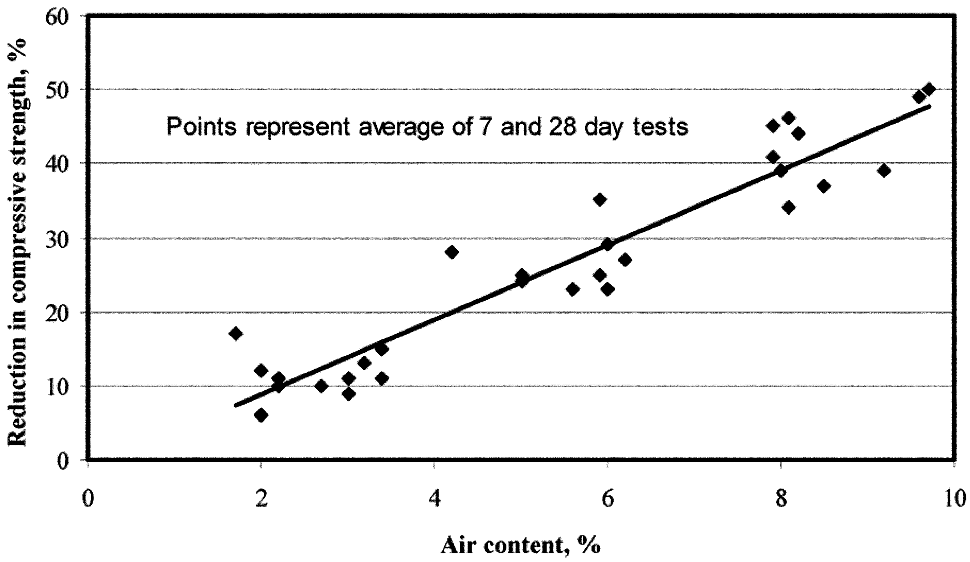


*Fig. 4.6(a)—Strength reduction by air entrainment (adapted from Gaynor [1968]).*

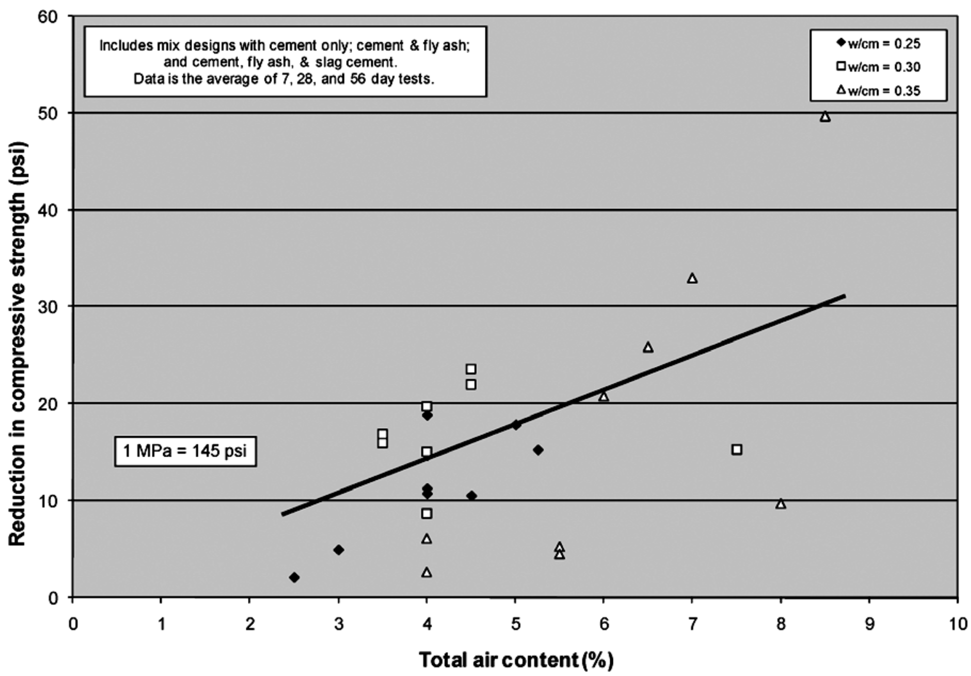


*Fig. 4.6(b)—Strength reduction by air entrainment (adapted from Ekenel et al. [2004]).*

As a result of the potentially detrimental effects it can have on the strength of HSC, air entrainment should be considered only when truly warranted. Reduction of air content by 1% for concrete compressive strength greater than 5000 psi is permitted by ACI 318-08, Section 4.4.1.

## 4.7—Aggregate proportions

Aggregates are an important consideration in proportioning HSC because they occupy the largest volume of the constituent ingredients in the concrete. Usually, HSCs have been produced using normal-density aggregates. Shideler (1957), Holm (1980), and Hoff (1992) reported on lightweight, high-strength structural concrete. Mather (1965) reported on high-strength, high-density concrete using high-density aggregate.

**4.7.1** *Fine aggregates*—Fine aggregate or sand has a significant effect on mixture proportions. Fine aggregate contains a much higher surface area for a given mass than the coarse aggregate. Because the surface area of aggregate particles is coated with a cementitious paste, the proportion of fine-to-coarse aggregate can have a direct effect on paste requirements. Furthermore, fine aggregate particles may be spherical, subangular, or very angular. Particle shape can alter paste requirements even though net volume of the fine aggregate remains the same.

The gradation of the fine aggregate plays an important role in properties of fresh and hardened concrete. For example, if the gradation has an overabundance of particles retained on the No. 50 and 100 (300 and 150 μm) sieve sizes, workability will be improved, but more paste will be needed to compensate for the increased surface area. This could result in a more expensive mixture, or if water were added to increase the paste volume, there would be a serious loss in strength. It is sometimes possible to blend fine aggregates from different sources to improve their gradation and capacity to produce higher-strength concrete. High-strength concretes have been produced using blends of manufactured and natural fine aggregates.

Low fine aggregate contents with high coarse aggregate contents have resulted in a reduction in paste requirements and have typically been more economical. Such proportions also have made it possible to produce higher strengths for a given amount of cementitious materials. If the proportion of fine aggregate is too low, however, there may be serious problems in workability.

Consolidation with mechanical vibrators may help overcome the effects of an under-sanded mixture, and using power finishing equipment can help offset the lack of finishability. Also, refer to Section 3.4.2.

**4.7.2** *Coarse aggregates*—In proportioning normal-strength concrete mixtures, the maximum size of coarse aggregate is usually controlled by clearance requirements in the structure, and the optimum amount of coarse aggregate depends on the fineness modulus of the fine aggregate. In HSC, however, it has been found that the highest strengths for a given *w/cm* are obtained by using smaller maximum-size coarse aggregate. To maintain workability, this results in a lower volume fraction of coarse aggregate. The selection of coarse aggregate size and content for HSC, however, may be influenced by requirements such as modulus of elasticity, creep, shrinkage, and heat of hydration. For these cases, larger aggregate sizes may be more desirable. Also, refer to Section 3.4.3.

**4.7.3** *Proportioning aggregates*—The amounts of coarse aggregate suggested in Table 4.2 are recommended for initial proportioning. The values given represent the fractional volume of coarse aggregate in the dry-rodded condition as a function of the nominal maximum size and for fine aggregate with a fineness modulus between 2.5 and 3.2.

In general, the least amount of fine aggregate consistent with necessary workability gives the best strength for a given paste. Mixtures with objectionably high coarse aggregate contents, however, may exhibit poor pumpability or may be significantly more prone to segregation during placement and consolidation.

## 4.8—Proportioning with supplementary cementitious materials and chemical admixtures

**4.8.1** *Supplementary cementitious materials*—High-strength mixtures have been successfully made with ternary blends consisting of highly reactive SCMs such as silica fume or HRM used in combination with materials such as fly ash and slag cement (Caldarone et al. 1994). Silica fume and HRM are commonly used at 5 to 15% by mass of the total cementitious materials content. In addition, high-strength mixtures have been produced using ternary blends composed

**Table 4.2—Recommended volume of coarse aggregate per unit volume of concrete***

| | Optimum coarse aggregate contents for nominal maximum sizes of aggregates to be used with sand with fineness modulus (FM) of 2.5 to 3.2 | | | |
|---|---|---|---|---|
| Nominal maximum size, in. | 3/8 | 1/2 | 3/4 | 1 |
| Fractional volume† of oven-dry rodded coarse aggregate | 0.65 | 0.68 | 0.72 | 0.75 |

*Table 4.2 taken from ACI 211.4R-93, Table 4.3.3.

†Volumes are based on aggregates in oven-dry rodded condition as described in ASTM C29 for unit weight of aggregates.

Notes: Refer to ASTM C136 for calculation of fineness modulus. 1 in. = 25.4 mm.

of portland cement, conventional fly ash, and ultra-fine fly ash (Obla et al. 2001).

Using fly ash often causes a slight reduction in the water demand of the mixture. Although generally ground finer than portland cement, the water demand of slag cement is usually about the same as that of portland cement. The opposite relationship has been found for other pozzolans. Dosages above approximately 5% of total cementitious material silica fume, for example, increase water demand, which makes the use of HRWRAs a requirement. Proprietary products containing silica fume may include carefully balanced chemical admixtures as well (Wolsiefer 1984). These SCMs often have other characteristics that are beneficial for HSC applications, such as temperature control, enhanced workability, or both.

**4.8.2** *Chemical admixtures*—Chemical admixture specifications are covered in ASTM C494/C494M. Advancements in chemical admixture technology have contributed significantly to the evolution of HSC. Chemical admixtures are used to control consistency (slump or slump flow), setting, rate of slump loss, water demand, rate of strength gain, and the effects of elevated temperatures.

**4.8.2.1** *Conventional and mid-range water-reducing admixtures (MRWRAs)*—The amount of conventional or mid-range water-reducing admixtures used in HSC varies depending upon the particular admixture and application. In addition to controlling water demand, the ability of these admixtures to control the rate of hydration as it relates to strength is of critical importance in the successful production of HSC.

Conventional WRAs generally reduce water demand approximately 5 to 10%. Mid-range water-reducing admixtures are designed to be used at higher dosages than conventional WRAs, and can reduce water demand by as much as 18% without the retardation associated with using higher dosages of conventional WRAs.

Generally, set-neutral WRAs or accelerating WRAs will not be as beneficial to long-term strength development as WRAs that retard setting. As the specified design strength increases, the ability of set-retarding admixtures to effectively control hydration as it relates to strength becomes increasingly important.

**4.8.2.2** *High-range water-reducing admixtures (HRWRAs)*—High-range water-reducing admixtures are frequently called superplasticizers, and are classified in ASTM C494/C494M as Types F and G. Water adjustments to HSC made with HRWRAs have been similar to those adjustments made when conventional WRAs are used. These adjustments have typically been larger due to the larger amount of water reduction, approximately 12 to 30%.

Self-consolidating HSC mixtures are frequently produced using HRWRAs in conjunction with viscosity-modifying admixtures, such as cellulose ether or welan or diutan gum (ACI 212.4R; BASF 2008). Generally, slump retention, batch-to-batch slump uniformity, and admixture efficiency can be increased when concrete is proportioned with a sufficient quantity of water such that measurable slump is produced without the HRWRA. For example, a mixture proportioned with enough water to produce a 1 to 2 in. (25 to 50 mm) slump (without the chemical admixture) would be expected to exhibit longer slump retention than a mixture proportioned with less water.

Unlike earlier melamine or naphthalene-based HRWRAs that performed more consistently after prewetting the cement, new-generation HRWRAs based on polycarboxylate chemistry can frequently be introduced without prewetting the cement. Therefore, once the water content has been established, new-generation admixtures can be introduced during the beginning phases of batching rather than at the end.

In HSC mixtures, HRWRAs are primarily used to lower the *w/cm* while maintaining workability. Due to the relatively large quantity of liquid that is frequently added in the form of HRWRAs, the water content of these admixtures should be included in the calculation of the *w/cm*.

**4.8.3** *Combinations*—Nearly all HSCs incorporate combinations of SCMs and chemical admixtures. Changes in the type, quantities, and combinations of these materials can affect both the fresh and hardened properties of HSC. Therefore, as discussed in Chapter 3, special attention has been given to their effects. Careful adjustments to mixture proportions have been made when there have been changes in admixture type, quantities, or combinations. Material characteristics have varied extensively, making experimentation with the candidate materials necessary.

High-range water-reducing admixtures frequently perform better in HSCs when used in combination with conventional WRAs or retarding WRAs. This is because of the increased slump retention and hydration control achievable through their use.

## 4.9—Workability

Workability is defined as “that property of freshly mixed concrete or mortar that determines the ease with which it can be mixed, placed, consolidated, and finished to a homogeneous condition” (American Concrete Institute 2009).

**4.9.1** *Consistency*—ASTM C143/C143M describes a standard test method for determining the slump of hydraulic-cement concrete that has been used to quantify the consistency of plastic, cohesive concrete mixtures. This test method is generally not relevant to stiff mixtures having measured slump values below 1/2 in. (13 mm), or flowing concrete mixtures having measured slump values above 7-1/2 in. (190 mm).

Other test methods such as the Vebe consistometer have been used with very stiff mixtures and may be a better aid in evaluating mixture proportions for some HSCs. Slump flow or spread is more relevant for determining the consistency of flowing or self-consolidating concretes than is the slump test (Aggoun et al. 2002).

Without uniform placement, structural integrity may be compromised. Without proper attention, high-strength mixtures tend to exhibit more early stiffening than lower-strength concrete. Concrete should be discharged before the mixture becomes unworkable. If adjustments in the field become necessary, it should be done using compatible chemical admixtures, not retempering water.

**4.9.2** *Placeability*—High-strength concrete, often designed with 1/2 in. (13 mm) or smaller nominal maximum-size aggregate and with a high cementitious material content, is inherently placeable provided that proper attention is given to optimizing the ratio of fine-to-coarse aggregate. Local material characteristics can have a marked effect on mixture proportions. The particle size distribution of cementitious fines can influence the character of the mixture. Admixtures have been found to significantly improve the placeability of HSC mixtures.

Placeability has been evaluated in mock-up forms before final approval of the mixture proportions. At that time, placement procedures, consolidation methods, and scheduling should be established because they can greatly affect the end product and will influence the apparent placeability of the mixture.

**4.9.3** *Flow properties and cohesion*—Slump values needed for desired flow characteristics can be designed for the concrete; however, full attention should 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 negative effects on flow and increase water demand for placeability with a corresponding reduction in strength.

Stickiness is inherent in mixtures with high cementitious materials contents. Certain cements or combinations of cementitious materials and admixtures have been found to cause undue stickiness that impairs workability. The cementitious materials content of the mixture has normally been the minimum quantity required for strength development combined with the maximum quantity of coarse aggregate within the requirements for workability. Using a MRWRA in addition to a HRWRA may reduce stickiness and improve workability of HSC (Nmai et al. 1998).

Mixtures that were designed properly but appear to change in character and become stickier should be considered suspect and quickly checked for proportions, possible false setting of cement, undesirable entrained air, or other changes. A change in the character of a high-strength mixture could be a warning sign for quality control. This is an example where a subjective judgment may sometimes be as meaningful as quantitative parameters.

## 4.10—Trial batches

Frequently, the development of a HSC mixture requires a large number of trial batches. Because each locality and project is unique, a number of laboratory and field evaluations are frequently necessary to develop mixtures having suitable materials and proportions (Hester and Leming 1989). To minimize the number of trial batches needed to define the optimum combination and quantity of materials, a statistical approach using a central-composite design technique has been used on some projects (Luciano et al. 1991).

In addition to laboratory trial batches, larger-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 that will be used in the work. To avoid accidental testing bias, some investigators have sequenced trial mixtures in a randomized order.

**4.10.1** *Laboratory trial batch investigations*—Laboratory trial batches are prepared to achieve several goals. They should be prepared according to ASTM C192/C192M. In addition, timing, handling, and environmental conditions similar to those that are likely to be encountered in the field should be considered in the evaluation process. Often, the mixing and resting periods prescribed in ASTM C192/C192M require modification for a longer final mixing time.

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 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 relevant characteristics of those mixtures. Strength characteristics at various test ages may be defined. Rate of slump loss, amount of bleeding, segregation, and setting time can be evaluated. The density (unit weight) of the mixture should be determined. Density monitoring can be a valuable quality control tool. Structural properties such as shrinkage and modulus of elasticity may also be determined. Although degrees of workability and placeability may be difficult to measure, at least a subjective evaluation should be attempted.

**4.10.2** *Field-production trial batches*—Once a desirable mixture has been formulated in the laboratory, field testing with production-sized batches is recommended. Laboratory trial batches frequently exhibit significantly higher strength than can be reasonably achieved in production, as shown in Fig. 4.3. Actual field water demand, and therefore concrete yield and *w/cm*, has varied significantly from laboratory design. Ambient temperatures and weather conditions have affected concrete performance. Practicality of production and of quality-control procedures has been evaluated better when production-sized trial batches were prepared using the equipment and personnel to be used in the actual work.

# CHAPTER 5—ORDERING, BATCHING, MIXING, TRANSPORTING, PLACING, CURING, AND QUALITY-CONTROL PROCEDURES

## 5.1—Introduction

Qualified producers, contractors, and testing laboratories are essential for successful construction with HSC. The batching, mixing, transporting, placing, and quality-control procedures for HSC are not different in principle from those procedures used for lower-strength concrete; however, some changes, refinements, and 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 HSC, it is critical. Because the production of HSC will normally involve using relatively large cementitious materials contents with resulting greater heat generation, some of the recommendations on production, delivery, placing, and curing given in ACI 305R may also be applicable.

## 5.2—Ordering

**5.2.1** *Batch size*—When ordering HSC, every effort should be made to divide the quantity of concrete produced and delivered into equally sized batches to help ensure both uniformity and consistency. For example, if 10 $yd^3$ (8 $m^3$) of concrete is required for a given placement, and the delivery equipment has a rated capacity of 9 $yd^3$ (7 $m^3$) each, it would be more prudent to batch two 5 $yd^3$ (3.5 $m^3$) batches rather than one 9 $yd^3$ (7 $m^3$) batch and one 1 $yd^3$ (0.8 $m^3$).

**5.2.2** *Lead time*—Orders for HSC should be placed at least several days in advance to allow ample time to inventory raw materials and schedule testing and inspection services.

## 5.3—Batching

**5.3.1** *Control, handling, and storage of materials*—Quality control, handling, and storage of raw materials need not be substantially different from the procedures used for conventional concrete as outlined in ACI 304R. As with all concrete, proper stockpiling of aggregates, uniformity of moisture in the batching process, and good sampling practice are essential.

In the committee's opinion, the moisture content of aggregates should be uniform, and the temperature of all ingredients should be kept such that the mixture design temperature is maintained between 65 to 75°F (18 to 24°C). The moisture content of fine aggregates should be monitored continuously through the use of calibrated moisture metering devices. If not automatically monitored, the moisture content of coarse aggregates should be routinely determined at least once per day, or whenever it is suspected that the moisture content is different from the value being used during production. It may be prudent to place a maximum limit of 150°F (66°C) on the temperature of the cementitious materials as batched, particularly under hot-weather concreting conditions. Maximum temperatures for concrete are specified in ACI 305R and ACI 301.

**5.3.2** *Measuring*—Materials for the production of HSC may be batched in manual, semiautomatic, or automatic plants. To maintain the proper *w/cm* necessary to secure HSC, accurate moisture determination in the fine aggregate is essential.

**5.3.3** *Charging of materials*—Batching procedures have important effects on the ease of producing thoroughly mixed, uniform concrete in both stationary and truck mixtures. The uniformity of concrete produced in central mixers is generally enhanced by loading the aggregate, cement, and water simultaneously (ribbon loading). High-range water-reducing admixtures are another consideration, because these admixtures are likely to be used in the production of HSC. Tests have shown (Ramachandran et al. 1998) that HRWRAs consisting of naphthalene or melamine condensates 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 mixed. Newer-generation polycarboxylic-based high-range water-reducers offer the ability to be introduced with the initial mixing water while providing effective water reduction and consistency. If there is evidence of improper mixing and nonuniform slump during discharge, procedures used to charge truck and central mixtures should be modified to ensure uniformity of mixing as required by ASTM C94/C94M.

## 5.4—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 304R. Experience and tests (Saucier 1968; Strehlow 1973) have indicated that HSC can be produced in all common types of mixers. Under some circumstances with HSC, however, it may prove beneficial to reduce the batch size below the rated capacity to ensure efficient mixing. High-strength concrete may be mixed at the job site in a truck mixer. It should not be assumed, however, that all truck mixers can successfully mix HSC, especially if the concrete has very low slump.

Close job control is essential for high-strength ready mixed concrete operations to avoid excessive waiting times at the job site due to slow placing operations. Water-reducing, set-retarding, high-range water-reducing, or a combination of these admixture types, have been used effectively to control water demand, rate of hydration, and slump loss, and increase strength. Water-reducing and set-retarding admixtures are usually introduced at the batching facility. High-range water-reducing admixtures have been introduced at the batching facility or at the site. If a HRWRA is added at the site, a truck-mounted dispenser or a field dispenser capable of measuring the quantity added is usually required.

**5.4.2** *Mixer performance*—The performance of mixers is usually determined by a series of uniformity tests performed in accordance with ASTM C94/C94M. Testing for mixer uniformity involves obtaining and testing samples from the first and last portion of the batch. Six tests are conducted: density, air content, slump, coarse aggregate content, yield, and 7-day compressive strength. Test results conforming to the limits of five of the six tests listed indicate uniform concrete within the limits of ASTM C94/C94M. It is important for the supplier of HSC to periodically check mixer performance and efficiency before production mixing.

**5.4.3** *Mixing time*—The mixing time required is based on the ability of the mixing unit to produce uniform concrete both within a batch and between batches. Manufacturers' recommendations, ACI 304R, and usual specifications, such as 1 minute for 1 $yd^3$ (0.8 $m^3$) plus 1/4 minute for each additional cubic yard of capacity, are used as satisfactory guides for establishing mixing time. Otherwise, mixing times can be based on the results of mixer performance tests. Mixing time is measured from the time all ingredients are in the mixer. Prolonged mixing may cause moisture loss and result in lower workability; if retempering is used to restore slump, strength potential can be reduced.

## 5.5—Transporting

**5.5.1** *General considerations*—High-strength concrete can be transported by a variety of methods and equipment, such as truck mixers, stationary truck bodies with agitators, pipelines, hoses, 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. Delivery time should be reduced to a minimum and special attention paid to scheduling and placing to avoid delays in unloading. When possible, batching facilities should be located close to the job site to reduce haul time.

**5.5.2** *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. At the job site, water is added and mixing is completed. This method evolved as a solution to long hauls and placing delays and is adaptable to the production of HSC where it is desirable to retain workability as long as possible. Free moisture in the aggregates, however, which is part of the mixing water, may cause some hydration to occur before mixing water is added.

**5.5.3** *Stationary truck body with and without agitator*—These transportation units 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 or from the side when the body is tilted. A discharge gate and vibrators mounted on the body are provided at the point of discharge. An apparatus that uniformly blends the concrete, as it is unloaded, is desirable. Water is not added to the truck body, however, because adequate mixing cannot be obtained with the agitator alone.

**5.5.4** *Pumping*—High-strength concrete will, in many cases, be very suitable for pump placement. Pumps are available that can handle low-slump mixtures and provide high pumping pressure. High-strength concrete is likely to have a high cementitious materials content and small maximum-size aggregate—both factors facilitate concrete pumping. Chapter 9 of ACI 304R provides guidance for the use of pumps for transporting HSC. 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 rigid lines and flexible pipe or hose to permit placing over a large area directly into the forms without rehandling. Direct communication between the pump operator and the concrete placing crew is essential. Continuous pumping is desirable because if the pump is stopped, restarting the movement of the concrete in the line may be difficult or impossible.

**5.5.5** *Belt conveyor*—Using belt conveyors to transport concrete has become normal practice in concrete construction. Guidance for using conveyors is given in ACI 304R. The conveyors should be adequately supported to obtain smooth, nonvibrating travel along the belt. The angle of incline or decline should be controlled to eliminate the tendency for coarse aggregate to segregate from the mortar fraction. Because the practical slump range for belt transport of concrete is 1 to 4 in. (25 to 100 mm), belts may be used to move HSC 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, proper planning, timing, and quality control are essential.

## 5.6—Placing procedures

**5.6.1** *Preparations*—Delivery of concrete to the job site should be scheduled so it will be placed promptly upon arrival. Equipment for placing the concrete should have adequate capacity to perform its functions efficiently so that placement delays are minimized. There should be ample vibration equipment and personnel to consolidate the concrete quickly after placement in congested areas. All placing equipment should undergo routine maintenance and should always be in first-class operating condition. Breakdowns or delays that stop or slow placement can seriously affect work quality. Delaying the placement of HSC can result in a greater loss in workability over time. Provisions should be made for an adequate number of standby vibrators; there should be at least one standby for each three vibrators in use. An HSC placing operation is in serious trouble, especially in hot weather, when vibration equipment fails and the standby equipment is inadequate.

**5.6.2** *Equipment*—A basic requirement for placing equipment is that the quality of the concrete, in terms of *w/cm*, slump, air content, and homogeneity, should be preserved. Selection of equipment should be based on its capability for efficiently handling concrete so that it can be readily consolidated. 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 should be very steep to prevent blockages. High-strength concrete should not be allowed to remain in buckets for extended periods of time, as delays can cause difficulty in discharging.

**5.6.3** *Consolidation*—Consolidation is important if the potential strength of HSC is to be achieved. The provisions of ACI 309R should be followed. High-strength concrete can be very sticky material; effective consolidation procedures may well start with mixture proportioning. Self-consolidating mixtures are gaining in popularity, particularly in precast applications, and require no vibration. Concrete mixtures requiring vibration should be vibrated as quickly as possible after placement into the forms. High-frequency vibrators should be small enough to allow clearance between the vibrating head and reinforcing steel. Coarse sands have been found to provide the best workability (Blick 1973). Nawy (2001) recommends a fineness modulus in the range of 2.5 to 3.2 for HSC to facilitate workability. The importance of full consolidation cannot be overstated as it is required for HSC to achieve its full potential.

**5.6.4** *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 shear walls in any given story be placed with the same strength concrete. For formwork economy, no changes in column size in 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 HSC 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 (CCHRB 1977).

## 5.7—Curing

**5.7.1** *Need for curing*—Curing is the process of maintaining a satisfactory moisture condition and a favorable temperature in concrete during the hydration period of the cementitious materials so that potential properties of the concrete can develop. Curing is essential in the production of quality concrete, and it is critical to the production of HSC. Curing of HSC is even more important than curing normal-strength concrete (Kosmatka et al. 2001). Underwater curing of very high-strength concrete test cylinders is not required, as curing in a moist room has been shown to be sufficient (Burg et al. 1999). The potential strength and durability of concrete will be fully developed only if it is properly cured for an adequate period before being placed in service. Also, cast-in-place HSC should be water-cured at an early age because 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 (Neville 1996).

**5.7.2** *Type of curing*—The potential strength and durability of HSC will fully develop only if the concrete is properly cured for an adequate period. Acceptable curing methods are discussed in ACI 308R. High-strength concretes are extremely dense, so appropriate curing methods for various structural elements should be selected in advance. Water-curing cast-in-place HSC is highly recommended due to the low *w/cm* employed. At a *w/cm* below 0.40, the ultimate degree of hydration is significantly reduced if an external supply of water is not provided. Water curing allows more cement to hydrate (Burg et al. 1999). Klieger (1957) reported that, for low *w/c* concretes, it is more advantageous to supply additional water during curing than is the case with higher *w/c* concretes. For concretes with a *w/c* 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 (6 to 7 MPa) greater at 28 days than that of comparable specimens made with dry aggregates and cured under damp burlap. Farny and Panarese (1994) reported that moist curing for 28 to 90 days has shown to increase strength. Klieger also noted that, although early strength is increased by elevated temperatures during mixing and early curing, later strengths are reduced by such high temperatures. Work by Pfeifer and Ladgren (1981), however, has shown that later strengths may have only minor reductions if the heat is not applied until after setting. Others (Saucier et al. 1965; Price 1951) have reported that moist-curing for 28 days and thereafter in air was highly beneficial in securing HSC at 90 days.

**5.7.3** *Methods of curing*—The most effective, but seldom used, method of water-curing consists of total immersion of the finished concrete unit in water. Ponding 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 at very early ages. 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 assist in retaining the original moisture in the concrete, but do not provide additional moisture nor completely prevent moisture loss. Monomolecular film-forming agents have been effectively employed for interim curing before deployment of final curing procedures for exposed surfaces susceptible to drying during finishing. These so-called “evaporation reducers” are not to be used as an aid to finishing.

## 5.8—Quality control and testing

**5.8.1** *Introduction*—In previous versions of this document, Chapter 4 covered information related to quality control and testing practices for HSC; since its last revision, Committee 363 has prepared a guide on quality control and testing HSC (ACI 363.2R). The information in this section briefly covers quality control and testing practices. For a detailed discussion of this subject, refer to ACI 363.2R.

**5.8.2** *Planning*—Thorough planning and teamwork by the inspector, contractor, architect/engineer, producer, and owner are essential for the successful use of HSC. A preconstruction meeting is essential to clarify roles of the members of the construction team and review the planned quality control and testing program. Where historical data are not available, materials and mixture proportions should be evaluated in the laboratory to determine appropriate material proportions. After the work has been completed in the laboratory, production-sized batches are recommended because laboratory trial batches sometimes exhibit strengths and other properties different from those achieved in production. Bidders should be prequalified before the award of a supply contract for concrete with a specified strength of 10,000 psi (70 MPa) or higher, or at least 1000 psi (7 MPa) higher than previously produced in the market local to the project. Qualified suppliers can be selected based on their successful preconstruction trials.

**5.8.3** *Quality assurance and quality control*—Quality assurance (QA) and quality control (QC) are defined as follows (American Concrete Institute 2009):

**quality assurance**—actions taken by an organization to provide and document assurance that what is being done and what is being provided are in accordance with the contract documents and standards of good practice for the work.

**quality control**—actions taken by an organization to provide control and documentation over what is being done and what is being provided so that the applicable standard of good practice and the contract documents for the work are followed.

The duties of QA/QC personnel should be defined clearly in the contract documents, based on the principles set out in the definitions.

**5.8.3.1** *Concrete plant*—QA/QC personnel should concentrate their efforts at the concrete plant until consistently acceptable production is achieved. Thereafter, spot checking the plant is recommended, unless the complexities of the project demand full-time monitoring. At the concrete plant, QA/QC personnel should ensure that the facilities, moisture meters, scales, and mixers meet the project specification requirements and those materials and procedures are as established in the planning stages.

**5.8.3.2** *Delivery*—QA/QC personnel should recognize that prolonged mixing will cause slump loss and reduced workability. Adequate job control should be established to prevent delays. Truck mixers used to transport HSC should be inspected regularly and certified to comply with the checklist requirements of the NRMCA Certification of Ready Mixed Concrete Production Facilities. Truck mixers should be equipped with a drum revolution counter, and their fins should comply with NRMCA criteria. The concrete truck driver should provide a delivery ticket that contains the information specified in ASTM C94/C94M. Every ticket should be reviewed by the inspector before discharge of concrete.

**5.8.3.3** *Placing*—Preparations at the project site are important. In particular, the contractor should be ready for placing the first truckload of concrete. QA/QC personnel should verify that forms, reinforcing steel, and embedded items are ready and that the placing equipment and vibration equipment are in working order before placing concrete. In construction, different strength concretes are often placed adjacent to one another. QA/QC personnel should be aware of the exact location for each approved mixture. When two or more concrete mixtures are being used in the same placement, it is mandatory that sufficient control be exercised at the point of discharge from each truck to ensure that the intended concrete is placed as specified.

**5.8.4** *Testing*—Measurement of mechanical properties during construction provides the basic information needed to evaluate whether specified strength is achieved and the concrete is acceptable. Experience indicates that the measured strength of HSC is more sensitive to testing variables compared with normal-strength concrete. Therefore, the quality of these measurements is very important. Testing and acceptance standards based on past studies may not be applicable to HSC. Sanchez and Hester (1990) pointed out the requirement for strict attention to quality control on projects incorporating concrete with strengths of 12,000 to 14,000 psi (85 to 100 MPa). Inadequate testing techniques and inter-laboratory inconsistencies have been found to cause more problems than have actually occurred with the concrete. Hester (1980) found differences in measured compressive strengths between laboratories to be as high as 10%, depending on the mixture and laboratories used.

Statistical methods are an excellent means to evaluate HSC. To be valid, the data (slump, density, temperature, air content, and strength) should be derived from samples obtained through a random sampling plan designed to reduce the possibility that choice (bias) will be exercised by the testing technician. Samples obtained should represent the quality of the concrete supplied; therefore, composite samples should be taken in accordance with ASTM C172. These samples are representative of the quality of the concrete delivered to the site and may not truly represent the quality of the concrete in the structure, which may be affected by site placing and curing methods. If additional samples are required to check the quality of the concrete at the point of placement (as in pumped concrete), this should be established at the preconstruction meeting.

Because much of the interest in high-strength structural concrete is limited to compressive strength and modulus of elasticity, these properties are of primary concern. Standard ASTM test methods are followed except where changes are dictated by the needs of the HSC. Results of an interlaboratory test program conducted by Burg et al. (1999) demonstrated that the current requirements for testing platens, capping materials, or specimen end conditions may be inadequate for testing HSC. For HSC, greater consideration should be given to testing-related factors, including specimen size and shape, mold type, consolidation method, handling and curing in the field and laboratory, specimen preparation, cap thickness, and testing apparatus (Lobo et al. 1994; Vichit-Vadakan et al. 1998). A detailed discussion of these factors is provided in ACI 363.2R.

## CHAPTER 6—PROPERTIES OF HIGH-STRENGTH CONCRETE

### 6.1—Introduction

Traditionally, concrete properties such as stress-strain relationship, modulus of elasticity, tensile strength, shear strength, and bond strength have been expressed in terms of the uniaxial compressive strength of 6 x 12 in. (152 x 305 mm) cylinders. The expressions have been based on experimental data of concrete with compressive strengths less than 8000 psi (55 MPa). For HSC, however, the uniaxial compressive strength is usually much higher than 8000 psi (55 MPa). Thus, the compressive strength is often obtained by using 4 x 8 in. (102 x 204 mm) cylinders because of the capacity limitation of testing machines. When 4 x 8 in. (102 x 204 mm) cylinders were cast in three layers, compressive strengths were generally slightly higher than that determined from 6 x 12 in. (152 x 305 mm) cylinders. The majority of the test data indicated that the difference may vary from 1 to 5% (Carino et al. 1994; Burg et al. 1999). ACI 363.2R presents more discussion on size effect and indicates 4 x 8 in. (102 x 204 mm) cylinders are suitable for acceptance testing purposes provided that the same size specimens were used to evaluate trial mixtures.

Various properties of HSC are reviewed in the following sections, and the applicability of current and proposed expressions for estimating properties of HSC is examined.

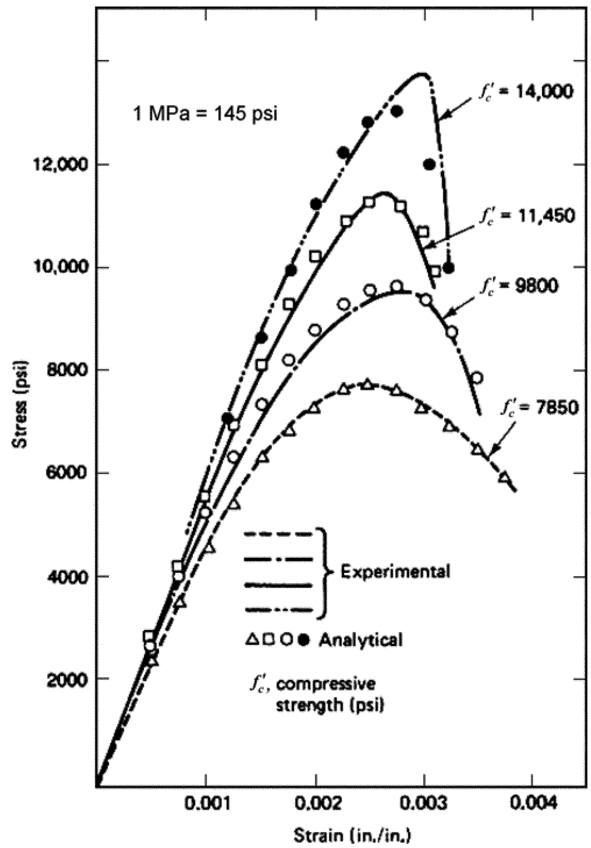


*Fig. 6.1—Stress-strain curves of concrete in compression (adapted from Nawy [2003]).*

### 6.2—Stress-strain behavior in uniaxial compression

Axial stress-versus-strain curves for concrete of compressive strength up to 14,000 psi (97 MPa) are shown in Fig. 6.1. The shape of the ascending part of the stress-strain curve is more linear and steeper for HSC, and the strain at the maximum stress is slightly higher for HSC (Jansen et al. 1995; Shah et al. 1981; Shah 1981). The slope of the descending part becomes steeper for HSC compared with normal 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 HSC. (Wang et al. 1978a; Shah et al. 1981; Holm 1980).

As there are no established standards for obtaining the complete stress-strain curves for concrete and the descending

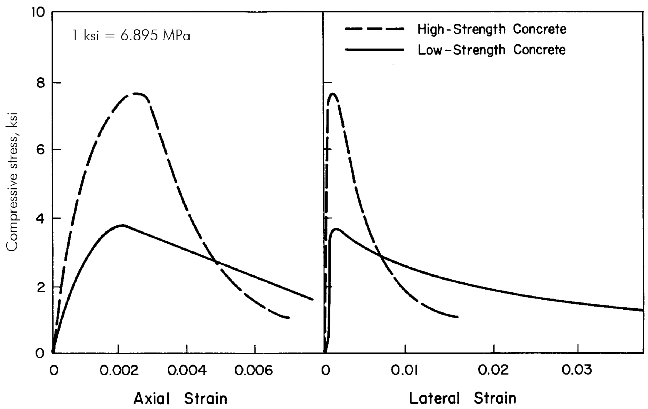


*Fig. 6.2—Axial stress versus axial strain and lateral strain for plain normal-density concrete (adapted from Ahman and Shah [1982a]).*

branch is dependent on the test method employed, the stress-strain curve should only be used for comparison purposes.

High-strength concrete exhibits less internal microcracking than lower-strength concrete for a given imposed axial strain (Carrasquillo et al. 1981). As a result, the relative increase in lateral strains is less for HSC (Fig. 6.2) (Ahmad and Shah 1982a,b). The lower relative lateral expansion during the inelastic range may mean that the effects of triaxial stresses will be proportionally different for HSC. For example, the influence of hoop reinforcement is observed to be different for HSC (Ahmad and Shah 1982a). It was reported that the effectiveness of spiral reinforcement is less for HSC than for normal-strength concrete (Ahmad and Shah 1982a).

## 6.3—Modulus of elasticity

In 1934, Thoman and Raeder reported values for the modulus of elasticity determined as the slope of the tangent to the stress-strain curve in uniaxial compression at 25% of maximum stress. The values varied from $4.2 \times 10^6$ to $5.2 \times 10^6$ psi (29 to 36 GPa) for concretes having compressive strengths ranging from 10,000 to 11,000 psi (69 to 76 MPa). Many other investigators (Ahmad and Shah 1985; Smith et al. 1964; Freedman 1971; Teychenné et al. 1978; Ahmad 1981; Burg and Ost 1994; Zia et al. 1993a,b; Iravani 1996; Myers and Carrasquillo 1998; Mokhtarzadeh and French 2000a) have reported values for the modulus of elasticity of HSCs on the order of $4.5 \times 10^6$ to $7.5 \times 10^6$ psi (31 to 52 GPa) depending on the method of determining the modulus and the mixture constituents and proportions. A comparison of several reported empirical equations including the expression given in ACI 318-05, for a concrete density of 145 lb/ft$^3$ (2346 kg/m$^3$) is presented in Fig. 6.3. No single empirical expression subsequently presented in this section estimates the modulus of elasticity for concretes with compressive strengths over 8000 psi (55 MPa) to a high degree of accuracy for the data set given in Fig. 6.3.

A correlation between the modulus of elasticity $E_c$ and the compressive strength $f_c'$ for normal-density concretes has been reported by several researchers as illustrated in Eq. (6-1) through (6-8).

$$E_c = 40{,}000(f_c')^{0.5} + 10^6 \text{ (psi) for } 3000 \text{ psi} < f_c' < 12{,}000 \text{ psi}$$

(Martinez et al. 1982)

$$E_c = 3320(f_c')^{0.5} + 6900 \text{ (MPa) for } 21 \text{ MPa} < f_c' < 83 \text{ MPa} \tag{6-1}$$

Equation (6-1) has generally proven to be a relatively reliable lower-bound expression (Fig. 6.3) for normal-density HSC based on most HSC data collected; however, it may be noted that studies have cited the concerns when using this expression in the case where it significantly underestimates the modulus of elasticity (Myers and Carrasquillo 1998; Gross and Burns 1999).

Several other recommendations for HSC have been proposed, including Eq. (6-2) by Cook (1989), Eq. (6-3) by Ahmad and Shah (1985), Eq. (6-4) by Berke et al. (1992), Eq. (6-5) by Tomosawa and Noguchi (1993), and Eq. (6-6) by Radain et al. (1993). Equation (6-7) is recommended in the FIP-CEB (1990) state-of-the-art report, and Eq. (6-8) reported by the NS 3473 concrete structures design rules (Norges Standardiseringsfund 1992). The "CEB-FIP Model Code 1990" relates the modulus of elasticity to the cube root of the compressive strength rather than the square root (CEB 1991)

$$E_c = w_c^{2.55}(f_c')^{0.315} \text{ (psi)}$$

$$E_c = 3.385 \times 10^{-5} w_c^{2.55}(f_c')^{0.315} \text{ (MPa)} \tag{6-2}$$

$$E_c = w_c^{2.5}(f_c')^{0.325} \text{ (psi) for } f_c' < 12{,}000 \text{ psi}$$

$$E_c = 3.385 \times 10^{-5} w_c^{2.5}(f_c')^{0.325} \text{ (MPa) for } f_c' < 84 \text{ MPa} \tag{6-3}$$

$$E_c = 2778(\text{CF}) + 6189(\text{SF}) + 452{,}545(\text{LN(age)}) + 1{,}796{,}695^* \text{ (psi)}$$

$$E_c = 32.297(\text{CF}) + 71.963(\text{SF}) + 3121(\text{LN(age)}) + 12{,}391^* \text{ (MPa)} \tag{6-4}$$

where CF and SF are variables for the aggregate type and inclusion of silica fume, respectively.

Note: 1 Such a high degree of accuracy based on the level of significant figures shown by the authors should not be expected in the committees' opinion due to the degree of scatter of modulus of elasticity data (Fig. 6.3).

$$E_c = 4.86 \times 10^6 k_1 \cdot k_2 (w_c/150)^2 (f_c'/8700)^{1/3} \quad \text{(psi)}$$

$$E_c = 3.35 \times 10^4 k_1 \cdot k_2 (w_c/2400)^2 (f_c'/60)^{1/3} \quad \text{(MPa)} \tag{6-5}$$

where $k_1$ and $k_2$ are variables for the aggregate type and inclusion of mineral admixture type, respectively.

$k_1$ = 1.2 for crushed limestone, calcined bauxite aggregates; = 0.95 for crushed quartzite, crushed

*Such a high degree of accuracy based on the level of significant figures shown by the authors should not be expected, in the committee's opinion, due to scatter of modulus of elasticity data (Fig. 6.3).

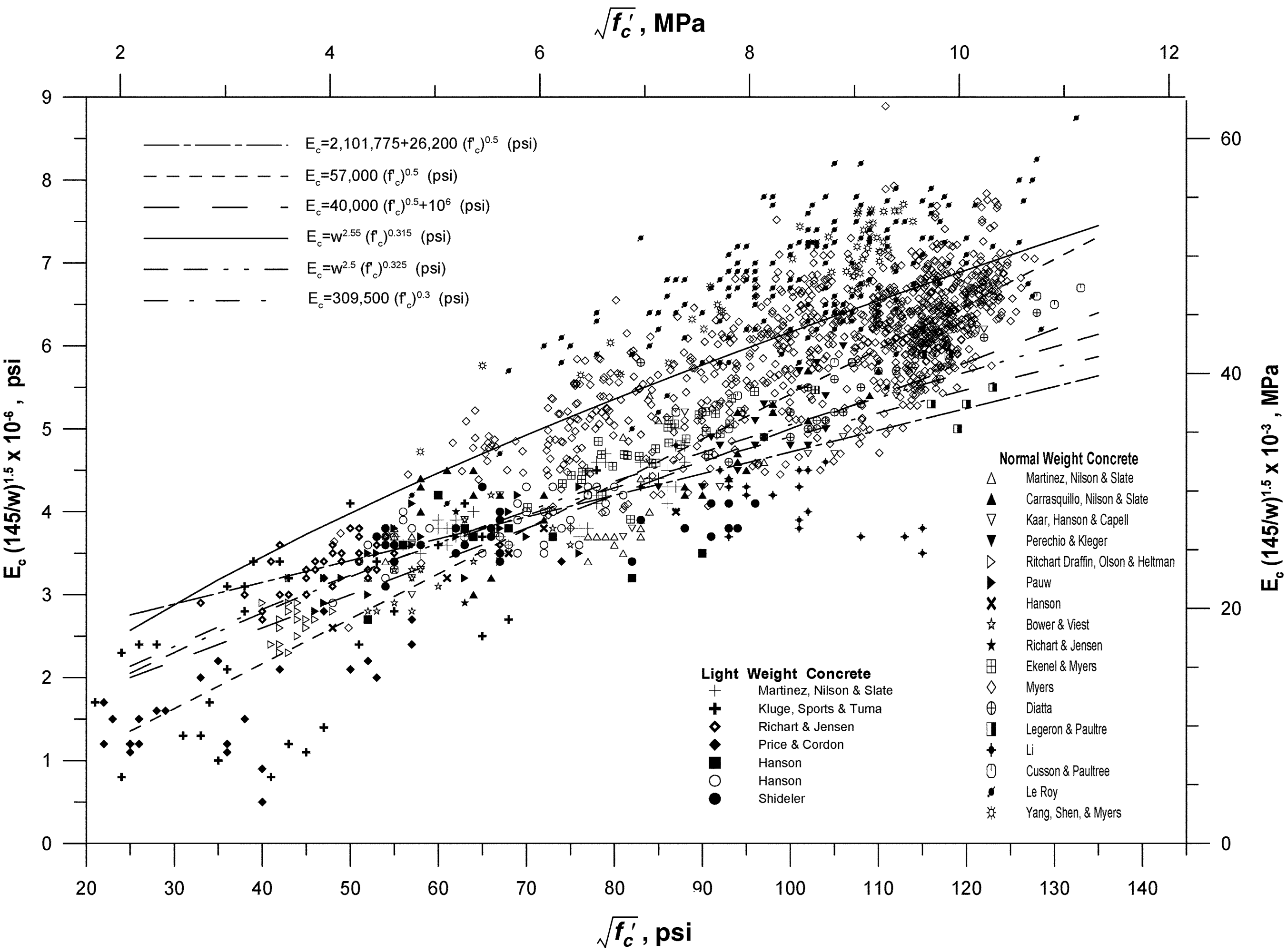


*Fig. 6.3—Modulus of elasticity versus square root of concrete strength, incorporating lower- and higher-strength concrete data (adapted from Myers and Yang [2004]).*

andesite, crushed basalt, crushed clay slate, and crushed cobblestone aggregates; = 1.0 for coarse aggregates other than above; and

$k_2$ = 0.95 for silica fume, slag cement, fly ash fume; = 1.10 for fly ash; = addition other than above

$$E_c = 2{,}101{,}775 + 26{,}200(f'_c)^{0.5\,\dagger} \quad \text{(psi)} \tag{6-6}$$

$$E_c = 14{,}495 + 2176(f'_c)^{0.5\,\dagger} \quad \text{(MPa)}$$

$$E_c = 593{,}400\alpha_\beta[(f_{ck} + 1160)/10]^{1/3} \text{ or}$$
$$593{,}400\alpha_\beta[f_{cm}/10]^{1/3} \text{ (psi) for } f'_c < 11{,}600 \text{ psi} \tag{6-7}$$

$$E_c = 21{,}500\alpha_\beta[(f_{ck} + 8)/10]^{1/3} \text{ or}$$
$$21{,}500\alpha_\beta[f_{cm}/10]^{1/3} \text{ (MPa) for } f'_c < 80 \text{ MPa}$$

†Such a high degree of accuracy based on the level of significant figures shown by the authors should not be expected, in the committee's opinion, due to scatter of modulus of elasticity data (Fig. 6.3).

where $\alpha_\beta$ is a variable for the aggregate type; $f_{ck}$ is the characteristic compressive strength of 6 x 12 in. (152 x 305 mm) cylinder; $f_{cm}$ is the compressive strength at 28 days of 6 x 12 in. (152 x 305 mm) cylinder; and $\alpha_\beta$ =1.2 for basalt, dense limestone aggregates, = 1.0 for quartzitic aggregates, = 0.9 for limestone aggregates, = 0.7 for sandstone aggregates.

$$E_c = 309{,}500 f'^{\,0.3\,\dagger}_c \text{ (psi) for 3600 psi} < f'_c < 12{,}300 \text{ psi} \tag{6-8}$$

$$E_c = 9500 f'^{\,0.3\,\dagger}_c \text{ (MPa) for 25 MPa} < f'_c < 85 \text{ MPa}$$

Curing conditions not only affect the compressive strength development as widely reported in normal- and high-strength concrete, but also other mechanical properties, including the modulus of elasticity. Table 6.1 compares the differences found in the modulus of elasticity based on curing condition by Myers and Carrasquillo (1998). This includes empirical relationships with and without a zero intercept.

Deviation from estimated values are highly dependent on the properties and proportions of the coarse aggregate as well as the curing condition, as illustrated in Fig. 6.4 and 6.5.

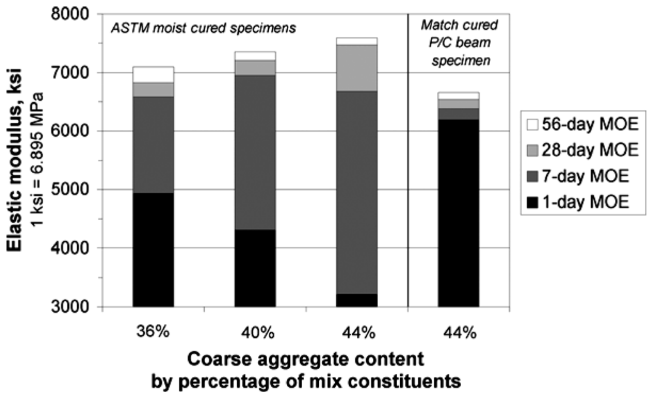


*Fig. 6.4—Modulus of elasticity versus coarse aggregate content by weight and curing condition (adapted from Myers [1999]).*

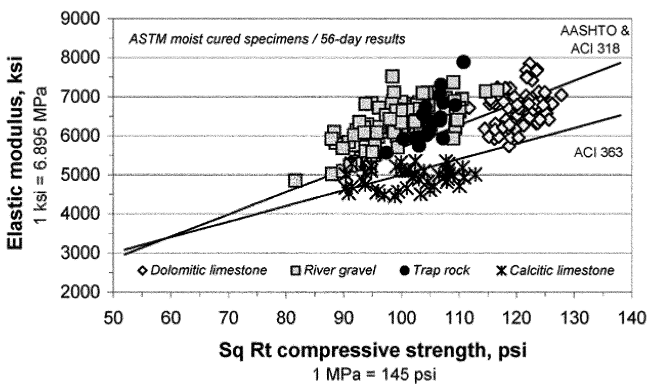


*Fig. 6.5—Modulus of elasticity versus square root compressive strength by coarse aggregate type (adapted from Myers [1999]).*

For example, higher values than estimated by Eq. (6-1) were reported by Russell and Corley (1978), Saucier et al. (1965), Pfeifer et al. (1971), Mokhtarzadeh and French (2000a), and Myers and Carrasquillo (1998).

Due to the significant influence of the aggregate type, content, and other mixture constituents, Myers and Carrasquillo (1998) recommended, and the committee concurs (ACI 363.2R), that the design engineer verify any modulus of elasticity that is assumed based on compressive strength for the design of HSC members through a trial field batching series on the specific mixture proportion design or by documented performance.

## 6.4—Poisson's ratio

Experimental data on values of Poisson's ratio for HSC are limited. Shideler (1957) and Carrasquillo et al. (1981) reported values for the Poisson's ratio of lightweight-aggregate HSC 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 (1978) reported values for the Poisson's ratio of normal-density HSCs (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 *w/c*.

**Table 6.1—Modulus of elasticity empirical equations reported based on curing condition**

| Curing condition Investigated | Empirical equation, psi With zero intercept | Without zero intercept |
|---|---|---|
| ASTM moist-cured cylinders | $E_c = 56{,}300 f_c'^{\,0.50}$ | $E_c = 38{,}200 f_c'^{\,0.50} + 2{,}110{,}000$ |
| Member-cured cylinders | $E_c = 55{,}050 f_c'^{\,0.50}$ | $E_c = 39{,}900 f_c'^{\,0.50} + 1{,}730{,}000$ |
| Match-cured cylinders | $E_c = 55{,}000 f_c'^{\,0.50}$ | $E_c = 17{,}200 f_c'^{\,0.50} + 4{,}250{,}000$ |

Note: 1 ksi = 1000 psi = 6.895 MPa.

**Table 6.2—Modulus of rupture empirical equations reported based on curing condition (Mokhtarzadeh and French 2000a)**

| Curing condition | Empirical equation, psi |
|---|---|
| ASTM moist-cured cylinders | $f_r = 5.92 f_c'^{\,0.57}$ |
| Steam-cured cylinders | $f_r = 23.57 f_c'^{\,0.4}$ |

Note: 1 ksi = 1000 psi = 6.895 MPa.

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

## 6.5—Modulus of rupture

The values reported by various investigators (Shideler 1957; Parrott 1969; Dewar 1964; Kaplan 1959b; Burg and Ost 1994; Iravani 1996; Mokhtarzadeh and French 2000a; Legeron and Paultre 2000) for the modulus of rupture of both lightweight and normal-density HSCs fall in the range of $7.5\sqrt{f_c'}$ to $12\sqrt{f_c'}$ (psi) [$0.62\sqrt{f_c'}$ to $0.99\sqrt{f_c'}$ (MPa)], where both the modulus of rupture and the compressive strength are expressed in psi. ACI 318-05 references Eq. (6-9) as its empirical model for the modulus of rupture of normal-density concrete. Equation (6-10) was recommended by Carrasquillo et al. (1982) for the estimation of modulus of rupture of normal-density concrete from compressive strength, as shown in Fig. 6.6. Other models have also been proposed for various sets of HSC data as a grouping and individually based on curing condition, as shown in Eq. (6-11) and Table 6.2 (Mokhtarzadeh and French 2000a)

$$f_r = 7.5 f_c'^{\,0.5} \text{ (psi)}$$

$$f_r = 0.62 f_c'^{\,0.5} \text{ (MPa)} \tag{6-9}$$

$$f_r = 11.7 f_c'^{\,0.5} \text{ (psi) for 3000 psi} < f_c' < 12{,}000 \text{ psi}$$

$$f_r = 0.94 f_c'^{\,0.5} \text{ (MPa) for 21 MPa} < f_c' < 83 \text{ MPa} \tag{6-10}$$

$$f_r = 0.71 f_c'^{\,0.79} \text{ (psi) for moist and steam cured}$$

$$f_r = 0.25 f_c'^{\,0.79} \text{ (MPa) for moist and steam cured} \tag{6-11}$$

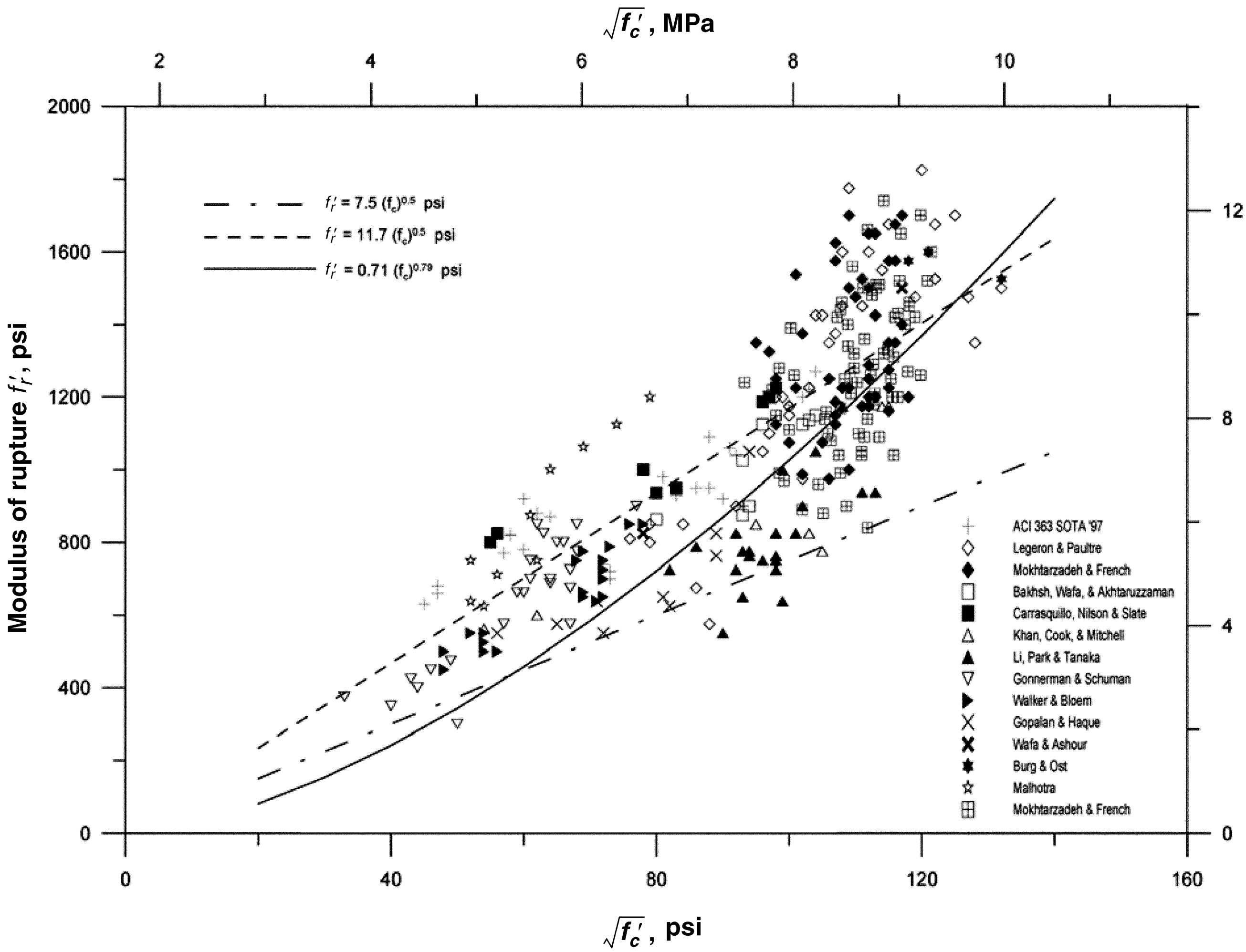


*Fig. 6.6—Relationships between modulus of rupture and square root of compressive strength (adapted from Myers and Yang [2004]).*

Equation (6-9) tends to underestimate modulus of rupture of normal-density concrete, and does not follow the trend of data presented in Fig. 6.6.

## 6.6—Splitting tensile strength

Dewar (1964) studied the relationship between the splitting tensile strength (cylinder splitting strength) and the compressive strength of concretes having compressive strengths of up to 12,100 psi (84 MPa) at 28 days. He concluded that at low strengths, the splitting tensile strength may be as high as 10% of the compressive strength, but at higher strengths, it may reduce to 5%. He observed that the tensile splitting strength was approximately 8% higher for crushed-rock-aggregate concrete than for gravel-aggregate concrete. In addition, he found that the splitting tensile strength was approximately 70% of the flexural strength at 28 days. ACI 318-05 references Eq. (6-12) as its empirical model for the splitting tensile strength of lightweight aggregate concrete. Carrasquillo et al. (1981) recommended Eq. (6-13) for estimating splitting tensile strength of normal-density concrete. Other researchers have reported empirical expressions that are relatively similar, including the effects of curing for particular data sets as shown in Eq. (6-14) and Table 6.3

**Table 6.3—Tensile splitting strength empirical equations reported based on curing condition**

| Curing condition | Empirical equation, psi (Myers and Carrasquillo 1998) | Empirical equation, psi (Mokhtarzadeh and French 2000a) |
|---|---|---|
| ASTM moist-cured cylinders | $f_{sp} = 8.58 f_c'^{0.50}$ | $f_{sp} = 0.42 f_c'^{0.79}$ |
| Member-cured cylinders | $f_{sp} = 8.66 f_c'^{0.50}$ | — |
| Match-cured cylinders | $f_{sp} = 10.9 f_c'^{0.50}$ | — |
| Steam-cured cylinders | — | $f_{sp} = 3.63 f_c'^{0.57}$ |

Note: 1 ksi = 1000 psi = 6.895 MPa.

$$f_{ct} = 6.7 f_c'^{0.5} \text{ (psi) for ACI 318-05} \tag{6-12}$$

$$f_{ct} = 0.56 f_c'^{0.5} \text{ (MPa) for ACI 318-05}$$

$$f_{sp} = 7.4 f_c'^{0.5} \text{ (psi) for 3000 psi} < f_c' < 12{,}000 \text{ psi} \tag{6-13}$$

$$f_{sp} = 0.59 f_c'^{0.5} \text{ (MPa) for 21 MPa} < f_c' < 83 \text{ MPa}$$

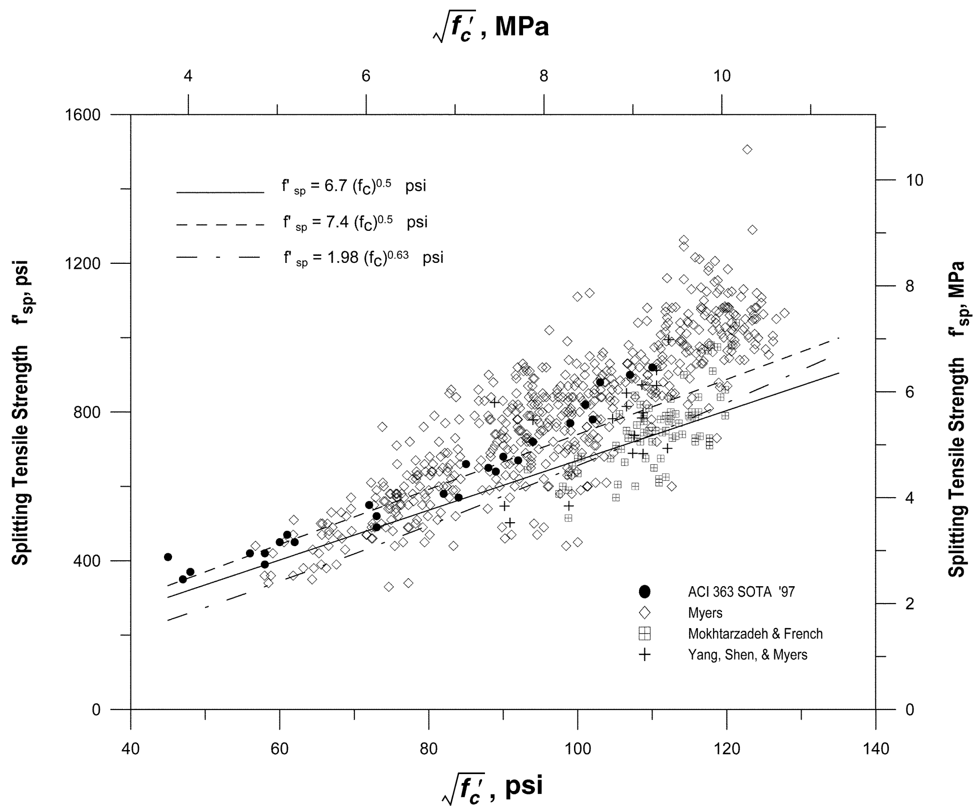


*Fig. 6.7—Relationships between splitting tensile strength and square root of compressive strength (adapted from Myers and Yang [2004]).*

$$f_{sp} = 1.98f_c'^{\,0.63} \text{ (psi) (Mokhtarzadeh and French 2000a)} \quad (6\text{-}14)$$

$$f_{sp} = 0.32f_c'^{\,0.63} \text{ (MPa)}$$

It may be noted that the empirical equations reported by Mokhtarzadeh and French (2000a) and Myers and Carrasquillo (1998) are both higher than Eq. (6-12). Splitting tensile strength results for HSC are shown in Fig. 6.7. As compressive strength increases, values for the splitting strength fall in the upper range of the empirical equations presented. Note that many researchers have shown that power function equations other than 0.5 fit the data better. It is apparent that the square root function does not follow the correct trend with increasing strength.

## 6.7—Fatigue behavior

The available data on the fatigue behavior of HSC is limited. Bennett and Muir (1967) studied the fatigue strength in axial compression of HSC with a 4 in. (100 mm) cube compressive strength of up to 11,100 psi (77 MPa) and found that after one million cycles, the strength of specimens subjected to repeated load at a minimum stress level of 1250 psi (9 MPa) varied between 66 and 71% of the static strength. 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 at a given number of cycles. To the extent that is known, the fatigue strength of HSC is the same as that for concretes of lower strengths.

## 6.8—Unit density

The measured values of the density of HSC are slightly higher than lower-strength concrete made with the same materials (Nawy 2001).

The "AASHTO LRFD Bridge Design Specifications" (AASHTO 2004) specifies that the density of plain normal-weight shall be taken as

$$145 \text{ lb/ft}^3 \text{ for } f_c' \leq 5000 \text{ psi}$$
$$\text{and } 140 + 0.001f_c' \text{ for } 5000 \text{ psi} < f_c' \leq 15{,}000 \text{ psi} \quad (6\text{-}15)$$

$$2323 \text{ kg/m}^3 \text{ for } f_c' \leq 34.5 \text{ MPa}$$
$$\text{and } 2243 + 6.9 \times 10^{-6}f_c' \text{ for } 34.5 \text{ MPa} < f_c' \leq 103.4 \text{ MPa}$$

## 6.9—Thermal properties

The thermal properties of HSCs fall within the approximate range for lower-strength concretes (Saucier et al. 1965; Parrott 1969). Quantities that have been measured are specific heat, diffusivity, thermal conductivity, and coefficient of thermal expansion (CTE). Gross and Burns (1999) reported on the CTE values observed in several high-strength mixtures, as shown in Fig. 6.8. Measured coefficients fell within the range of 4.0 to 7.3 με/°F (7.1 to 13.1 με/°C), which is similar to the range of values suggested by Mindess and Young (2003) for all concretes. Kowalsky et al. (2002) also reported test values of 4.2 με/°F and 4.9 με/°F (7.4 με/°C and 8.7 με/°C) from their studies of high-strength girders.

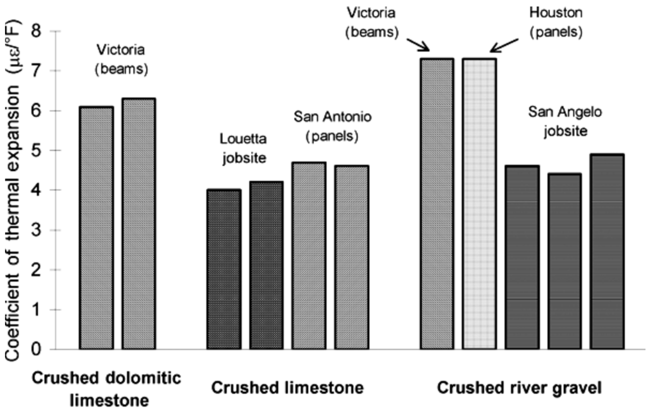


*Fig. 6.8—Coefficient of thermal expansion by aggregate type and source (adapted from Gross and Burns [1999]).*

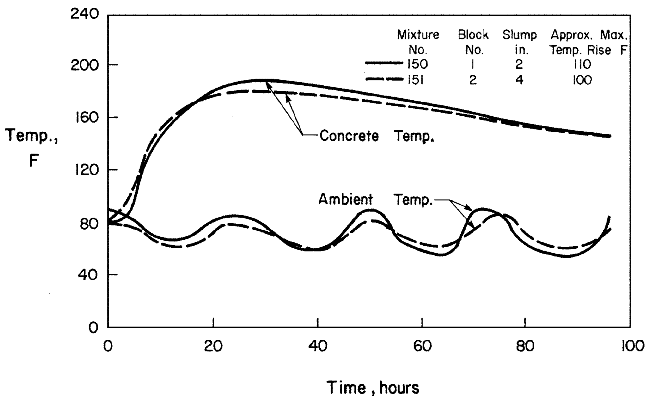


*Fig. 6.9—Temperature rise of high-strength field-cast 10 x 20 x 5 ft (3 x 6 x 1.5 m) blocks (adapted from Saucier et al. [1965]).*

## 6.10—Heat evolution due to hydration

Temperature rise within concrete due to hydration depends on the cement content, *w/cm*, member size, ambient temperature, and other environmental conditions. Freedman (1971) concluded from data of Saucier et al. (1965) in Fig. 6.9 that the temperature rise of HSCs will be approximately 11 to 15°F per 100 lb/yd$^3$ (10 to 14°C per 100 kg/m$^3$) of cement. Values for temperature rise on the order of 100°F (56°C) in HSC columns containing 846 lb/yd$^3$ (502 kg/m$^3$) of cement were measured in a building in Chicago, as shown in Fig. 6.10 (CCHRB 1977). This temperature rise can often be controlled or reduced by using SCMs as replacement materials instead of cement. The temperature for HSC girders with surface area-to-volume ratios from 0.4 to 0.15 was reported to range from 5 to 11°F per 100 lb/yd$^3$ (5 to 10°C per 59 kg/m$^3$) of cementitious material for mixtures with 30 to 32% fly ash replacement. These mixtures had temperature rises from 50 to 110°F (28 to 61°C) in HSC members with the aforementioned surface area-to-volume ratios containing 985 lb/yd$^3$ (581 kg/m$^3$) of cementitious materials (Myers and Carrasquillo 2000). The temperature rise will be affected by the shape and geometry of the structural element, as illustrated in the bridge girder shown in Fig. 6.11. Other methods to assist with temperature control, such as cooling material stock piles, using ice replacement, nitrogen cooling and early morning casting operations, or both, can be used as well.

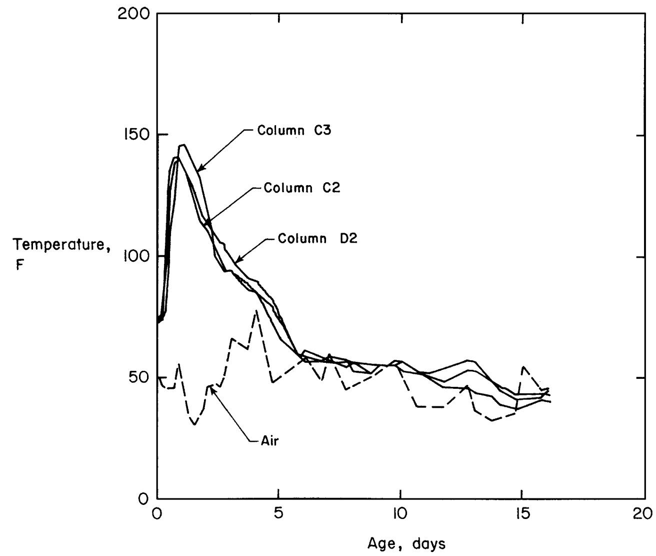


*Fig. 6.10—Measured concrete temperatures at Water Tower Place (adapted from CCHRB* Task Force Report *No. 5 [CCHRB 1977]).*

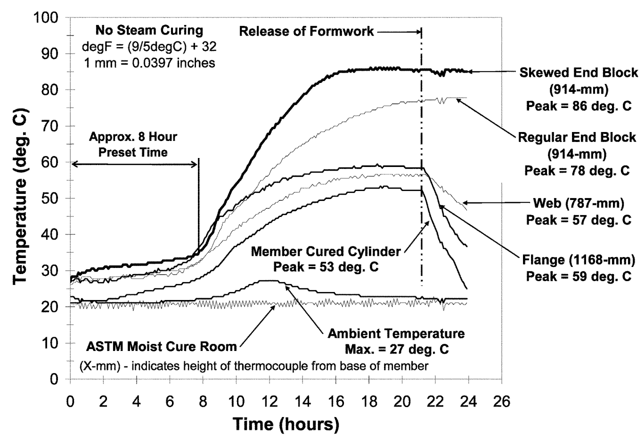


*Fig. 6.11—Measured concrete temperatures in precast prestressed concrete Texas U-beam (adapted from Myers and Carrasquillo [1998]).*

Hydration temperature can have a more pronounced influence on the mechanical properties of HSC compared with conventional concrete, particularly if the volume-to-surface area ratio of the member or structural component is large. Myers and Carrasquillo (2000) reported that hydration temperatures that exceeded 170°F (77°C) had negative effects on the mechanical and transport properties. This includes a reduced compressive strength and modulus of elasticity at early and later ages as well as an increased permeability at later ages. Higher hydration temperatures caused more extensive and wider cracking on the micro-structural level. The effect on compressive strength is illustrated in Fig. 6.12 and 6.13.

## 6.11—Strength gain with age

High-strength concrete shows a higher rate of strength gain at early ages compared with lower-strength concrete, but at later ages, the difference is not significant (Fig. 6.14) (Wischers 1978; Carrasquillo et al. 1981; Smith et al. 1964;

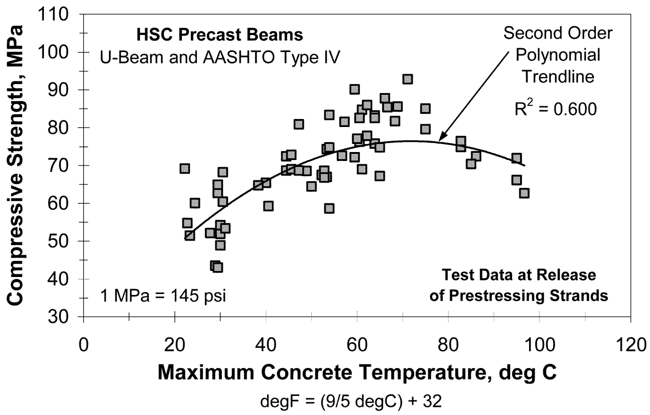


*Fig. 6.12—Effect of temperature rise on compressive strength of high-strength plant-cast girders at release (adapted from Myers and Carrasquillo [2000]).*

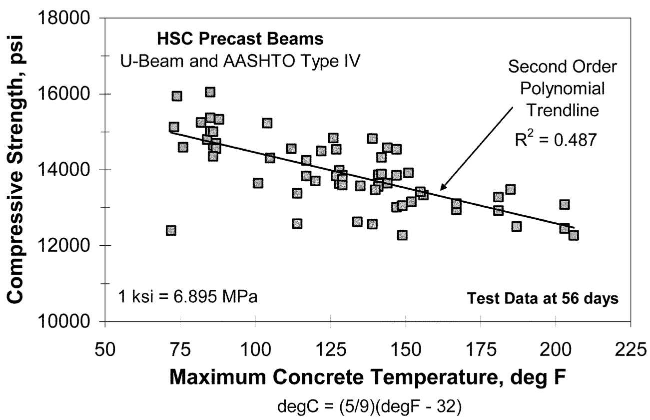


*Fig. 6.13—Effect of temperature rise on compressive strength of high-strength plant-cast girders at 56 days (adapted from Myers and Carrasquillo [2000]).*

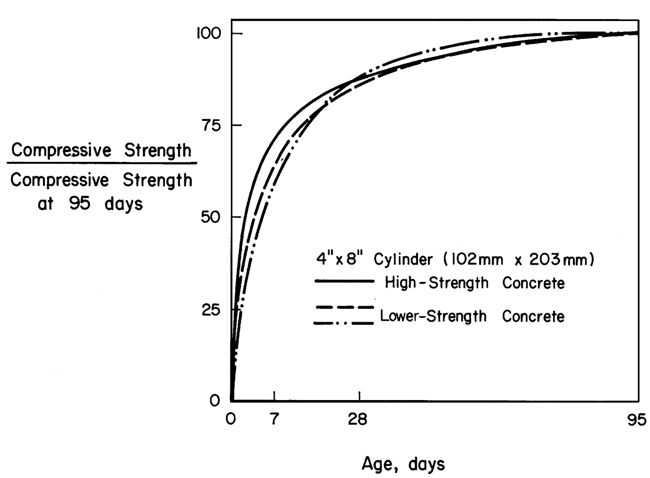


*Fig. 6.14—Normalized strength gain with age for moist-cured limestone concretes (adapted from Carrasquillo et al. [1982]).*

Freedman 1971). Parrott (1969) reported typical ratios of 7- to 28-day strengths of 0.8 to 0.9 for HSC and 0.7 to 0.75 for lower-strength concrete, whereas Carrasquillo et al. (1981) found typical ratios of 7- to 95-day strength of 0.60 for low-

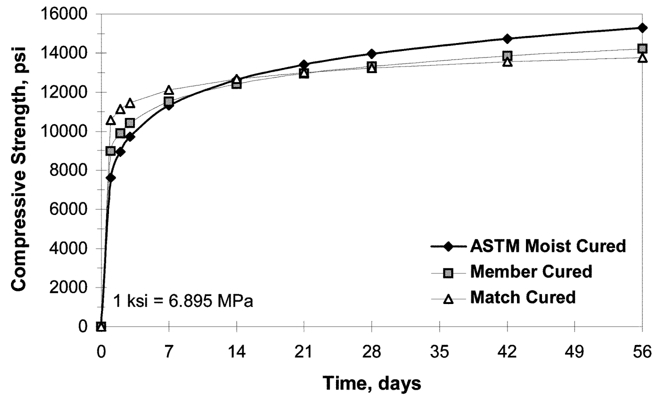


*Fig. 6.15—Compressive strength gain with age for precast girders under varied curing conditions (adapted from Myers and Carrasquillo [1998]).*

strength concrete, 0.65 for medium-strength concrete, and 0.73 for HSC. It seems likely that the higher rate of strength development of HSC 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 HSC due to a low *w/cm*.

The curing condition will also have an influence on the strength gain with time as with conventional concrete. This variation can be more pronounced in HSC when dealing with more massive structural shapes, as illustrated in Fig. 6.15. For this bridge girder, there is a difference of approximately 10% between moist-cured cylinders and match-cured cylinders at 56 days. When developing a QC/QA program, this may be an important consideration for HSC.

## 6.12—Resistance to freezing and thawing

Information about the air content requirement for HSC to produce adequate resistance to freezing and thawing is contradictory. For example, Saucier et al. (1965) concluded from accelerated laboratory freezing-and-thawing tests that if HSC is to be frozen under wet conditions, air-entrained concrete should be considered despite the loss of strength due to air entrainment. Other studies concur (Ernzen and Carrasquillo 1992; Mindess et al. 2003), but report lower than traditional air-entrainment levels are required for resistance to freezing and thawing of HSC. In contrast, Perenchio and Klieger (1978) obtained excellent resistance to freezing and thawing of all of the HSCs 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 HSC. Hale and Russell (2000) also concluded that air entrainment is not necessary to achieve adequate resistance to freezing and thawing with a *w/cm* less than 0.36. Several researchers (Cohen et al. 1992; Mokhtarzadeh et al. 1995; Fagerlund 1994) have developed durable non-air-entrained mixtures. There is consensus among experts, however, that members that are not subjected to becoming saturated above the critical saturation threshold of 91.7% do not warrant air entrainment for freezing-and-thawing protection. On the other hand, for members exposed to critical saturated conditions, there is no well-documented field experience to prove that air entrainment is not needed (Kosmatka et al. 2001). Practitioners may use ASTM test method C666 test to evaluate freezing-and-thawing resistance and C672 for scaling resistance of HSC.

## 6.13—Abrasion resistance

Abrasion is wearing due to repeated rubbing and friction. For pavements, abrasion results from traffic wear. Adequate abrasion resistance is important for pavements and bridge decks from the standpoint of safety. Excessive abrasion leads to an increase in accidents as the pavement becomes polished, reducing its skid resistance (Zia et al. 1993a,b).

Primary factors affecting abrasion include compressive strength, aggregate properties, surface finishing, curing, and the use of surface hardeners or toppings. Higher-strength concretes can be expected to have higher wear resistance than lower-strength mixtures with similar constituents, provided that they are finished and cured under similar conditions (Myers and Carrasquillo 1998). The abrasion resistance of aggregate is also important in determining the abrasion resistance of concrete (ACI 210R). This is particularly true when an exposed aggregate surface is used. Aggregates commonly used in the production of HSC are stiffer and typically more wear-resistant. Proper finishing and curing have significant beneficial effects on the abrasion resistance of concrete. Because many HSCs have a low *w*/*cm* with little bleed water, proper curing techniques are critical for good abrasion resistance of HSC. Fentress (1973) noted that when proper curing techniques are practiced in conjunction with a hardened finish, improved wear resistance results. Generally, the longer the duration of moist curing, the better the wear resistance. Proper surface finishing and curing techniques can only improve the abrasion resistance of HSC, just as with conventional concretes. Laboratory research by Hadchiti and Carrasquillo (1988) has shown that the incorporation of fly ash cement replacement does not affect the wear resistance of the concrete. Concrete strength is the governing factor affecting abrasion resistance rather than the material that makes up the cementitious fraction of the concrete, as illustrated in Fig. 6.16.

Almeida (1994) found that the abrasion resistance of HSC varied inversely with the *w*/*c*, cement paste volume, and porosity of concretes. He also reported that the use of a HRWRA improved the abrasion resistance for a given mixture by 25%. There are some minor differences, however, between the two types of concretes. Because bleed water is typically not a concern for low *w*/*cm* concretes, the timing of surface finishing and techniques used are less critical when compared with conventional concretes. In fact, some experts feel that less finishing for HSCs provides a reduced surface disruption and is actually better for the quality of the concrete. High-strength concrete has been used in dam stilling basins for its abrasion resistance and in the Confederation Bridge in Canada for resistance to ice abrasion (USACE 1995; FHWA 1996).

Experimental work on the abrasion resistance of highway concrete pavements subjected to heavy traffic from studded tires has been carried out. Increasing the concrete strength

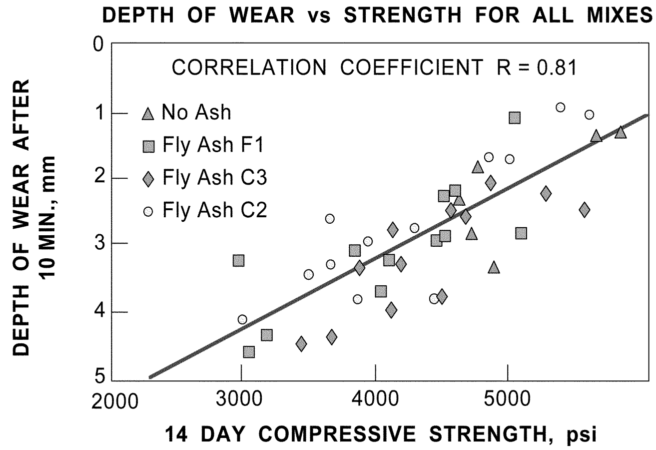


*Fig. 6.16—Depth of wear versus replacement type (adapted from Hadchiti and Carrasquillo [1988] (1 mm = 0.0394 in.; and 1 psi = 0.006895 MPa).*

from 7100 to 14,300 psi (50 to 100 MPa) reduced the abrasion by roughly 50%. At 21,400 psi (150 MPa), the abrasion of the concrete was comparable to that of high-quality massive granite. Compared with a standard Ab 16t asphalt highway pavement, this represents an increase in the service life of the pavement by a factor of approximately 10.

## 6.14—Shrinkage

All concrete undergoes non-load-induced volume change from initial placement through its service life. The magnitude and rate of this volume change is a complex phenomenon that is not completely understood, yet has an important influence on the resulting performance, especially durability, of concrete. For normal-strength concrete, volume change due to diffusion of internal water into the outer environment, commonly termed drying shrinkage, is the predominate mechanism. For HSCs that have a low *w*/*cm* and high binder content, other volume-change mechanisms influence the overall magnitude and rate of volume change. Most important among these are chemical shrinkage and autogenous shrinkage. Chemical shrinkage refers to the reduction in absolute volume of solids and liquids in paste resulting from cement hydration. The absolute volume of hydrated cement products is less than the absolute volume of cement and water before hydration (Kosmatka et al. 2001). Chemical shrinkage results in the development of internal voids in the paste structure, and does not translate into significant overall volume change in concrete. Autogenous shrinkage is that portion of chemical shrinkage that starts at initial set and results in overall volume external volume change in concrete. Chemical and autogenous shrinkage are more difficult to measure than drying shrinkage, and thus, there is comparatively less data on these phenomena. Sufficient data have, however, been developed (Tazawa 1999) to conclude that autogenous shrinkage can be significant for HSC, with values of $200 \times 10^{-6}$ to $400 \times 10^{-6}$ being reported for concrete with *w*/*cm* less than 0.40 and silica fume contents of not less than 10%.

Experimental data have generally shown no clear trend with respect to drying shrinkage of HSC, though it is often

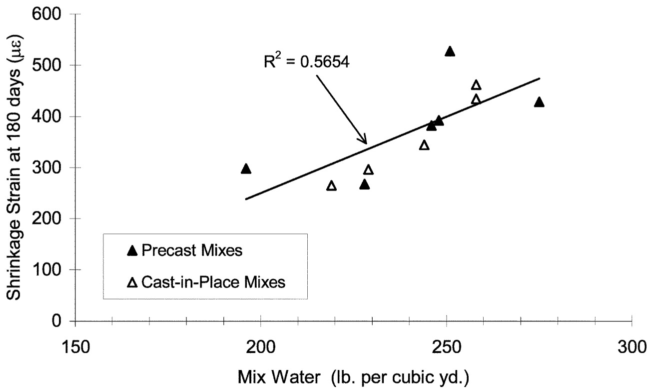


*Fig. 6.17—Drying shrinkage versus quantity of mixing water (adapted from Gross and Burns [1999]) (1 lb/yd$^3$ = 0.593 kg/m$^3$).*

suggested that the drying shrinkage of HSC is similar to the shrinkage of normal-strength concretes (Burg and Ost 1994). Ngab et al. (1981) noted slightly higher shrinkage for HSC when compared with normal-strength concrete made with similar materials. Smadi et al. (1985) also observed higher shrinkage for HSC (8500 to 10,000 psi [59 to 69 MPa]) as opposed to normal-strength concrete (5000 to 6000 psi [35 to 41 MPa]), but observed less shrinkage for HSC than for low-strength concrete (3000 to 3500 psi [21 to 24 MPa]). Swamy and Anand (1973) observed a high initial rate of shrinkage for HSC made with finely ground portland cement, but noted that shrinkage strains after 2 years were approximately equal to values suggested in CEB (1991). Freedman (1971) reported that shrinkage was unaffected by changes in the *w/cm*, but is approximately proportional to the percentage of water by volume in the concrete. This is consistent with long-term shrinkage tests by Gross and Burns (1999) that indicated shrinkage largely depended on the amount of mixture water (Fig. 6.17) and less than the “standard” values provided in ACI 209R. Other laboratory studies (Ngab et al. 1981) and field studies (CCHRB 1977; Pfeifer et al. 1971; Kaplan 1959a) have indicated that creep and drying shrinkage results were similar to results found for normal-strength concrete, whereas others (Mokhtarzadeh and French 2000a) have reported results similar to findings by Gross and Burns. Nagataki and Yonekura (1978) reported that the shrinkage of HSC containing HRWRAs was less than for lower-strength concrete.

Although there is no clear consensus among researchers with regard to the magnitude of drying shrinkage of HSC as compared with normal-strength concrete, there is general agreement that drying rates in HSC will be slower than in normal-strength concrete. Thus, it is likely that strains due to drying shrinkage only will develop slower in HSC.

From a practical viewpoint, the importance of volume change in concrete relates mainly to cracking potential. To the extent that volume change issues in HSC are not totally understood, there exists a comparable lack of understanding of cracking potential in HSC due to noninduced phenomena. Wiegrink et al. (1996) concluded that HSCs they tested had poorer shrinkage cracking performance than normal-strength concrete. Similar results were reported by Bloom and Bentur (1995) and Samman et al. (1996).

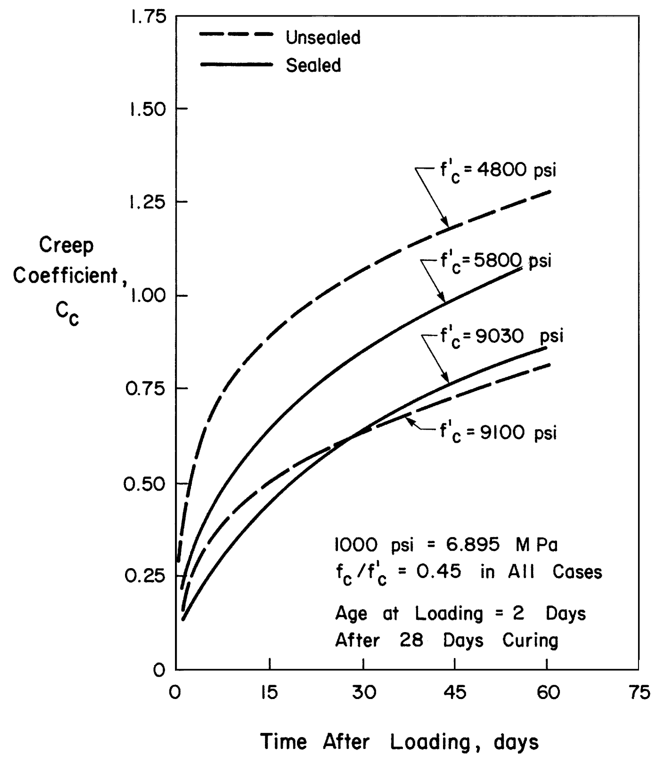


*Fig. 6.18—Relationship between creep coefficient and time for sealed and unsealed concrete specimens (adapted from Ngab et al. [1981]).*

## 6.15—Creep

Parrott (1969) reported that the total strain observed in sealed HSC under a sustained loading of 30% 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% lower than that of lower-strength concrete. The total long-term strains of drying and sealed HSC were 15 and 65% higher, respectively, than for a corresponding lower-strength concrete at a similar relative stress level. Ngab et al. (1981) found little difference between the creep of HSC under drying and sealed conditions. The creep of HSC made with HRWRAs is reported by Nagataki and Yonekura (1978) to be decreased significantly. Maximum specific creep was less for HSC than for lower-strength concrete loaded at the same age (Gross and Burns 1999; Ngab et al. 1981; Russell and Corley 1978; CCHRB 1977). An example is shown in Fig. 6.18 (Ngab et al. 1981).

High-strength concretes, however, are subjected to higher stresses. Therefore, the total creep will be about the same for any strength concrete. No problems due to creep were found in columns cast with HSC (Pfeifer et al. 1971). Gross and Burns (1999) reported that creep was largely dependent on the amount of mixing water and suggested that the lower creep values observed in instrumented HSC girders may result in less prestress losses compared with the values determined by prediction method for conventional concrete. As is found with lower-strength concrete, creep decreases as the age at loading increases (Ngab et al. 1981). Specific creep increases with an increased *w/cm* (Perenchio and Klieger

1978), and there is a linear relationship with the applied stress (Ngab et al. 1981). This linearity extends to a higher stress-strain ratio than for lower-strength concrete. Mokhtarzadeh and French (2000b) reported that higher curing temperatures result in higher specific creep. They also determined that the general ACI 209R equation was suitable for estimating the creep coefficient of HSC at any time *t*, whereas others reported contradictory results (Gross 1998).

## 6.16—Permeability

In 2003, Mindess et al. reported that the *w/cm* of concrete was the single parameter that had the largest influence on durability. As *w/cm* decreased, porosity of the paste decreased, resulting in less-permeable concrete. In recent years, SCMs have demonstrated their effectiveness in reducing permeability. The permeability of all concrete depends on the curing method and length of time cured (Whiting and Kulman 1987). Moist-curing not only significantly influences the strength development, but also impacts the permeability of the concrete. As the moist-curing period is increased, the strength development will increase, and the permeability will be lower (Neville 1981). The use of highly porous aggregates will increase the permeability of the concrete because substances can flow more easily through aggregate pores than through smaller pores of the cement paste (Neville 1981; Young 1988). Research studies have reported (Ernzen and Carrasquillo 1992; Myers et al. 1997) that for HSC, the addition of air entrainment does not drastically affect the permeability of the concrete. The use of HRWRAs has been shown to reduce the permeability of concrete by allowing the reduction in the *w/cm* and the uniform distribution of cement particles. The HRWRA helps disperse cement grains more uniformly within the paste, resulting in a pore structure with fewer coarse pores. This results in a reduced permeability. Other research (Zakka 1989), however, has shown that the use of a HRWRA can increase the permeability of the concrete when compared with a control mixture without a HRWRA during hot-weather concreting under certain conditions.

Pozzolans and slag cement have been shown to reduce the permeability of HSC; for example, Fig. 6.19 shows the effects of fly ash on the charge passed in the ASTM C1202 (or AASHTO T277) test. In the case of fly ash, several studies have demonstrated that the incorporation of fly ash in the concrete matrix will reduce its total porosity and result in a finer pore structure compared with the matrix without any fly ash (Feldman 1981; Manmohan and Metha 1981). Other studies have suggested that the pozzolanic reaction of the fly ash has the tendency to break the interconnected pore system (Marsh et al. 1985). Some experts (Haque et al. 1992) contend that the fly ash binds a significant amount of the free chloride ions, which can ingress into concrete from the surface. The binding of these chloride ions reduces the amount of free chloride ions that are available to reach the level of the reinforcing steel to initiate corrosion. Other studies (Tikalsky and Carrasquillo 1989) have shown that the use of fly ash increases the concrete permeability at early ages, but improves it at later ages.

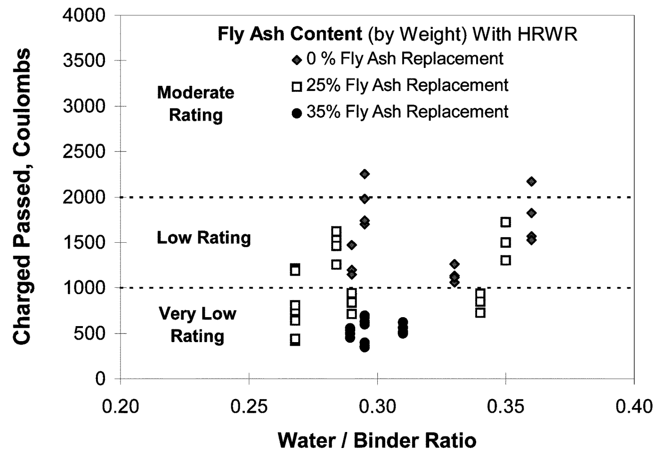


*Fig. 6.19—Charged passed at 56 days versus* w/cm *for mixtures with and without fly ash (adapted from Myers [2001]).*

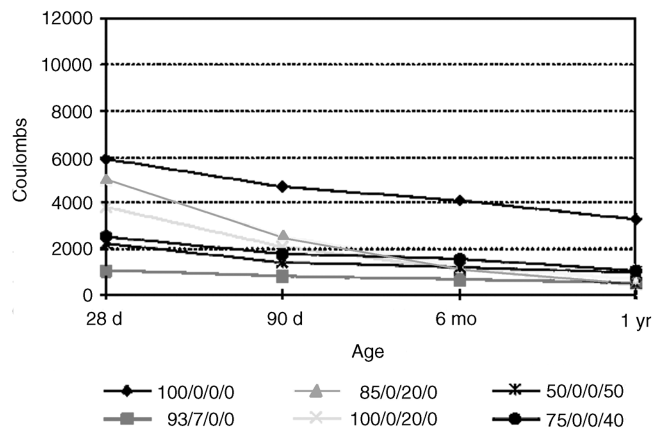


*Fig. 6.20—Reduction in permeability of concrete cured at 73°F (23°C) (adapted from Ozyildirim [1998]).*

Other pozzolans, slag cement, and combinations of these materials have been shown to reduce the permeability of HSC effectively, as shown in Fig. 6.20. Ozyildirim (1998) demonstrated the effectiveness of using silica fume, fly ash, and slag.

The reduction in the size of capillary pores increases the probability of transforming continuous pores into discontinuous ones (Philleo 1986). Because capillary porosity is related to permeability (Powers et al. 1954), the permeability to liquids and vapors is thus reduced by silica fume additions. Hooton's (1986) data for cement pastes with a 0.25 *w/cm* indicated water permeability of $0.9 \times 10^{-13}$ m/s and $< 0.1 \times 10^{-13}$ m/s for 28-day cured pastes containing 10 and 20% by volume of silica fume, respectively. When no silica fume was added, permeability was higher: $3.8^{-13}$ m/s. ACI 234R concludes that the contribution of silica fume to the reduction in water permeability is very large, with the reduction in permeability coefficient being up to an order of magnitude or more depending on the mixture composition and the dosage of silica fume.

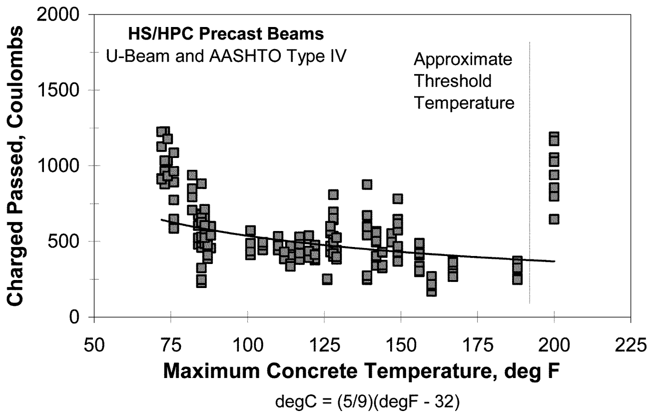


*Fig. 6.21—Charged passed at 56 days versus maximum concrete temperature during hydration (adapted from Myers and Carrasquillo [2000]).*

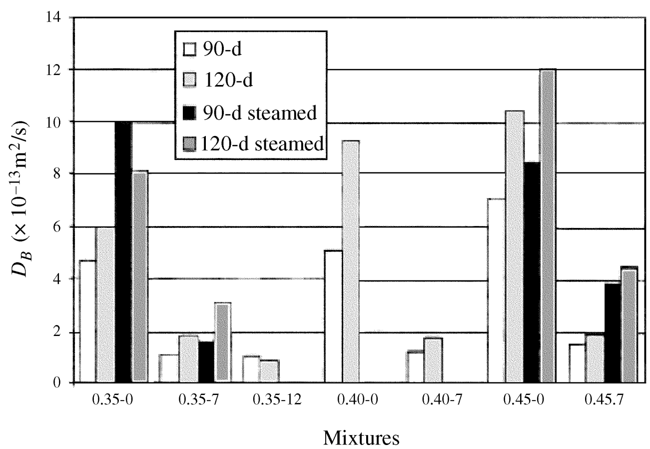


*Fig. 6.22—Bulk diffusion coefficient* $D_B$ *of all mixtures (adapted from Hooton [1986]). (Note: 1* $m^2/s$ *= 10.9* $ft^2/s$*.)*

Curing temperature has also been reported to influence the pore size distribution of a cement paste. According to Young (1988), at higher curing temperatures, there is an increase in the volume of large pores, which increases the permeability of the cement paste. Myers and Carrasquillo (2000) reported that peak temperature development during hydration has a dramatic influence on the permeability of HSC based on the rapid chloride permeability test ASTM C1202 (or AASHTO T277) as illustrated in Fig. 6.21. As peak hydration temperature increases, the permeability is reduced until a threshold temperature is reached. At this point, increased cracking on the microstructure level occurs due to extreme temperatures and results in increased permeability.

Hooton et al. (1997) demonstrated the reduction in permeability that results when concrete is steam-cured with silica fume, as illustrated in Fig. 6.22.

## 6.17—Scaling resistance

Surface scaling is caused by repeated application of deicing salts in combination with freezing-and-thawing cycles. Concrete surface damaged by salt scaling becomes roughened and pitted as a result of spalling and flaking of small pieces of mortar near the surface. Even high-quality concrete with adequate air entrainment can suffer scaling by deicing chemicals. The exact cause of scaling is not known, but it is recognized that when deicing chemicals are applied to melt ice, the heat consumption causes a rapid drop in the temperature of the concrete just below the surface, resulting in damage from the effects of rapid freezing or differential thermal strains. Furthermore, deicing chemicals can accumulate in the surface layer of the concrete, forming relatively concentrated solutions. When water stays on the concrete surface, it flows toward the concentrated chemical solution, causing an osmotic action accompanied by hydraulic pressures. These pressures may, in turn, cause salt scaling.

The best prevention of scaling is to eliminate the weak layer of material by proper mixture proportioning and good construction practice in placing, finishing, and curing. Over-vibration, too much toweling, and excessive bleeding should all be avoided. Well-cured concrete pavements, allowed to dry for a period before deicing salts are applied, will generally have good scaling resistance (Zia et al. 1993a,b).

Cement can also affect the scaling resistance. It has been found that using finer cements can improve the scaling resistance (Fagerlund 1975; Marchand et al. 1994). Gagne et al. (1991) studied 17 low *w/cm* concretes (0.26 to 0.30) with various cements and silica fume, without air entrainment, with compressive strengths in the range 8600 to 12,860 psi (60 to 90 MPa). They reported minimal and no scaling damage. Li et al. (1994) also found no salt scaling of non-air-entrained concretes at 50 cycles for *w/cm* values of 0.24 to 0.33. At 100 cycles, however, concrete with *w/cm* = 0.30 had scaling damage, and concrete with *w/cm* = 0.33 had severe damage. Pinto and Hover (2001) reported that no air entrainment was necessary for concrete mixtures with a *w/c* of 0.25 to achieve scaling resistance. Air entrainment was necessary for mixtures with *w/c* greater than 0.25. High-reactivity metakaolin has also been used as an effective SCM for HSC, which proved to have satisfactory performance in scaling resistance (Caldarone et al. 1994).

## 6.18—Fire resistance

It is well established that mechanical properties of concrete are adversely affected by thermal exposure (ACI 216.1). Normal-strength concrete loses between 10 to 20% of its original compressive strength when heated to 572°F (300°C) and between 60 to 75% at 1112°F (600 °C). The effect on modulus of elasticity of normal-strength concrete is reported to be similar. Phan and Carino (2000) studied the influence of high temperature on HSC. Their study reported that HSC has a higher strength loss than normal-strength concrete in the temperature range between 77 and 752°F (25 and 400°C). Above 1112°F (600°C), the differences between normal-and high-strength concrete are less pronounced. Higher occurrences of explosive spalling of specimens were also observed above 572°F (300°C) by Phan and Carino (2000). Kodur (2000) also studied the spalling characteristics of HSC subjected to fire and reported that spalling is not only influenced by concrete strength, but also concrete density, aggregate type, load intensity, reinforcement configuration, and layout. Kodur reported that the addition of synthetic fibers could improve HSC's resistance to spalling, although it is unclear how synthetic fibers would affect the residual mechanical properties of the concrete after exposure to fire.

## CHAPTER 7—STRUCTURAL DESIGN CONSIDERATIONS

### 7.1—Introduction

High-strength concretes have some characteristics and engineering properties that are different from those of lower-strength concretes. These distinctions are increasingly important as strengths increase, and should be recognized by design engineers in predicting the performance and safety of structures. Tests of unreinforced HSC have shown, for example, that such material in many cases may be characterized as linearly elastic up to stress levels approaching the maximum stress. Thereafter, the descending branch of the stress-strain curve falls more steeply than for lower-strength concretes, as illustrated in Fig. 7.1 (Carrasquillo et al. 1981, 1982; Kaar et al. 1978; Perenchio and Klieger 1978; Wang et al. 1978a).

Extensive research has provided a solid understanding of the behavior of HSC. In this chapter, emphasis is placed on design of structural members. Where recommendations are provided, they are based on the best current experimental information. With additional research, these recommendations may be subject to further review and revision.

This chapter focuses on the design of structural members with design compressive strengths in excess of 8000 psi (55 MPa). It is important to recognize, however, that there is no sudden change in structural behavior at this value of compressive strength. Instead, structural behavior changes gradually as concrete compressive strengths increase. In all cases, the same basic principles of mechanics apply for members constructed with any concrete strength.

The use of higher-strength concretes permits efficient structural designs, allowing members to span longer distances, be smaller in cross section, and carry larger loads. These designs are likely to be controlled by serviceability and other practical design considerations instead of strength. As a result, special considerations may be required in the design of HSC structural members.

This chapter discusses only normal-density HSC, made with typical cementitious and pozzolanic materials and admixtures. Structural design of low-density (lightweight) HSC and fiber-reinforced HSC is not discussed. Similarly, only steel reinforcement is considered, although research is currently being performed on the use of other types of reinforcement, such as fiber-reinforced polymers, in conjunction with HSC.

### 7.2—Concentrically loaded columns

Because the strength of columns is generally controlled by the compressive strength of concrete, there are significant advantages to using HSC in columns, especially those that carry axial loads alone. Significant research has been completed that focuses on various aspects of structural behavior for HSC columns. Much of this research has been summarized by Joint ACI-ASCE Committee 441 (ACI 441R), which reports on the state of the art for HSC columns. For brevity, only an overview of concentrically loaded HSC columns is provided in this report. The reader is referred to the ACI 441R for more thorough treatment of the subject.

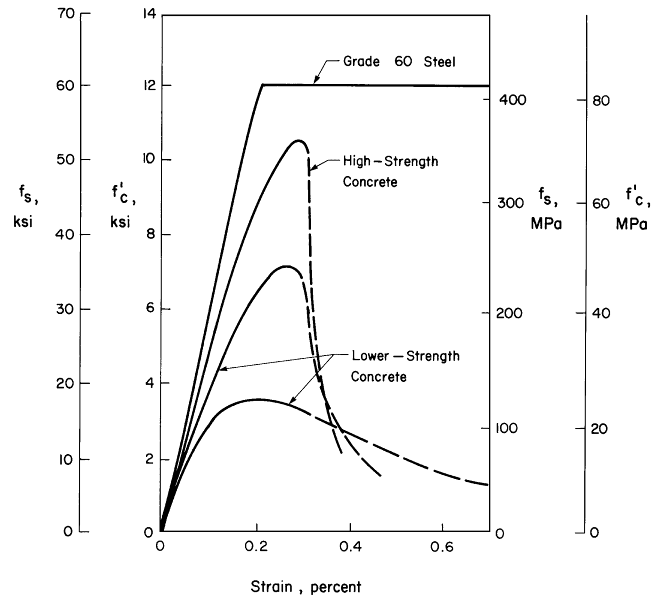


*Fig. 7.1—Concrete and steel stress-strain curves.*

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 design requirements reflect this by effectively requiring a minimum eccentricity in the computation of design capacity. It is useful, however, to first examine the behavior of columns carrying axial load only. Eccentrically loaded columns are discussed in Section 7.5.

**7.2.1** *Axial strength*—Present design practice, in calculating the nominal strength of an axially loaded member, is to assume a direct addition law summing the strengths of the concrete and steel. The justification for this is seen in Fig. 7.1, which superimposes typical stress-strain curves in compression for three concretes with that for reinforcing steel having a 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 stress-strain reaches the range of significant nonlinearity (approximately 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. The steel reaches its yield point at about the same strain in this case; thus, concrete is at its maximum stress, steel is at $f_y$, and the strength of the column is predicted by

$$P = 0.85 f'_c (A_g - A_{st}) + f_y A_{st} \qquad (7\text{-}1)$$

where $f'_c$ is the specified compressive strength of the concrete; $f_y$ is the yield strength of steel; $A_g$ is the gross area of section; and $A_{st}$ = total area of longitudinal steel.

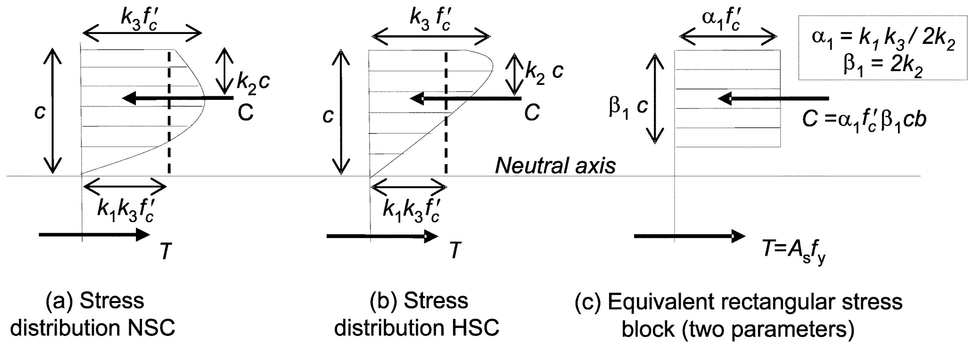


*Fig. 7.2—Compressive stress distributions.*

The factor 0.85 is used to account for the difference in the in-place strength of concrete in columns compared with the strength of the same concrete mixture obtained from standard compression tests on cylinders. The factor was derived from a series of experimental tests on lower-strength concrete columns in the 1930s (Richart and Brown 1934), and is related to the effects of vertical casting on compressive strength, the slower loading rate for columns as compared with cylinder tests, and the effects of sustained loads.

A similar analysis holds for HSC columns, except the steel will yield before the concrete reaches its peak strength. The steel, however, will continue to yield at essentially constant stress until the concrete is fully stressed. Prediction of column strength may therefore still be based on a summation of the concrete and steel contributions.

It should be noted that experimental research has also shown that HSC columns are more susceptible to premature spalling of the concrete cover. The early spalling of cover concrete in HSC columns is attributed to differences in the rate of drying shrinkage between the cover concrete and the inner core (Collins et al. 1993). This differential shrinkage causes tensile stresses to be developed in the cover concrete. These tensile stresses, in combination with the presence of a plane of weakness caused by the reinforcement, lead to a cracking pattern that may result in spalling of the concrete cover. Early spalling is accentuated by the presence of closely spaced longitudinal or transverse column reinforcement (Cusson and Paultre 1994; Razvi and Saatcioglu 1994).

Ibrahim and MacGregor (1997) compiled results of 90 tests on concentrically loaded columns reported in the literature (Cusson and Paultre 1994; Martinez et al. 1984; Richart and Brown 1934; Sheikh and Uzumeri 1980; Yong et al. 1988), with concrete strengths of up to 17,000 psi (117 MPa) and with a wide range of lateral reinforcement ratios. They concluded that Eq. (7-1) was not conservative for all tests. Attard and Stewart (1998) analyzed the same set of 90 column tests and concluded that the 0.85 in Eq. (7-1) could be replaced by an expression that is a function of the concrete compressive strength

$$0.92 - (f'_c/160{,}000) \quad (f'_c \text{ in psi})$$
$$0.92 - 0.0009 f'_c \quad (f'_c \text{ in MPa}) \qquad \text{(7-2)}$$

A second approach convenient for design uses the stress block parameter $\alpha_1$, which is discussed in Section 7.3.1. In stress block computations, $\alpha_1$ reflects the percentage of $f'_c$ assumed to act uniformly over the portion of the cross section in compression (Fig. 7.2). Though the parameter $\alpha_1$ is not explicitly intended for use in calculating the pure axial capacity of a cross section, it can be easily and rationally substituted in place of the 0.85 coefficient in Eq. (7-1). There is general agreement that the parameter $\alpha_1$ decreases as $f'_c$ increases (though ACI 318-05 considers a uniform value of $\alpha_1$ equal to 0.85 for all concrete strengths). Different proposals for $\alpha_1$ are summarized in Fig. 7.3. Equations for $\alpha_1$ corresponding to each of these proposals may be found by consulting each of these references individually (Attard and Stewart 1998; Azizinamini et al. 1994; CSA A23.3 1994; Ibrahim and MacGregor 1997; NZS 3101).

A third, conservative option for the calculation of axial capacity of HSC is to ignore the concrete cover and use only the confined core area in calculating the capacity using Eq. (7-1) (Razvi and Saatcioglu 1994).

**7.2.2** *Effects of confinement reinforcement*—Lateral reinforcement in columns, particularly in the form of continuous spirals, has two beneficial effects on column behavior: 1) it greatly increases the strength of the core concrete inside the spiral by confining the core against lateral expansion under load; and 2) it increases the axial strain capacity of the concrete, permitting a more gradual and ductile failure, that is, a tougher column (Ahmad and Shah 1982a,b; Fafitis and Shah 1985; Martinez et al. 1984; Yong et al. 1988). The basis for design of spiral steel is that the strengthening effect of the spiral should be at least equal to the column strength lost when the concrete shell outside of the spiral spalls off under load. The ACI 318-05 equation for minimum volumetric ratio of spiral is

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y} \qquad \text{(7-3)}$$

where $\rho_s$ is the ratio of volume of spiral reinforcement to volume of concrete core; $A_g$ is the gross area of concrete section; $A_c$ is the area of concrete core; $f'_c$ is the specified compressive strength of concrete; and $f_y$ is the yield strength of spiral steel.

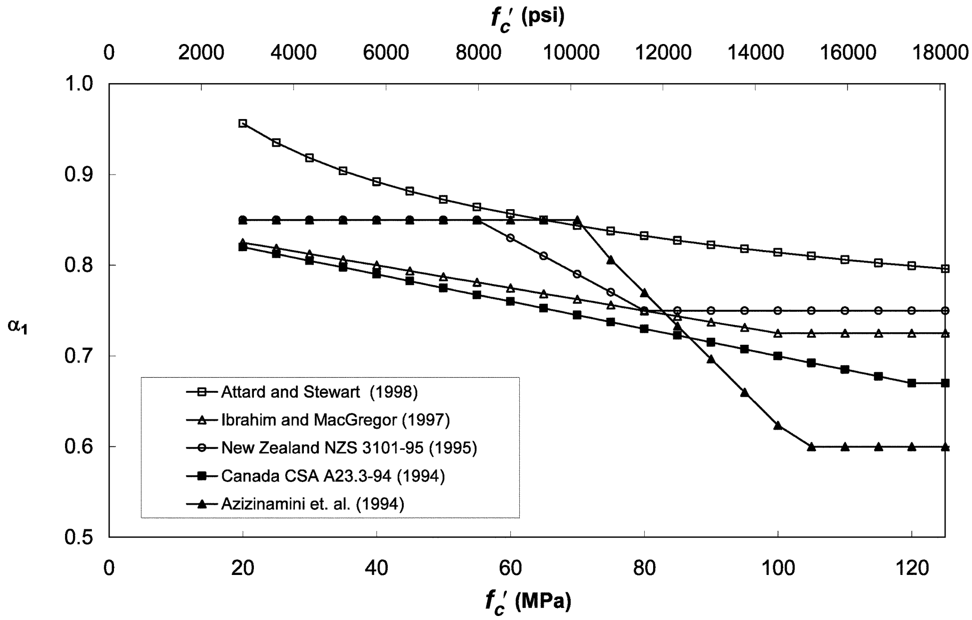


*Fig. 7.3—Proposed values of $\alpha_1$ as a function of concrete compressive strength.*

The increase in compressive strength of columns provided by spiral steel is based on an experimentally derived relationship for strength gain (ACI Committee 105 1933)

$$\bar{f}_c - f_c'' = 4.0 f_2' \tag{7-4}$$

where $\bar{f}_c$ is the compressive strength of spirally reinforced concrete column; $f_c''$ is the compressive strength of unconfined concrete column; and $f_2'$ is the concrete confinement stress produced by spiral.

This relationship can be shown to lead directly to Eq. (7-3). The concrete confinement stress produced by spirals $f_2'$ is calculated on the assumption that the spiral steel has yielded, using the hoop tension equation

$$2A_{tr}f_y = f_2' d_c s \tag{7-5}$$

or

$$f_2' = \frac{2A}{d_c s} \tag{7-6}$$

where $A_{tr}$ is the area of spiral steel; $d_c$ is the diameter of concrete core; and $s$ is the pitch of spiral.

Experimental research has shown that spiral reinforcement is less effective for columns of HSC than for columns of normal-strength concrete (Abdel-Fattah and Ahmad 1989; Ahmad and Shah 1982a; Bjerkeli et al. 1990; Cusson and Paultre 1994; Fafitis and Shah 1985; Martinez et al. 1984; Rangan et al. 1991; Razvi and Saatcioglu 1994; Yong et al. 1988). This reduced effectiveness can be attributed to less lateral expansion of the concrete core, which leads to lower stresses in the spirals at the peak load in HSC columns. In some tests, the stress in the steel spiral at peak load for HSC columns has been found to be significantly less than the yield strength assumed in Eq. (7-3) (Ahmad and Shah 1982a; Martinez et al. 1984).

The reduced effectiveness of spirals is particularly evident for small volumetric spiral ratios. Razvi and Saatcioglu (1994) compiled data from several researchers and demonstrated that there is a relationship between the nondimensional parameter $\rho_s f_y / f_c'$ and the adequacy of Eq. (7-1) in calculating the axial capacity of HSC columns. In particular, a large percentage of columns with small volumetric spiral ratios, or with parameters $\rho_s f_y / f_c'$ less than about 0.20, failed at loads lower than predicted using Eq. (7-1), whereas columns with large volumetric ratios failed at loads greater than those predicted using Eq. (7-1). Those columns with low volumetric ratios tended to exhibit spalling of the concrete cover before the confinement engaging the core concrete. Research has also shown that the parameter $\rho_s f_y / f_c'$ can be related to the ductility of HSC columns increasing the strain limit and flattening the negative slope of the stress-strain curve past the point of peak stress (Razvi and Saatcioglu 1994).

An important observation relating to spirally reinforced columns is that the level of confinement stress corresponding to spirals designed by ACI 318-05 is generally rather low for all columns. Confinement stress becomes significantly lower for larger-diameter columns, assuming that the cover requirements remain constant. This follows directly from Eq. (7-3). For larger columns, the ratio $A_g/A_c$ decreases; consequently, the required spiral steel ratio becomes smaller, and the effective confinement stress becomes proportionately smaller. Spiral requirements per ACI 318 for low- and high-strength concrete with 15 and 50 in. (38 and 172 mm) column core diameters are compared in Table 7.1.

Tests show that for lower-strength concrete, even the reduction in confinement stress for larger-diameter columns will produce a column with very large strain capacity without significant loss of resistance. For HSC, the reduction

**Table 7.1—Comparison of spiral requirements per ACI 318-05**

| $d_c$, in. (mm) | $A_g/A_c$ | $\rho_s$ | | $\rho_s f_y/f_c'$ | $s$, in. (mm) |
|---|---|---|---|---|---|
| $f_c'$ = 3000 psi (21 MPa) (No. 3 spiral bar) | | | | | |
| 15 (38) | 1.44 | 0.0099 | | 0.198 | 2.96 (75) |
| 50 (172) | 1.12 | 0.0028 | | 0.056 | 3.17 (81) |
| $f_c'$ = 10,000 psi (69 MPa) (No. 5 spiral bar) | | | | | |
| 15 (38) | 1.44 | | 0.0330 | 0.198 | 2.50 (64) |
| 50 (172) | 1.12 | | 0.0093 | 0.056 | 2.67 (68) |

of confinement stress produces a column with virtually no post-peak strain capacity. Even the higher confinement stress associated with the smaller-diameter HSC column produces a column with the undesirable characteristic of a sharp drop off of resistance immediately after peak stress (Martinez et al. 1984).

Tied columns result in lower strength and ductility, for both normal- and high-strength concrete, in comparison to spiral columns. Experimental data suggest similar trends for tied HSC columns as for HSC columns with spirals. HSC columns with large levels of lateral confinement in the form of ties have been shown to exhibit improved strength and ductility over columns with minimal ties (Cusson and Paultre 1994; Sheikh and Uzumeri 1980; Vallenas et al. 1977).

**7.2.3** *Cyclic and dynamic loading*—High-strength concrete is relatively free of internal microcracking, even at load levels close to ultimate, when loaded monotonically (Carrasquillo et al. 1981). High-strength concrete, however, is reported to be more brittle than lower-strength concrete (Carrasquillo et al. 1981), lacking much of the ductility that accompanies progressive crack growth. Experimental research indicates that fatigue strength is essentially independent of compressive strength (Bennett and Muir 1967). Research indicates that failure of concrete subject to repeated loading can be approximately predicted by the concept of the envelope curve, which is directly related to the short-term monotonic stress-strain curve (Ahmad 1981). For HSC, each load application causes relatively less incremental damage. The number of cycles to failure, however, may not necessarily be larger because of the greater negative slope of the post-peak envelope curve.

Bing et al. (2000) conducted an experimental investigation into the effect of strain rate on concentrically loaded high-strength reinforced concrete columns. Thirty columns were tested with different concrete compressive strengths, confining reinforcement configurations, and strain (loading) rates. They observed that the strength of the columns increased with increasing strain rates. The compressive strength, however, was found to be less sensitive to strain rate for HSC than for normal-strength concrete.

**7.2.4** *Sustained loading*—In most structures, concrete is subjected to sustained loads. Time-dependent strains associated with these stresses have a profound effect on structural behavior. 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 overstresses through creep.

Creep may be described either in terms of the creep coefficient

$$C_c = \frac{\varepsilon_{creep}}{\varepsilon_{initial}} \tag{7-7}$$

where $C_c$ is the creep coefficient; $\varepsilon_{initial}$ is initial strain upon application of load; and $\varepsilon_{creep}$ is additional time-dependent strain due to creep.

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

$$\delta_c = \frac{\varepsilon_{creep}}{\sigma_{initial}} \tag{7-8}$$

where $\delta_c$ is specific creep (unit creep coefficient); and $\sigma_{initial}$ is initial stress due to sustained load.

The two can be related through the modulus of elasticity

$$C_c = E_c\delta_c \tag{7-9}$$

Though specific numerical relationships cannot be stated for all HSCs, there is general agreement that creep of HSC is significantly less than that of lower-strength concrete (Burns et al. 1997; Gross 1998; Huo and Tadros 2000; Ngab et al. 1980, 1981; Russell and Corley 1978; Smadi et al. 1982, 1987). As a result, for axially loaded HSC columns, creep shortening for a given stress will be less than that of lower-strength concrete columns, a fact of possible significance in high-rise concrete structures (Russell and Corley 1978). In addition, the distribution of load between concrete and steel of HSC columns will be less subject to change with the passage of time. Therefore, the elastic distribution of HSC stresses varies less with time.

For normal-strength concrete, the ratio of the long-term to short-term strength has been established as 70 to 75% (Rüsch 1960). Experimental tests have shown that this reduction in strength for sustained loads is less pronounced for HSC than for normal-strength concrete. The ratio of long-term to short-term strength ranges from about 70 to 95% for HSC, and the ratio increases as the short-term compressive strength increases (Iravani and MacGregor 1998; Ngab et al. 1981; Smadi et al. 1985). The long-term to short-term strength ratio has been observed to be higher for HSC using silica fume than for HSC without silica fume (Iravani and MacGregor 1998).

## 7.3—Beams and one-way slabs

High-strength concrete beams behave according to the same principles of mechanics that have been used to describe behavior of beams made of lower-strength concrete. The material properties described in Chapter 6, however, can have a significant effect on certain aspects of structural performance for HSC beams (Leslie et al. 1976; Nedderman 1973; Pastor et al. 1984; Zia 1977, 1983).

**7.3.1** *Flexural strength*—In present practice, proportioning of beam sections is generally based on conditions at ultimate, a hypothetical state of incipient collapse at factored loads. At ultimate, the compressive stress distribution in beams is directly related to the shape of the stress-strain curve in uniaxial compression. Figure 7.2(a) shows the generally parabolic shape of the compressive stress distribution in a beam made of lower-strength concrete. For HSC, the stress-strain curve is more linear than parabolic, resulting in the compressive stress distribution shown in Fig. 7.2(b).

The nominal resisting moment for a cross section may be calculated knowing the internal forces $T$ and $C$ and the internal lever arm between them. Considering basic mechanics principles, the nominal flexural strength of under-reinforced beams is controlled primarily by the internal tension force, which depends in turn on the quantity and yield strength of tensile reinforcement. The flexural strength of a tension-controlled under-reinforced beam is therefore not strongly affected by the magnitude of the compressive strength used in design. For beams with typical levels of reinforcement, doubling the compressive strength of concrete used in the beam will generally only result in an increase of flexural strength of about 10%. This makes the use of HSC somewhat inefficient for under-reinforced (tension-controlled) beams. For over-reinforced (compression-controlled) sections, which are not permitted by ACI 318 for flexural members, the flexural strength will be much more impacted by the compressive strength of concrete. The flexural strength of over-reinforced concrete sections can be enhanced greatly through the use of HSC.

The actual shape of the compressive stress distribution at ultimate may be considered irrelevant if one knows: a) the magnitude of the compressive resultant $C$; and b) the depth in the beam at which it acts. These may be established in terms of three parameters characteristic of a given stress distribution (Fig. 7.2):

$k_1$ = ratio of average to maximum compressive stress in beam;

$k_2$ = ratio of depth to compressive resultant to neutral axis depth; and

$k_3$ = ratio of maximum stress in beam to maximum stress in corresponding axially loaded cylinder.

For ordinary design purposes, it is convenient to work with an equivalent rectangular compressive stress distribution, shown in Fig. 7.2(c), with a magnitude of the compressive resultant and line of action equivalent to those of the actual compressive stress distribution. Such an equivalent distribution is specifically referenced and permitted in ACI 318-05. Two parameters are required to define the equivalent distribution, $\alpha_1$ and $\beta_1$. The relationship between these two parameters and the three parameters for the generic stress distribution, $k_1$, $k_2$, and $k_3$, can be seen in Fig. 7.2. Using the equivalent rectangular stress block, the nominal flexural strength of a singly reinforced beam that is under-reinforced can be calculated by

$$M_n = bd^2 f_c' \omega\left(1 - \frac{\omega}{2\alpha_1}\right) \tag{7-10}$$

where $M_n$ is nominal moment strength at section, in.-lb (N·mm); $b$ is the width of the cross section, in. (mm); $d$ is the distance from extreme compression fiber to centroid of tension reinforcement, in. (mm); $f_c'$ is the specified compressive strength of concrete, psi (MPa); $f_y$ is the yield strength of the tension reinforcement, psi (MPa); $\alpha_1$ is the stress block parameter as defined in Fig. 7.2; $A_s$ is the area of tension reinforcement, in.$^2$ (mm$^2$); and

$$\omega = \rho\frac{f_y}{f_c'} = \left(\frac{A_s}{bd}\right)\frac{f_y}{f_c'}$$

The term $1/2\alpha_1$ in Eq. (7-10) can be shown to be equivalent to $k_2/k_1k_3$. For any stress distribution, regardless of the shape, a unique value of $k_2/k_1k_3$ exists, as does a companion value of $\alpha_1$. ACI 318-05 assumes a uniform value of concrete compression equal to $0.85f_c'$, thus setting the parameter $\alpha_1$ equal to 0.85 for all cases. The $\alpha_1$ term does not specifically appear in ACI 318; however, work undertaken by Tadros et al. (2003) and Ibrahim and Macgregor (1997) may be referenced for greater discussion and detail concerning the $\alpha_1$ term. ACI 318-05 specifies values of $\beta_1$ equal to 0.85 for concretes with a compressive strength of 4000 psi (28 MPa) or lower and 0.65 for concretes with a compressive strength of 8000 psi (55 MPa) or higher, with a linear relationship between $\beta_1$ and $f_c'$ for intermediate values. For the parameters specified in ACI 318-05, Eq. (7-10) thus reduces to

$$M_n = bd^2 f_c' \omega(1 - 0.59\omega) \tag{7-11}$$

For HSC, the stress-strain curve differs somewhat from that for normal-strength concrete, and is more linear than parabolic. Therefore, different stress block shapes have been proposed for use in calculating flexural capacity for HSC. Proposals have included a triangular stress block (Leslie et al. 1976) and a trapezoidal (bilinear) stress block (Pastor et al. 1984; Zia 1983). CSA A23.3 permits the use of equations that model the actual stress-strain curve of the concrete, with a peak value taken as $0.90f_c'$.

Experimental research on eccentrically loaded HSC columns has also resulted in several proposed modifications to the ACI 318 values for $\alpha_1$ and $\beta_1$ in the equivalent rectangular stress block. In these proposals, the factors are generally represented as functions of the compressive strength $f_c'$. Proposals for the factor $\alpha_1$ are summarized in Fig. 7.3, and proposals for the factor $\beta_1$ are summarized in Fig. 7.4. The ACI 318 function for $\beta_1$ is plotted for comparison. Equations corresponding to each of these proposals may be found by consulting each of these references individually (Attard and Stewart 1998; Azizinamini et al. 1994; CSA A23.3; Ibrahim and MacGregor 1997; NZS 3101). Figure 7.3 shows that the proposed values of $\alpha_1$ range from approximately 0.60 to 0.95.

For a triangular distribution with a peak stress of $f_c'$ and a compressive stress block extending to the neutral axis, values of $k_1$, $k_2$, and $k_3$ are equal to 1/2, 1/3, and 1, respectively. As such, $k_2/k_1k_3$ is equal to 0.667 for the triangular distribution, and considering that $\alpha_1$ is equivalent to $k_1k_3/2k_2$ (as shown in Fig. 7.3), $\alpha_1$ is equivalent to 0.75.

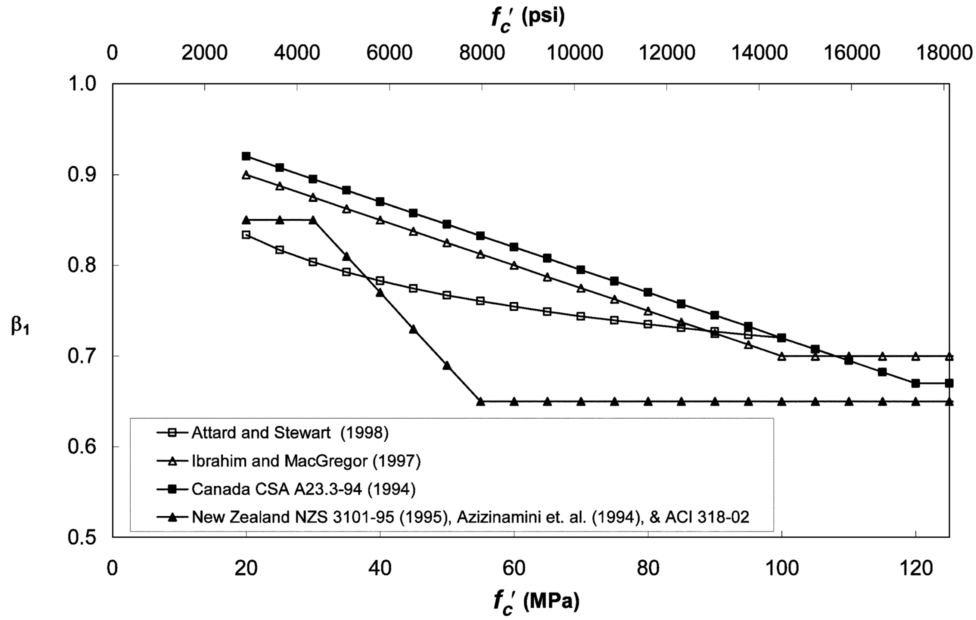


*Fig. 7.4—Proposed values of $\beta_1$ as a function of concrete compressive strength.*

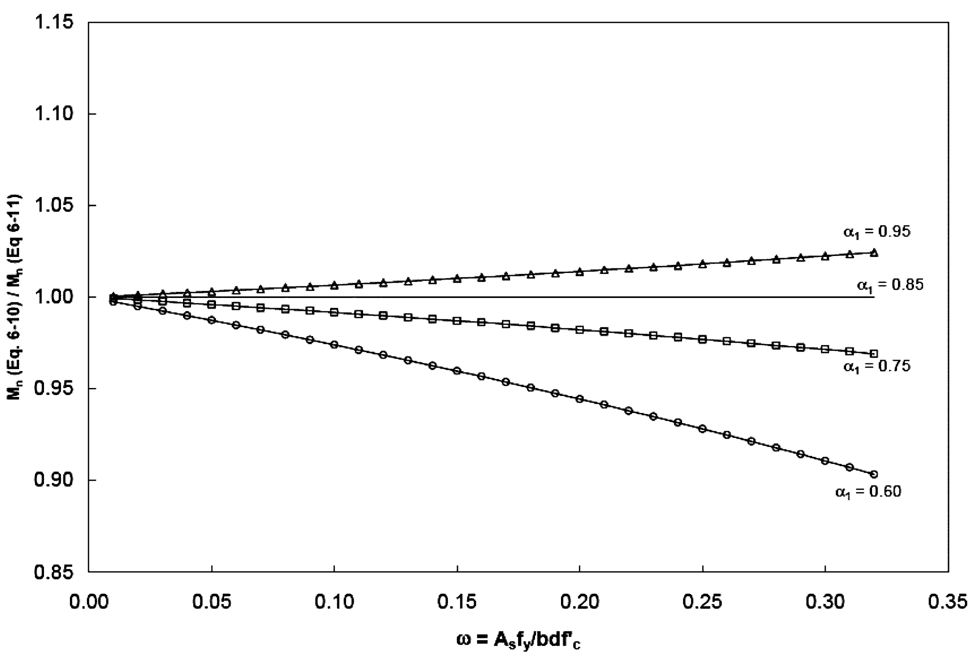


*Fig. 7.5—Effect of compressive stress distribution shape on calculated flexural capacity.*

The effect of the shape of the compressive stress distribution on the calculated flexural capacity can be seen in Fig. 7.5, which plots the ratio of the nominal moment capacity calculated by Eq. (7-10) to the nominal capacity calculated per current ACI 318 provisions using Eq. (7-11). Figure 7.5 is valid for under-reinforced behavior with singly reinforced rectangular sections and shows that the calculated flexural capacity deviates little as the shape of the stress block, and corresponding value of $\alpha_1$, is varied. Figure 7.6 shows a comparison of flexural strengths calculated using the ACI 318 equivalent rectangular stress block, a triangular stress block, and a stress block based on experimentally derived stress-strain curves, with measured flexural strengths for HSC beams with concrete strengths of 10,600 to 11,800 psi (73 to 81 MPa) (Wang et al. 1978b). Though using the actual stress-strain curves can be seen to give the best correlation with experimental data, the ACI 318 approach can be seen to provide a satisfactory conservative estimate. It is therefore suggested that the existing ACI 318 equivalent rectangular stress block is acceptable for use in the design of under-reinforced HSC beams.

For over-reinforced sections, failure occurs when the concrete reaches its ultimate strain in compression before yielding of the tensile reinforcement. As such, the section is compression-controlled and fails in a brittle manner, and the

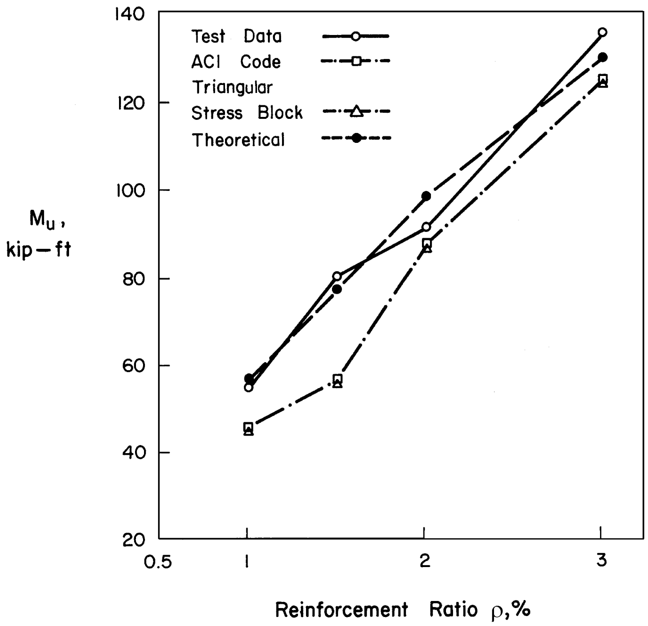


*Fig. 7.6—Comparison of flexural strength of beams for several compressive stress distributions (adapted from Wang et al. [1978b]).*

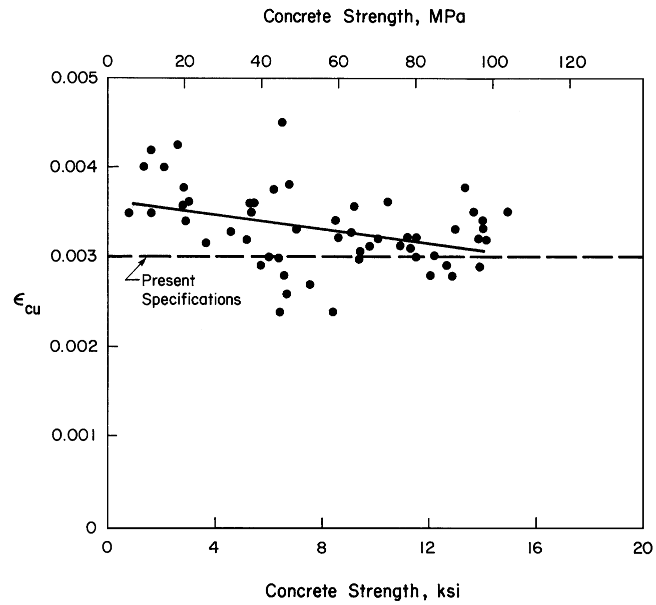


*Fig. 7.7—Ultimate concrete flexural strain $\varepsilon_{cu}$ versus compressive strength (adapted from Kaar et al. [1978]).*

flexural strength will be significantly affected by the shape of the concrete compressive stress block. The behavior of over-reinforced beams would be similar to that of eccentrically loaded columns, which are discussed in Section 7.5.1, and the same methods of calculating strength would thus be employed. Note, however, that experimental research has shown the ACI 318 equivalent rectangular stress block to be acceptable for use in the design of HSC beams (Mansur et al. 1997).

**7.3.2** *Limiting compressive strain and section ductility*—Whereas HSC reaches its peak stress at a compressive strain slightly higher than that for lower-strength concrete, the ultimate strain is lower for HSC, both in uniaxial compression tests and in beam tests (Martinez et al. 1984; Pastor et al. 1984; Ozbakkaloglu and Saatcioglu 2004). It has been suggested that this result is apparently due to energy release from the testing equipment. Figure 7.7 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 satisfactorily represent the experimental results for high-strength concrete as well as lower-strength concrete, although it is not as conservative for HSC. Additional experimental research has confirmed the ACI 318 limit of 0.003 to be satisfactory for HSC (Alca et al. 1997; Mansur et al. 1997).

Section ductility is important because it allows for plastic hinging to develop in beams. The formation of plastic hinges allows for adequate deformability to warn of impending failure, and allows for the redistribution of moments in structurally indeterminate systems. Considering the more limited strain capacity of unreinforced HSC in compression, it is necessary to evaluate the ductility of beams made of HSC.

Ductility can be defined in many ways, including as a ratio of the deflection (or cross-section curvature) at failure to the deflection (or curvature) at the load producing yield of the reinforcement

$$\mu = \frac{\Delta_u}{\Delta_y} \tag{7-12}$$

or

$$\mu = \frac{\psi_u}{\psi_y} \tag{7-13}$$

where $\mu$ is the ductility index; $\Delta_u$ is beam deflection at failure load; $\Delta_y$ is beam deflection at the load producing yielding of tensile steel; $\psi_u$ is cross-section curvature at failure load; and $\psi_y$ is cross-section curvature at the load producing yielding of tensile steel.

Early tests by Pastor et al. (1984) of singly reinforced HSC beams with no compression reinforcement and no confinement reinforcement found that a lower ductility resulted for beams made with higher concrete strengths. Subsequent tests by the same researchers, however, showed that the use of compression reinforcement and confinement reinforcement had a beneficial effect on the ductility of HSC beams.

Several researchers have investigated the plastic rotation capacity of HSC beams (Alca et al. 1997; Mansur et al. 1997; Peece and Fabbrocino 1999; Pendyala et al. 1996; Shin et al. 1989). The plastic rotation capacity can be defined as the deformability of the cross section after the yielding of the tensile reinforcement and resulting formation of a plastic hinge. In general, research has shown that the plastic rotation capacity of HSC beams is at least that of comparable normal-strength concrete beams. Alca et al. (1997) tested 12 simply supported beams with concrete strengths ranging from 7250 to 13,000 psi (50 to 90 MPa) and found that the plastic rotation capacity exceeded that predicted using the ACI 318 equivalent rectangular stress block with an extreme fiber concrete compressive strain of 0.003.

These observations can be explained using basic principles of mechanics. It can be seen that the neutral axis in a HSC beam will be closer to the extreme compression fiber than in a lower-strength concrete beam with the same quantity and strength of tension reinforcement, that is, a shallower compressive stress block will produce the same compressive resultant force required for internal equilibrium. This smaller neutral axis depth will lead to higher plastic strains in the tension reinforcement, resulting in ductile behavior. Research suggests that HSC may exhibit a lower extreme fiber compressive strain at failure but this effect is compensated for by the reduction in neutral axis depth. For under-reinforced HSC beams with quantities of reinforcement close to the balanced steel ratio, however, little ductility or plastic rotation capacity can be expected. For over-reinforced beams, little to no ductility should be expected, regardless of the concrete compressive strength.

Experimental testing of HSC beams has also demonstrated that, while confinement reinforcement does increase ductility, HSC beams are less sensitive to confinement because of less volume dilation of the concrete itself (Pendyala et al. 1996). This lesser volume dilation leads to less engaging of the confining pressure provided by the passive confining reinforcement. Furthermore, it should be noted that although HSC beams exhibit adequate plastic rotation capacity, they have generally been found to fail in an explosive manner when the extreme concrete compression fiber crushes (Pendyala et al. 1996; Wang et al. 1978b). High-strength concrete beams do not exhibit the softening typical of the failure of normal-strength concrete beams.

ACI 318 sets a lower limit on the amount of tensile reinforcement 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 ACI 318 expression for minimum steel ratio 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. Because the latter is known to be greater for HSC than for lower-strength concrete, the ACI 318 minimum reinforcement requirement depends on $f_c'$

$$\rho_{min} = \frac{3\sqrt{f_c'}}{f_y} \geq \frac{200}{f_y} \qquad (f_c' \text{ and } f_y \text{ in psi})$$

$$(7\text{-}14)$$

$$\rho_{min} = \frac{0.25\sqrt{f_c'}}{f_y} \geq \frac{1.38}{f_y} \qquad (f_c' \text{ and } f_y \text{ in MPa})$$

**7.3.3** *Shear and diagonal tension*—In the U.S., design for shear is based on conditions at factored loads. The total shear resistance is made up of two parts: $V_s$ provided by the stirrups, and $V_c$, 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.

There is general agreement among researchers that ACI 318-05 Eq. (11-3) and the more complex ACI 318-05 Eq. (11-5), shown below as Eq. (7-15) and Eq. (7-16), respectively

$$V_c = 2\sqrt{f_c'}\,b_w d \qquad (f_c' \text{ in psi})$$

$$(7\text{-}15)$$

$$V_c = 2\left(\frac{\sqrt{f_c'}}{6}\right)b_w d \qquad (f_c' \text{ in MPa})$$

$$V_c = \left(1.9\sqrt{f_c'} + 2500\rho_w\frac{V_u d}{M_u}\right)b_w d \leq 3.5\sqrt{f_c'}\,b_w d \qquad (f_c' \text{ in psi})$$

$$(7\text{-}16)$$

$$V_c = \left(\sqrt{f_c'} + 120\rho_w\frac{V_u d}{M_u}\right)\frac{b_w d}{7} \leq 0.3\sqrt{f_c'}\,b_w d \qquad (f_c' \text{ in MPa})$$

are conservative for both normal- and high-strength concrete (Kong and Rangan 1998; Mphonde and Frantz 1984, 1985; Pendyala and Mendis 2000; Russell and Roller 1990). Research has also indicated, however, that the level of conservatism in the ACI 318 $V_c$ prediction decreases as the concrete compressive strength increases (Pendyala and Mendis 2000). This trend is generally attributed to a reduction in aggregate interlock for HSC. 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 (Carrasquillo et al. 1981, 1982). 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 HSC beams can be expected to have a smooth surface, which are likely to be deficient in aggregate interlock.

This potential reduction in the concrete contribution term for HSC can be compensated for by increasing the minimum level of transverse shear reinforcement for higher-strength concretes. This minimum reinforcement level is intended to prevent brittle shear failures when inclined cracking occurs. In ACI 318-99 and earlier editions, concrete compressive strengths used in shear design were essentially limited to 10,000 psi (69 MPa), as unless sufficient additional transverse reinforcement was provided, higher concrete strengths were to be replaced by 10,000 psi (69 MPa) in all shear calculations. Essentially, twice the normal minimum transverse reinforcement was required for 10,000 psi (69 MPa) concrete, and three times the normal minimum transverse reinforcement was required for 15,000 psi (103 MPa) concrete.

An evaluation of available experimental data from several researchers (Angelakos et al. 2001; Collins and Kuchma

1999; Johnson and Ramirez 1989; Kong and Rangan 1998; Ozcebe et al. 1999; Pendyala and Mendis 2000; Russell and Roller 1990; Sarsam and Al-Musawi 1992; Yoon et al. 1996) led to a modification in the ACI 318 minimum shear reinforcement requirement. ACI 318-05 Eq. (11-13) specifies the minimum required transverse reinforcement as

$$A_{v,\,min} = 0.75\sqrt{f_c'}\frac{b_w s}{f_y} \geq 50\frac{b_w s}{f_y} \qquad (f_c' \text{ and } f_y \text{ in psi})$$

(7-17)

$$A_{v,\,min} = 0.0625\sqrt{f_c'}\frac{b_w s}{f_y} \geq 0.33\frac{b_w s}{f_y} \qquad (f_c' \text{ and } f_y \text{ in MPa})$$

This modification provides a smooth transition in the minimum transverse reinforcement over a range of concrete strengths, but still reflects the lower conservatism of the $V_c$ term found in the research data for HSC.

**7.3.4** *Torsion*—Limited experimental work has been completed on the behavior of HSC beams under torsion. Rasmussen and Baker (1995) tested 12 identical beams of varying concrete strengths and observed that the HSC beams exhibited higher cracking load and torsional capacity than identical normal-strength concrete beams. They also observed that for a given torque, the HSC beams had a higher torsional stiffness, smaller crack widths, and lower reinforcement stresses.

Koutchoukali and Belarbi (2001) tested nine reinforced concrete beams under pure torsion, with concrete strengths ranging from 7300 to 13,800 psi (50 to 95 MPa). They found that the ACI 318-05 equation for cracking torque

$$T_{cr} = 4\sqrt{f_c'}\frac{A_{cp}^2}{p_{cp}} \qquad (f_c' \text{ in psi})$$

(7-18)

$$T_{cr} = 0.33\sqrt{f_c'}\frac{A_{cp}^2}{p_{cp}} \qquad (f_c' \text{ in MPa})$$

(where $A_{cp}$ is the area enclosed by outside perimeter of concrete cross section, and $p_{cp}$ is the outside perimeter of concrete cross section) underestimated the experimentally determined cracking torque by approximately 31%. Determination of the cracking torque can be important in design because ACI 318 permits a redistribution of torsional moment to adjoining members in indeterminate systems when torsion is due to compatibility torsion. In such cases, the member is designed to resist only the cracking torque.

In cases where torsion is due to equilibrium torsion, the member should be designed to resist the full torsional moment. In the same study, it was found that the expression for the torsional strength of reinforced concrete beams given by ACI 318-05 Eq. (11-21) underestimated the experimentally determined torsional strength by approximately 11%. They concluded that the ACI 318 equation was appropriate for design with HSC. The results of that investigation also indicated that the torsional strength of the cross section was independent of concrete strength. This is consistent with the space-truss model approach used in the development of the ACI 318 expression, in which the concrete is assumed not to contribute torsional strength to the section at ultimate (that is, there is no $T_c$ term comparable to the $V_c$ term for shear).

Minimum reinforcing is required so that a beam subjected to torsion does not fail immediately upon cracking. Minimum requirements for transverse and longitudinal reinforcement are given by ACI 318-05 Eq. (11-23) and (11-24), respectively. The minimum transverse requirement was modified in ACI 318-05 to be consistent with the minimum requirement for transverse shear reinforcement, as discussed in Section 7.3.3.

Alternative expressions for the minimum torsional reinforcement requirements have been proposed in the literature (Koutchoukali and Belarbi 2001).

**7.3.5** *Bond, anchorage, and development length*—Present ACI 318 methods of design for development length and anchorage of tensile steel are based on tests involving normal-strength concretes. Over the last decade, a significant amount of research has been done in on development length and anchorage in HSC (Azizinamimi et al. 1999a,b; Esfahani and Rangan 1998a,b; Hwang et al. 1996; Yerlici and Özturan 2000). In general, the average bond strength between reinforcement and surrounding concrete has been shown to be higher for HSC (Esfahani and Rangan 1998a,b; Hwang et al. 1996; Yerlici and Özturan 2000). Bond failures in HSC beams, however, have been found to be more brittle than for normal concrete strengths. Azizinamimi et al. (1999a,b) tested 70 HSC beams with No. 8 or No. 11 (No. 25 or No. 36) bar tension lap splices, and concluded that the ACI 318 development length equations were satisfactory for prediction of strength, but did not ensure adequate ductility.

At this time, ACI 318 limits the concrete compressive strength used in development length and anchorage calculations to 10,000 psi (69 MPa). On the basis of observed research results, Azizinamimi et al. (1999a) recommend that for compressive strengths above 10,000 psi (69 MPa), the mandatory ACI 318 limit on compressive strength used in calculations be followed, or alternatively that the actual concrete compressive strength be used and that a specified minimum quantity of transverse confinement reinforcement be provided over the tension development or tension splice length

$$A_{sp} = 0.5nA_b\left(\frac{f_c'}{15{,}000}\right) \qquad (f_c' \text{ in psi})$$

(7-19)

$$A_{sp} = 0.5nA_b\left(\frac{f_c'}{103.4}\right) \qquad (f_c' \text{ in psi})$$

where $A_{sp}$ is the area of transverse reinforcement crossing the potential plane of splitting through the reinforcement being developed; $n$ is the number of spliced bars ($n = 1$ for a single bar); and $A_b$ is the area of a single spliced bar.

Additionally, it is recommended that the maximum spacing of transverse reinforcement (stirrups) not exceed 12 in. (300 mm).

**7.3.6** *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 6 to be approximately $11.7\sqrt{f_c'}$ for normalweight concretes with strengths in the range from 3000 to 12,000 psi (21 to 83 MPa). The ACI 318 value of $7.5\sqrt{f_c'}$ assumed for design may therefore be considered to be a lower bound for HSC. The assumption of a modulus of rupture lower than the actual value for a flexural member is neither conservative nor exaggerated, but simply results in an inaccurate prediction of cracking load. This will result in inaccurate estimation of both elastic and creep deflections. The calculation of a cracking load has no effect on the flexural strength.

The direct tensile strength is seldom measured, but is of interest in studying web-shear cracking in prestressed concrete members, for example. Both modulus of rupture and tensile splitting strength of HSC are typically higher than the corresponding values for lower-strength concrete. In this respect, empirically derived equations for flexural shear and torsional shear strength could be used conservatively for HSC calculations based on the lower-strength material. Other aspects of concern are discussed in Sections 7.3.3 and 7.3.4.

**7.3.7** *Deflections*—The main uncertainty in predicting elastic deflections of reinforced concrete beams are: a) the loading magnitudes and load arrangement; b) the elastic modulus $E_c$; and c) effective moment of inertia, which depends on the extent of cracking of the beam. The effective moment of inertia can be estimated as a function of the cracking moment and maximum applied moment, and the gross and cracked cross section moments of inertia. The equation for effective moment of inertia, $I_e$, included in the ACI 318 is

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad (7\text{-}20)$$

where $M_{cr}$ is the cracking moment; $M_a$ is the maximum applied moment; $I_g$ is the gross moment of inertia of section; and $I_{cr}$ is the moment of inertia of cracked transformed section.

Experimental research has shown that Eq. (7-20) may be used to predict the elastic deflections of reinforced concrete beams with reasonable accuracy (Leubkeman et al. 1985; Pastor et al. 1984; Paulson et al. 1989). Each of the values in Eq. (7-20), with the exception of the maximum applied moment, depends on cross section dimensions and material properties. In particular, the moment of inertia of the cracked transformed section depends on the elastic modulus of the concrete, and the cracking moment depends on the modulus of rupture of the concrete.

Thus, the accurate prediction of elastic deflections depends upon two material properties: the elastic modulus and the modulus of rupture. For the most accurate predictions, these values should be determined on the basis of testing.

Time-dependent deflections of beams due to creep and shrinkage are calculated by applying multipliers to computed elastic deflections. According to ACI 318, additional long-term deflections are obtained using the following multiplier

$$\lambda_\Delta = \frac{\xi}{1 + 50\rho'} \qquad (7\text{-}21)$$

where $\rho'$ is the reinforcement ratio for nonprestressed compression reinforcement; and $\xi$ is the time-dependent factor taken from ACI 318.

This procedure is generally valid for HSC members, but experimental data indicate that the multipliers are significantly less because of the lower creep coefficient typical of HSC. Experimental research (Leubkeman et al. 1985; Paulson et al. 1989) has provided an indication of long-term multipliers and their variation with time, up to about 1 year after loading. Results are summarized in Fig. 7.8, and some clear trends are evident:

1. For 3600 psi (25 MPa) concrete beams, 1-year multipliers of 0.85, 0.60, and 0.50 for beams with $\rho'/\rho$, equal to 0, 0.5, and 1.0, respectively, are less than the ACI 318 1-year values of 1.40, 1.10, and 0.80, which were determined for lower-strength concretes;
2. For HSC beams, deflection multipliers are still lower than the ACI 318 values. For example, for high-strength beams with no compression steel, the value of 0.55 at 1 year is only 40% of the ACI 318 value and 65% of the experimental value for lower-strength concrete; and
3. The influence of compression steel may be less important for HSC beams than for lower-strength beams. For beams of lower-strength concrete, addition of compression steel having an area equal to that of the tensile steel reduces 1-year deflections by 41%. For HSC, the beam with compression steel shows approximately a 35% reduction. This could be expected because the role of compression steel is mainly to reduce the creep of the concrete in the compression zone under sustained loads; the HSC with lower creep coefficient needs less help in this respect.

Based on the research program described previously, ACI 435R recommends that the long-term deflection multiplier $\xi$ be modified by a factor $\mu$ for HSC

$$\mu = 1.3 - 0.00005 f_c' \qquad (f_c' \text{ in psi})$$

$$\mu = 1.3 - 0.00725 f_c' \qquad (f_c' \text{ in MPa}) \qquad (7\text{-}22)$$

where $0.7 \le \mu \le 1.0$

Because of the variability in the actual creep values for all concretes, including HSC, however, experimental creep data should be used when available. The use of Eq. (7-21) and (7-22) should only be expected to provide approximate estimates of the actual time-dependent deflection.

**7.3.8** *Cyclic loading*—Limited experimental work has shown that HSC beams perform well under cyclic loads. Shin et al. (1989) found that deflection ductility indexes, as defined in Eq. (7-12), of at least 4.0 could be developed in HSC beams under cyclic loads. Additional research has

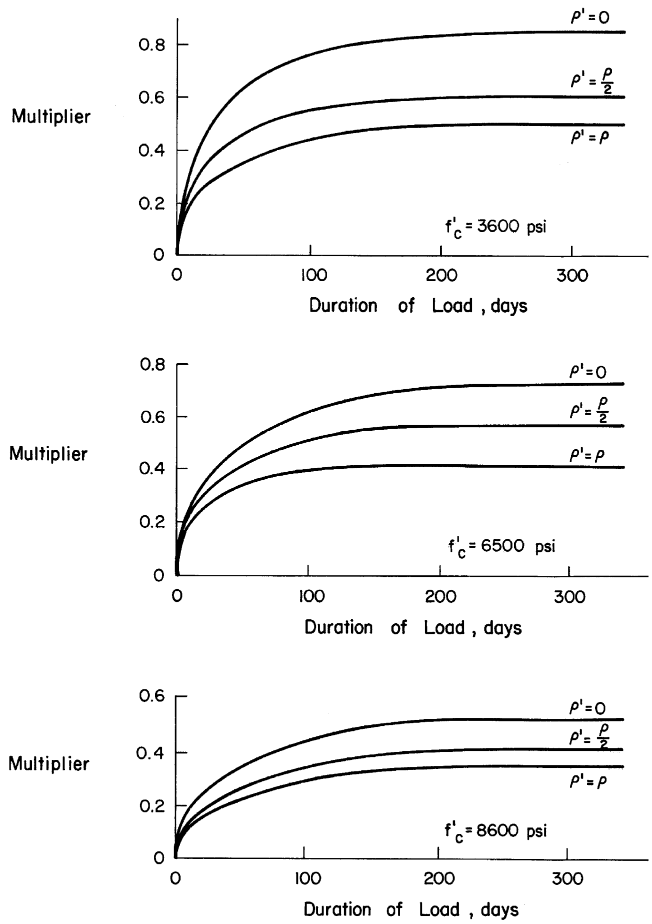


*Fig. 7.8—Long-term deflection multipliers for different strength concrete beams (adapted from Leubkeman et al. [1985] and Paulson et al. [1989]) (Note: 1 psi = 0.069 MPa.)*

indicated that HSC beams exhibit improved hysteretic performance, including better displacement ductility and smaller strength degradation, over comparable normal-strength concrete beams (Fang et al. 1994; Xiao and Ma 1998).

## 7.4—Prestressed concrete beams

Characteristics of HSC, 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. In particular, the compressive strength, modulus of rupture, modulus of elasticity, creep, and shrinkage properties of HSC all can affect the design and performance of prestressed concrete beams.

**7.4.1** *Allowable stresses*—Allowable stresses in prestressed concrete, at release of prestress and at service, are given in ACI 318. Allowable compressive stresses are given as a function of $f_c'$ at release or service, whereas allowable tensile stresses are given as a function of $\sqrt{f_c'}$ at the corresponding time. For HSC, there is sufficient experimental evidence to merit an increase in allowable tensile stresses, to a higher factor of $\sqrt{f_c'}$ than is currently specified by ACI 318, on the basis that the modulus of rupture is higher for HSCs than for normal-strength concretes (Gross and Burns 1999; Myers and Carrasquillo 1998).

Regardless of whether the allowable tensile stresses are increased or not, the use of HSC allows for more efficient prestressed concrete beam designs with longer spans, larger beam spacing, or both. For example, prestressed beams with span-depth ratios as large as 35 have been used in the construction of highway bridges with 56-day concrete compressive strengths of 14,000 psi (97 MPa) (Gross and Burns 2000). Alternatively, shallower depth sections may be used for a given span. This increased efficiency is a result of the higher absolute stresses permitted at release and service, which allow for utilization of larger prestress forces. The resulting benefits of using HSC for pretensioned girders are illustrated in terms of a computed relative cost index in Fig. 7.9 (Russell et al. 1997).

**7.4.2** *Prestress losses*—Prestress losses are affected significantly by the use of HSC. For a given level of prestress force and cross section size, the use of HSC will reduce prestress losses because of the higher elastic modulus and lower creep properties of HSC. These effects, however, can be offset by a higher absolute sustained concrete stress.

Many prestress loss computation methods, such as those given in the AASHTO LRFD Specifications (AASHTO 2004) and the *PCI Design Handbook* (PCI 2004), are empirical in nature, and were developed for concretes with compressive strengths of 6000 psi (41 MPa) or lower. These methods tend to significantly overestimate prestress losses for HSC members because they do not account for the higher elastic modulus and lower creep typical of HSC (Gross 1998). Note that ACI 318 does not provide specific prestress loss equations, except for friction losses in post-tensioned members.

Prestress loss equations for the design of HSC pretensioned girders have been developed and are proposed in *NCHRP Report 496* (Tadros et al. 2003). These equations are necessarily more complex than prestress loss equations for lower-strength concretes because of the increased sensitivity of HSC designs to several factors that affect prestress losses. These parameters include material properties such as creep, shrinkage, and modulus of elasticity, as well as environmental conditions and construction sequence. The NCHRP 496 method is derived based on the principles of mechanics and is transparent, allowing for input of material properties and other known parameters. As a result, the method generally results in better estimation of prestress losses for HSC members.

A comparison of measured and predicted prestress losses is shown in Fig. 7.10 for a 13,300 psi (92 MPa) HSC prestressed beam from a high-performance concrete demonstration bridge project in Texas. In all cases, predicted values were computed using measured material properties and considering the actual construction sequence. The NCHRP 496 loss equations can be seen to result in significantly better prediction than the other methods.

**7.4.3** *Time-dependent deflections*—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 very small net deflections associated

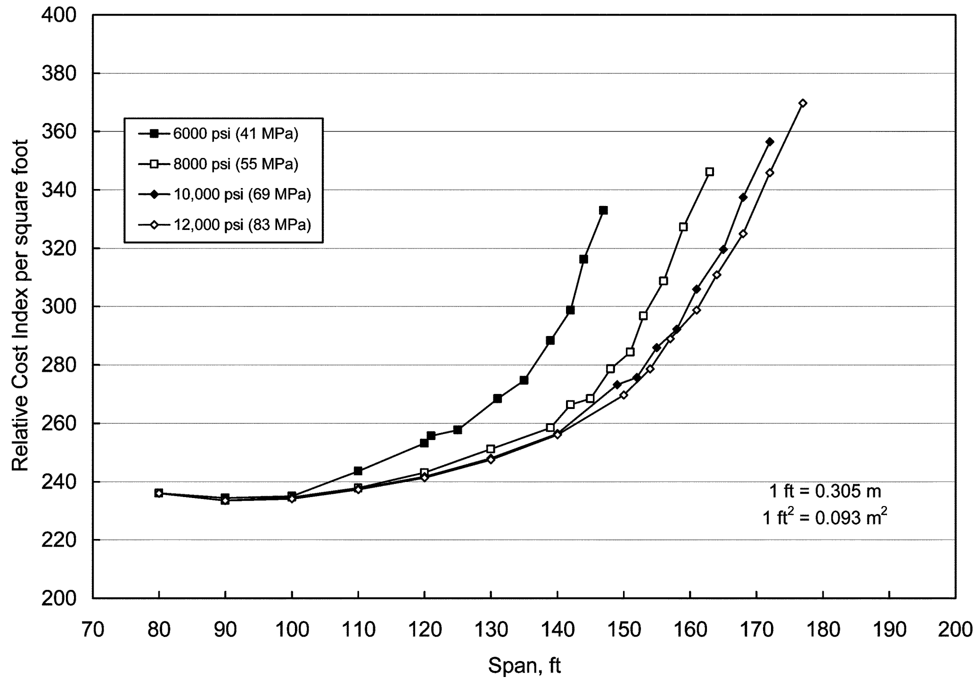


*Fig. 7.9—Relative cost index for pretensioned girders of different concrete strengths (Russell et al. 1997).*

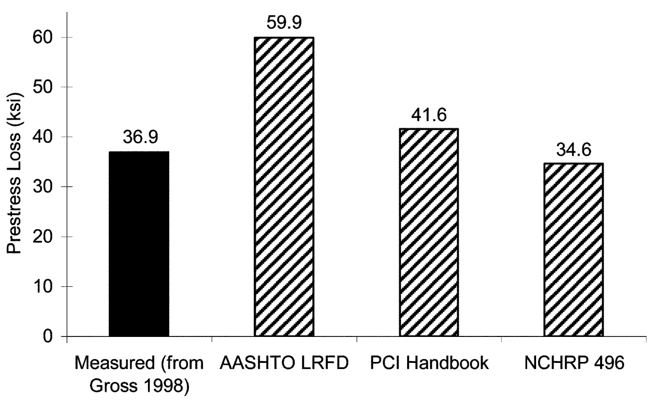


*Fig. 7.10—Comparison of measured and predicted prestress losses for a 13,300 psi (92 MPa) pretensioned girder.*

with all sustained loads for typical span-depth ratios. As HSC is used, however, span-depth ratios may be increased substantially, and small variations in creep or modulus of elasticity properties can have significant effects on the camber and deflection behavior of prestressed beams (Gross 2000).

**7.4.4** *Transfer and development length of strand*—Current ACI 318 design equations for transfer length and development length of pretensioned strands were developed from testing performed in the late 1950s and early 1960s (Buckner 1995; Tabatabai and Dickson 1993). These equations indicate that both transfer length and development length are independent of concrete strength. The testing, however, only included concrete strengths up to approximately 5000 psi (34 MPa). With the advent of higher-strength concretes, additional testing and analyses have been performed (Abrishami and Mitchell 1993; Ahlborn et al. 1995; Barnes and Burns 2000; Bruce et al. 1994; Castrodale et al. 1988; Cousins et al. 1990; Gross and Burns 1995; Kahn et al. 2002; Kowalsky et al. 2002; Mitchell et al. 1993; Ozyildirim and Gomez 2000; Roberts-Wollman et al. 2000; Russell and Burns 1996; Shahawy et al. 1992; Shing et al. 2000; Zia and Mostafa 1977). The results, in general, indicate that transfer length and development length decrease with an increase in concrete compressive strength. Several equations have been developed that incorporate a concrete strength factor. Researchers, however, have not been able to clearly quantify the effects of concrete strength because of large scatter in the measured data.

## 7.5—Eccentrically loaded columns

As mentioned in Section 7.2, there are significant advantages to using HSC in columns, especially those with relatively small eccentricities. Much of the research that has been completed on HSC columns under eccentric loading is summarized in ACI 441R. In the interest of brevity, only an overview of eccentrically loaded HSC columns is provided in this report. The reader is referred to ACI 441R for a more thorough treatment of the subject.

**7.5.1** *Combined flexural and axial loads for short columns*—Sections under combined flexural and axial load represent a transition between the extreme cases of pure axial load and pure flexure discussed in Sections 7.2.1 and 7.3.1, respectively. For design, it is common practice to develop a flexure-axial load interaction diagram in which points on the diagram represent unique combinations of axial load and bending moment magnitudes that cause failure. As for the case of pure bending, failure is generally assumed to occur at the point in which the extreme concrete compression fiber reaches a strain of 0.003 (ACI 318).

It was pointed out in discussing beams in Section 7.3.1 that the shape of the compressive stress distribution in HSC 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. 7.1. For under-reinforced concrete beams, with strength controlled by the yield strength of the reinforcement (that is, "tension-controlled"), the actual shape of the compressive stress block used in calculation of the nominal flexural strength is of little importance as long as the internal lever arm to the compressive resultant is close to the true value. The conventional rectangular stress block suggested by ACI 318 and discussed in Section 7.3.1, and equations for determining nominal flexural strength based on the rectangular stress block will normally be satisfactory.

In the case of combined bending and axial load, 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 of compression-controlled behavior, a more accurate representation of the concrete compressive stress block is important, such that the neutral axis location, tensile steel strain, and tensile steel stress are accurately computed.

ACI 441R reports conflicting experimental results over whether the current ACI 318 equivalent rectangular stress block approach is accurate for HSC columns. Some researchers have found the ACI 318 approach to overestimate the flexural strength at a given axial load. Ibrahim and MacGregor (1996) tested 94 eccentrically loaded HSC columns and found that more than half of them failed at flexural strengths lower than predicted by ACI 318. Azizinamini et al. (1994) reported that the threshold at which ACI 318 becomes unconservative is approximately 10,000 psi (70 MPa).

Several refinements to the equivalent rectangular stress block approach used by ACI 318 have been proposed as discussed in Section 7.3.1 (Attard and Stewart 1998; Azizinamini et al. 1994; CSA A23.3; Ibrahim and MacGregor 1997; NZS 3101). The proposed refinements generally follow a two-parameter equivalent rectangular stress block, like that of ACI 318, which can be seen in Fig. 7.2. Only two parameters, $\alpha_1$ and $\beta_1$, are needed to define the stress block. Proposals for the factor $\alpha_1$ are summarized in Fig. 7.3, and proposals for the factor $\beta_1$ are summarized in Fig. 7.4.

For cases of high eccentricity, with low axial loads and high bending moments such that the combination causes under-reinforced behavior, the existing ACI 318 equivalent rectangular stress block may be considered adequate. If a complete interaction diagram is to be developed, however, it is recommended that a single stress block approach, such as one of those suggested in the literature and summarized in Fig. 7.3 and 7.4, be used in computing all points on the diagram.

**7.5.2** *Slenderness effects*—The moment magnification method for dealing with slenderness effects in reducing the strength of reinforced concrete columns should be valid for HSC, because this approach is largely independent of material properties. 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 HSC may at least be questioned, recognizing the significantly lower creep coefficient for HSC. In addition, calculations should incorporate measured values of the modulus of elasticity for improved accuracy.

Note that HSC columns, compared with normal-strength concrete columns supporting identical loads, will have reduced cross sections, and thus are more likely to require consideration of slenderness effects.

# CHAPTER 8—ECONOMIC CONSIDERATIONS

## 8.1—Introduction

As previous chapters have demonstrated, HSC continues to be a state-of-the-art material and, like most state-of-the-art materials, it commands a premium price. In many areas and for many uses, the benefits of HSC more than compensate for the increased costs of raw materials and quality control. This chapter contains summaries of several cost studies about the use of HSC as well as a discussion about the effect of raw materials and quality control on overall costs.

## 8.2—Cost studies

**8.2.1** *Buildings*—In 1975, Chicago-based structural engineers Schmidt and Hoffman (1975) compiled charts indicating the cost of supporting 100,000 lb (445 kN) of service load came to \$5.02 per story with 6000 psi (41 MPa) concrete, \$4.21 with 7500 psi (52 MPa) concrete, and \$3.65 with 9000 psi (62 MPa) concrete.

*Architectural Record* (1976) noted that " ...a 30 x 30 in. column of 6000 psi concrete might require an amount of reinforcing steel equal to 4% of the column area for a given load, whereas the same column in 9000 psi would require only 1% steel—the minimum allowed by code."

In 1998, Moreno conducted a pricing study dramatically demonstrating the cost advantage of replacing vertical steel reinforcement in short tied columns with HSC. This study was made for a 40 in. (1000 mm) square column supporting a factored design load ($1.4D + 1.7L$) of 1000 kips (4.45 MN) and based on the following prices at the time the document was developed:

| | | |
|---|---|---|
| Reinforcing steel | \$1200/ton in place | \$1323/metric ton in place |
| 6000 psi (41 MPa) concrete | \$93.92/yd$^3$ in place | \$122.71/m$^3$ in place |
| 8000 psi (55 MPa) concrete | \$99.08/yd$^3$ in place | \$129.59/m$^3$ in place |
| 10,000 psi (69 MPa) concrete | \$104.34/yd$^3$ in place | \$136.47/m$^3$ in place |
| 12,000 psi (83 MPa) concrete | \$111.90/yd$^3$ in place | \$146.36/m$^3$ in place |
| 14,000 psi (97 MPa) concrete | \$156.90/yd$^3$ in place | \$205.22/m$^3$ in place |
| 16,000 psi (110 MPa) concrete | \$180.00/yd$^3$ in place | \$235.43/m$^3$ in place |
| Formwork | \$3.00/ft$^2$ in place | \$32.29/m$^2$ in place |

As Fig. 8.1 shows, using HSC with a minimum amount of steel reinforcement is the most economical solution.

The cost effectiveness of using HSC to carry a compressive load is illustrated in Fig. 8.2. As the compressive strength of

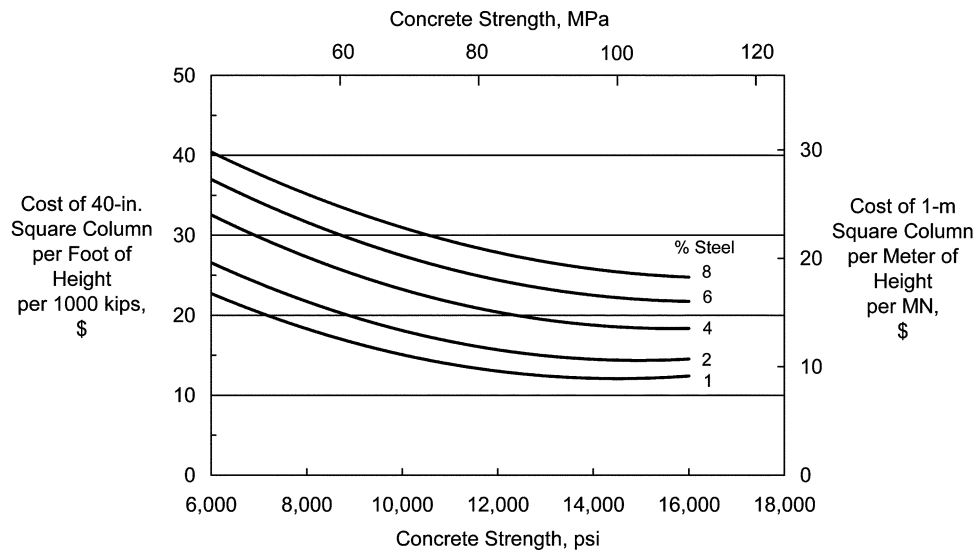


*Fig. 8.1—Cost of columns (data compiled by ACI Committee 363 from Moreno [1998]).*

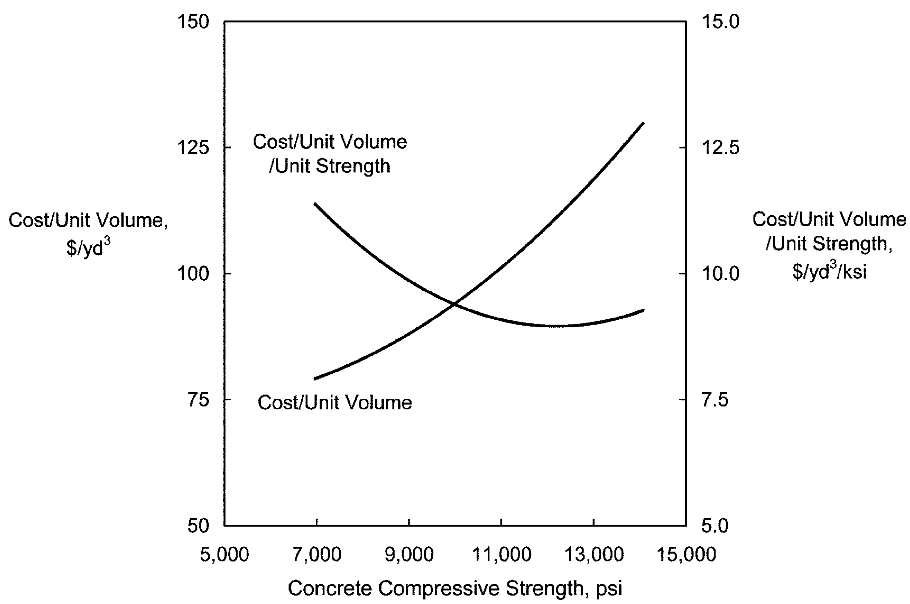


*Fig. 8.2—Variation of concrete costs with concrete strength. (Note: $1/yd$^3$ = $1.308/m$^3$; $1/yd$^3$/ksi = $0.1897/m$^3$/MPa; and 1 psi = 6.895 kPa).*

concrete increases, the cost per unit volume of concrete also increases. In contrast, the cost/unit volume/unit strength decreases as concrete strength increases to approximately 12,000 psi (85 MPa).

Because maximum column size is important for architectural and rental reasons, the ability to limit column sizes for taller structures often allows the use of a concrete solution instead of one of structural steel. Moreno (1998) compared the cost of a 23-story commercial building assuming that the maximum available concrete strengths were either 6000 or 12,000 psi (41 or 83 MPa). The use of 6000 psi (41 MPa) concrete resulted in a column size of 34 x 34 in. (865 x 865 mm) and a resulting cost of $0.92/ft$^2$ ($9.90/m$^2$) of tributary floor area. The use of 12,000 psi (83 MPa) concrete reduced the column size to 24 x 24 in. (610 x 610 mm) and the cost to $0.52/ft$^2$ ($5.60/m$^2$). The major reduction in cost resulted from the decrease in the amount of formwork required for the smaller columns.

Moreno (1998) also reported a cost comparison for the exterior column of a 62-story building based on maximum concrete strengths ranging from 7000 to 18,000 psi (48 to 124 MPa). The design was based on minimizing the column size for each concrete strength while not allowing a reinforcement percentage below 1.0%. The cost comparison was made for both the column and the caisson below the column. The results are summarized in Table 8.1 and show that, despite the higher unit cost for HSC, the total cost of both the column and caisson became less as the concrete strength increased. Again, the savings in formwork, reinforcement, and concrete volume were greater than the increase in the unit cost of the HSC.

**8.2.2** *Bridges*—Beginning in the 1990s, there has been a trend in North America toward increased use of HSC in prestressed concrete bridge girders (Russell 1997). Using HSC allows for longer span lengths, wider girder spacing, or shallower sections. Longer span lengths allow for a reduction in the number of substructure elements. Wider girder spacing results in fewer girders for a given width of bridge. Shallower sections allow for less height of embankments or the approaches. All of the benefits offset the higher initial cost for the HSC girders.

With bridges, there is an additional cost associated with maintenance and repair. Using HSC with its enhanced durability is likely to result in less maintenance and longer service life. With the introduction of life-cycle costing, the long-term economic benefits are likely to more than offset the premium costs for initial construction.

In Quebec, Canada, two design alternatives were compared for the construction of the Saint-Remi Bridge crossing Autoroute 50. The HSC alternative saved 5% on the first cost (Coulombe and Ouellet 1995). A preconstruction study for two prototype bridges on Highway 407 in Toronto, Canada, showed a savings of Can\$1.39/ft$^2$ (Can\$15/m$^2$) for the deck and Can\$30,000 for each precast girder that could be eliminated by using a higher-strength concrete (Bickley 1999).

Before the construction of the East River Bridge in Nova Scotia, a detailed life-cycle cost analysis showed the following potential savings on first cost and life-cycle costs (Fletcher 1997).

| | Conventional-strength concrete | HSC |
|---|---|---|
| Construction costs | Can$484,697 | Can$444,815 |
| Life cycle costs (70 years)* | Can$578,827 | Can$545,587 |

*Net present value.

The predicted first cost savings of 8% were confirmed by the bid price of the successful contractor.

Ralls (1998) compared the initial cost of two high-performance concrete bridges in Texas. The Louetta Road Overpass consists of two adjacent bridges with spans of 121.5, 135.5, and 134.0 ft (37.0, 41.3, and 40.8 m). High-strength concrete with compressive strengths up to 13,100 psi (90.4 MPa) were required. The beams were bid at \$100/linear ft (\$328/linear m). At that time, other normal-strength concrete beams of the same cross section had an average cost of \$120/linear ft (\$394/linear m). The unit bridge cost for the total structure was \$24.09/ft$^2$ (\$259/m$^2$) of deck area for the two bridges. This is comparable to the average of \$23.61/ft$^2$ (\$254/m$^2$) of deck area for the 12 normal-strength concrete bridges built on the same project and slightly less than other projects at that time. The cost savings on the girders did not translate into a direct savings on the bridge cost because of other high-performance concrete features of the bridge (Myers and Carrasquillo 1998).

**Table 8.1—Cost of exterior column**

| Maximum concrete strength, psi (MPa) | 18,000 (124) | 14,000 (97) | 10,000 (69) | 7000 (48) |
|---|---|---|---|---|
| Concrete cost, \$/yd$^3$ (\$/m$^3$) | 175.00 (229.00) | 145.33 (190.09) | 84.25 (110.20) | — |
| Square column size, in. (mm) | 40 (1016) | 44 (1118) | 48 (1219) | 54 (1372) |
| Column cost*, \$/ft$^2$ (\$/m$^2$) | 1.84 (19.80) | 2.10 (22.60) | 2.85 (30.68) | 3.86 (41.55) |
| Column cost*, \$/ft$^2$ (\$/m$^2$) | 0.53 (5.70) | 0.56 (6.03) | 0.57 (6.14) | 0.60 (6.46) |

*Unit costs baed on tributary floor area to column and caisson loads.

The North Concho River Overpass in San Angelo consists of an eight-span, high-performance concrete eastbound bridge with spans ranging from 64 to 157 ft (20 to 48 m) and a nine-span normal-strength concrete westbound bridge with spans ranging from 37 to 140 ft (11 to 43 m). Using HSC in the prestressed concrete girders on the eastbound bridge permitted either fewer girders to be used when span lengths were fixed or longer spans to be used. This resulted in one less span for the eastbound bridge. Concrete strengths for the prestressed girders on the eastbound bridge varied from 5800 to 14,700 psi (40 to 101 MPa), and on the westbound bridge from 5000 to 8900 psi (35 to 61 MPa). The beams for the bridges were bid at \$115/linear ft (\$377/linear m). The contractor spread the cost of the HSC beams to all beams. Based on contractor-supplied information, the costs for the eastbound and westbound bridges were calculated. The calculations indicated a cost of \$90/linear ft (\$295/linear m) for normal-strength AASHTO Type IV beams, and \$186/linear ft (\$610/linear m) for HSC beams. The cost of high-strength beams was approximately twice the cost of the normal-strength concrete beams. This, however, only translated into a 16% cost increase for the completed bridge—\$47.39/ft$^2$ versus \$40.91/ft$^2$ (\$510/m$^2$ versus \$440/m$^2$). The increased cost for the HSC girders, however, was not entirely related to the use of HSC. Long-span girders required a two-stage pretensioning and post-tensioning sequence as well as special transportation and handling precautions, which also increased the costs (Tadros et al. 1999). In Nebraska, two similar bridges were constructed. One bridge used 11 prestressed concrete girders with normal-strength concrete. The other bridge used seven girders with HSC. The bid prices were \$19.89/ft$^2$ (\$214.09/m$^2$) for the normal-strength concrete bridge, and \$22.13/ft$^2$ (\$238.21/m$^2$) for the HSC bridge. Subsequently, it was estimated that if six lines of girders had been used instead of seven, the cost would have dropped to \$20.57/ft$^2$ (\$221.41/m$^2$).

In Virginia, the use of HSC in the prestressed concrete girders of two bridges enabled the use of five girder lines rather than the seven lines that would have been required with conventional-strength concrete (FHWA 2003). The total cost per unit area of bridge deck for each bridge was less than the average federal-aid cost for bridges built in the same year.

An optimization study by Russell et al. (1997) compared the costs of prestressed concrete bridges constructed with different girder cross sections and different concrete strengths for various span lengths. They concluded that the maximum useful concrete compressive strength was in the range of 9000 to 10,000 psi (62 to 69 MPa) when 0.5 in. (13 mm) diameter strands at 2 in. (50 mm) centers was used. Beyond these strength levels, the efficiency of the additional strands decreased as they had to be placed in the webs. Consequently, the cost began to increase rapidly. A similar conclusion was reached by Zia et al. (1989).

Using 0.6 in. (15 mm) diameter strands at 2 in. (50 mm) centers allowed a more effective use of higher-strength concretes (Russell et al. 1997). Some cross-sectional shapes were more cost effective, depending on span length. The study also showed that girder spacing wider than 14 ft (4 m) may not be economical, as the advantage of fewer girders is offset by the additional cost of the deck spanning a longer distance.

High-strength concrete has proven to be beneficial in long-span cable-stayed bridges. In bidding to build a cable-stayed bridge across the Ohio River, a concrete superstructure proposal was lower than a steel superstructure by 29%, or roughly $10 million. The two lane crossing between Huntington, WV and Proctorville, OH included the first major asymmetrical stayed-girder structure in the U.S. The bridge has a main span of 900 ft (274 m) 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 of $33.3 million. The designer specified box girders only 5 ft (1.5 m) deep, cast of 8000 psi (55 MPa) HSC (*Engineering News-Record* 1981).

There are also savings on rehabilitation projects. On the rehabilitation of the deck of the Fraser River Bridge in British Columbia, Canada, a savings of Can$1.39/ft$^2$ (Can$15/m$^2$) was realized. Using HSC also resulted in a shorter construction period. High-early-strength concrete also shortened the time to rehabilitate the deck of the Jacques-Cartier Bridge in Quebec, Canada. Because the rehabilitation resulted in a temporary 1.9 mile (3 km) detour, the socioeconomic savings of the more rapid repair were calculated to be Can$150,000 (Bickley 1999).

## 8.3—Selection of materials

The selection of concrete constituent materials affects the cost per unit volume of concrete. For normal-strength concrete, the highest constituent material cost is for the cement. Using fly ash in HSC, however, can reduce the total cost of the cementitious materials. In contrast, using silica fume can result in an increase in the cost of the cementitious materials. Using silica fume also requires using a HRWRA, which further increases the cost. For very high-strength concretes, however, using silica fume and a HRWRA may be the only way to achieve the required strength.

The Royal Bank Plaza Project in Toronto, Canada, a 43-story building constructed from 1973 to 1976, was one of the first buildings to use fly ash in HSC. All of the various strength concrete mixtures on the project included local fly ash. This resulted in a savings of approximately Can$100,000 over the contract, and produced concretes with extremely good fresh and hardened properties (Bickley and Payne 1979).

There are economic benefits to be obtained by specifying compressive strengths at 56 or 90 days rather than the traditional age of 28 days. This allows the later-age strength gain to be used. In many applications, the high strength is not required at 28 days and, in the case of columns in buildings, may not be needed until the building is complete. It may be unnecessary, therefore, to specify the strength at 28 days. The economical advantages of specifying later-age strengths are particularly beneficial when fly ash or slag cements are used in the concrete. Consequently, many specifications for HSC now allow the specified compressive strength to be achieved by 56 or 90 days.

## 8.4—Quality control

Whereas selection of materials will influence costs, another factor more closely associated with using HSC is the cost of the increased testing, quality control, and inspection. These activities are essential to ensure that the required quality and consistency of the HSC are achieved.

In the Royal Bank Plaza Project, a number of precautions were necessary. The concrete supplier was required to have a QC 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 arrived at the site and that any minor adjustments would be made before sending the truck to the site. Regular visits were made to the batch plant to check batching procedures and to obtain test samples. Furthermore, a full-time technician was employed to carry out sampling and testing on site.

On the Richmond-Adelaide Center, Toronto, built in 1978, not only did the supplier maintain full-time inspection on the site to ensure that the delivered material met requirements, but the engineers employed by the owner also maintained full-time inspection and regularly inspected the batch plant. Often, this type of stringent QC is required and is more commonplace today than in 1978.

For the Palace Hotel, the New York City Department of Buildings stipulated that at least two suppliers of concrete prequalify concrete mixtures for 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 that the mixing water temperature be limited to no more than 50°F (10°C) and that truck drums be hosed down if standing in direct sunlight. Furthermore, all trucks were limited to 10 yd$^3$ (7.6 m$^3$) loads, despite capacities of 16 yd$^3$ (12.2 m$^3$). Although professional inspection adds to cost in the short term, the continuing education of suppliers and concrete subcontractors in the areas of QC will ultimately create better concretes of all strengths and result in enhanced and more long-term economical use of materials.

The ACI "Guide to Quality Control and Testing of High-Strength Concrete" (ACI 363.2R) discusses quality control and testing practices for HSC. The guide indicates that small variations in mixture proportions and deviations from standard testing practices can have greater adverse effects on the actual or measured strengths of HSC than with normal-strength concrete. Consequently, the guide recommends several changes from normal procedures. Topics discussed in the guide include a preconstruction meeting; prequalification of concrete suppliers; preconstruction testing; concrete plant inspection; concrete delivery, placing, and curing; sampling and testing of concrete; prequalification of testing laboratories; and evaluation of compressive strength results.

Because the amount of quality control required with HSC is greater than with normal-strength concrete, the cost of quality control will be higher. Resulting benefits, however, include the ability to produce higher-strength concretes and a reduction in the variability of the concrete. The latter is beneficial in that the overdesign of concrete mixtures can be reduced at all strength levels.

### 8.5—Conclusions

The economic advantages of HSC in the columns of high-rise buildings have been established for many years. The economic advantages of using HSC in bridges have been demonstrated on several projects and comparison studies. These bridge projects have led the way in the use of HSC. As more bridges are built with HSC, it is anticipated that the short-term and long-term economic benefits will become more apparent.

## CHAPTER 9—APPLICATIONS

### 9.1—Introduction

High-strength concrete has been primarily used in high-rise buildings, long-span bridges, and offshore structures. This chapter summarizes some of the applications around the world. A more extensive list of HSC applications is given in a CEB (1994) report.

### 9.2—Buildings

The largest application of HSC in buildings has been in the columns of high-rise structures. HSC provides the most economical material to carry a compressive load while minimizing the interruption to rentable floor space. The data in Table 9.1 indicate a growth in the use of HSC in buildings throughout the world.

The early history of HSC columns in Chicago is described in *Task Force Report* No. 5 of the CCHRB (1977). Since 1972, more than 34 buildings in the Chicago area have been constructed with columns having a design compressive strength of 9000 psi (62 MPa) or greater.

In some high-rise buildings, smaller-size columns have been used to minimize the interruption to parking spaces in the lower stories. In the Richmond-Adelaide Center in Toronto, the use of HSC columns enabled the architect to increase the use of the underground parking garage by approximately 30%. In a 15-story parking garage at 900 N. Michigan Avenue, Chicago, HSC was specified for the columns to reduce their lateral stiffness, yet provide sufficient capacity to carry the vertical load. Consequently, when the floor system was post-tensioned, the amount of post-tensioning force being resisted by the columns was minimized (Russell

**Table 9.1—Buildings with high-strength concrete**

| Building | City | Year* | Total stories | Maximum design concrete strength, psi | Maximum design concrete strength, MPa |
|---|---|---|---|---|---|
| Trump Tower | New York | — | 68 | 8000 | 55 |
| Collins Place | Melbourne | — | 44 | 8000 | 55 |
| Helmsley Palace Hotel | New York | 1978 | 53 | 8000 | 55 |
| Larimer Place Condominiums | Denver | 1980 | 31 | 8000 | 55 |
| City Center Project | Minneapolis | 1981 | 52 | 8000 | 55 |
| NCNB Corporate Center | Charlotte | 1990 | 60 | 8000 | 55 |
| 499 Park Avenue | New York | — | 27 | 8500 | 59 |
| The Seine Johuku | Nagoya | — | 45 | 8700 | 60 |
| Rialto Tower | Melbourne | 1985 | 60 | 8700 | 60 |
| Bank of China Tower | Hong Kong | 1989 | 70 | 8700† | 60† |
| New Century Hotel | Beijing | 1990 | 31 | 8700† | 60† |
| Central Plaza | Hong Kong | 1992 | 78 | 8700† | 60† |
| Jin Mao | Shanghai | 1997 | 88 | 8700† | 60† |
| SEG Plaza | Shenzhen | 1998 | 75 | 8700† | 60† |
| Royal Bank Plaza | Toronto | 1975 | 43 | 8800 | 61 |
| Richmond-Adelaide Center | Toronto | 1978 | 33 | 8800 | 61 |
| Midcontinental Plaza | Chicago | 1972 | 50 | 9000 | 62 |
| Frontier Towers | Chicago | 1973 | 55 | 9000 | 62 |
| Water Tower Place | Chicago | 1975 | 79 | 9000 | 62 |
| River Plaza | Chicago | 1976 | 56 | 9000‡ | 62‡ |
| Chicago Mercantile Exchange | Chicago | 1982 | 40 | 9000§ | 62§ |
| Grande Arche de la Defense | Paris | 1988 | — | 9400 | 65 |
| Columbia Center | Seattle | 1983 | 76 | 9500 | 66 |
| Interfirst Plaza | Dallas | 1983 | 72 | 10,000 | 69 |
| Scotia Plaza | Toronto | 1988 | 68 | 10,000 | 69 |
| Platinum Tower | Panama | 1993 | 56 | 10,000 | 70 |
| Governor Phillip Tower | Sydney | 1993 | 54 | 10,000 | 70 |
| Eugene Terrace | Chicago | 1987 | 44 | 11,000 | 76 |
| Telecom Corporate | Melbourne | 1992 | 47 | 11,600 | 80 |
| Building D 3 | Brussels | — | 24 | 11,600† | 80† |
| Petronas Twin Towers | Kuala Lumpur | 1995 | 85 | 11,600† | 80† |
| Baiyoke Tower | Bangkok | 1996 | 90 | 11,600 | 80 |
| e-Tower | São Paulo | 2002 | 42 | 11,600 | 80 |
| 311 S. Wacker Drive | Chicago | 1988 | 70 | 12,000‖ | 83‖ |
| One Peachtree Center | Atlanta | 1990 | 62 | 12,000 | 83 |
| Society Tower | Cleveland | 1990 | 63 | 12,000 | 83 |
| Trump World Tower | New York | 2000 | 90 | 12,000 | 83 |
| 505 5th Avenue | New York | 2004 | 30 | 12,000 | 83 |
| BfG Building | Frankfurt | 1990 | 47 | 12,300† | 85† |
| Bay Adelaide Center | Toronto | 1991 | 57 | 12,300 | 85 |
| Dain Bosworth/ Nieman Marcus | Minneapolis | — | 39 | 14,000 | 97 |
| 900 N. Michigan Ave. | Chicago | 1986 | 15 | 14,000 | 97 |
| Pacific First Center | Seattle | 1987 | 45 | 14,000# | 97# |
| Two Union Square | Seattle | 1987 | 62 | 14,000# | 97# |
| 225 W. Wacker Drive | Chicago | 1988 | 30 | 14,000 | 97 |
| 111 George Street | Brisbane | 1993 | 27 | 14,500 | 100 |
| De Geno Leasing House | Eschborn | 1995 | — | 15,200† | 105† |
| Herriot's | Frankfurt | 2002 | 18 | 18,100 | 125 |
| Brillia Tower | Tokyo | 2004 | 45 | 18,850 | 130 |

*Year in which high-strength concrete was cast.
†Cube strength
‡Two experimental columns of 11,000 psi (76 MPa) were included.
§Two experimental columns of 14,000 psi (97 MPa) were included.
‖Nine-thousand psi (62 MPa) concrete used in floor slabs at lower levels.
#Nineteen-thousand psi (131 MPa) concrete indirectly specified to achieve a high modulus of elasticity.

**Table 9.2—Bridges with high-strength concrete**

| Name | Country | Year | Maximum span | | Maximum design concrete strength | |
|---|---|---|---|---|---|---|
| | | | ft | m | psi | MPa |
| East Huntington Bridge, West Virginia | U.S. | 1984 | 900 | 274 | 8000 | 55 |
| Annacis Bridge, British Columbia | Canada | 1986 | 1526 | 465 | 8000 | 55 |
| Route 104 Bridge, New Hampshire | U.S. | 1996 | 65 | 20 | 8000 | 55 |
| Route 40, Brookneal, Virginia | U.S. | 1996 | 80 | 24 | 8000 | 55 |
| Federation Bridge, Prince Edward Island | Canada | 1997 | 820 | 250 | 8000 | 55 |
| Nitta Highway Bridge | Japan | 1968 | 98 | 30 | 8500 | 59 |
| Kaminoshima Highway Bridge | Japan | 1970 | 282 | 86 | 8500 | 59 |
| Joigny Bridge | France | 1989 | 150 | 46 | 8700* | 60* |
| Stongasundet | Norway | 1990 | 249 | 76 | 8700 | 60 |
| Portneuf, Quebec | Canada | 1992 | 84 | 25 | 8700 | 60 |
| Normandy Bridge | France | 1994 | 2808 | 856 | 8700 | 60 |
| Mirabel, Quebec | Canada | — | 136 | 42 | 8700 | 60 |
| Wanxian Yangtze River Highway | China | 1997 | 1378 | 420 | 8700* | 60* |
| Raftsundet Bridge | Norway | 1998 | 878 | 298 | 8700*† | 60*† |
| Tower Road Bridge, Washington | U.S. | 1981 | 161 | 49 | 9000 | 62 |
| Esker Overhead, British Columbia | Canada | 1990 | 164 | 50 | 9000 | 62 |
| Pertuiset Bridge | France | 1988 | 433 | 132 | 9400 | 65 |
| Halgelandsbrua | Norway | 1990 | 1394 | 425 | 9400 | 65 |
| Varoad | Norway | 1994 | 853 | 260 | 9400* | 65* |
| Great Belt Link | Denmark | 1998 | 5328 | 1624 | 9400 | 65 |
| Stolma Bridge | Norway | 1998 | 987 | 301 | 9430* | 65* |
| Braker Lane Bridge, Texas | U.S. | 1986 | 85 | 26 | 9600 | 66 |
| Fukamitso Highway Bridge | Japan | 1974 | 85 | 26 | 10,000 | 69 |
| Deutzer Bridge | Germany‡ | 1978 | 607 | 185 | 10,000*† | 69*† |
| Virginia Avenue, Richlands, Virginia | U.S. | 1997 | 74 | 23 | 10,000 | 69 |
| I-25 over Yale Avenue, Colorado | U.S. | 1998 | 115 | 35 | 10,000 | 69 |
| Route 22 at Mile Post 6.57, Ohio | U.S. | 1998 | 117 | 36 | 10,000 | 69 |
| State Route 18 over State Route 516, West Virginia | U.S. | 1998 | 137 | 42 | 10,000 | 69 |
| Charenton Canal Bridge, Louisiana | U.S. | 1999 | 73 | 22 | 10,000 | 69 |
| Highway 199 over Uphapee Creek, Alabama | U.S. | 2000 | 114 | 35 | 10,000 | 69 |
| US 401 Over Neuse River, North Carolina | U.S. | 2000 | 92 | 28 | 10,000 | 69 |
| State Route 920 Over I-75, Georgia | U.S. | 2002 | 127 | 39 | 10,000 | 69 |
| Zwickaver Mulde River | Germany | 2001 | 128 | 39.0 | 10,150* | 70* |
| Ootanabe Railway Bridge | Japan | 1973 | 79 | 24 | 11,400 | 79 |
| Akkagawa Railway Bridge | Japan | 1976 | 150 | 46 | 11,400 | 79 |
| Elorn Bridge | France | 1994 | 1312 | 400 | 11,600 | 80 |
| 120th Street and Giles Road, Nebraska | U.S. | 1996 | 75 | 23 | 12,000 | 83 |
| Louetta Road Overpass, Texas | U.S. | 1998 | 136 | 41 | 13,100 | 90 |
| North Concho River Overpass, Texas | U.S. | 1998 | 157 | 48 | 14,700 | 101 |
| CNT Super Bridge | Japan | 1993 | 131 | 40 | 14,800 | 102 |

*Cube strength.
†Lightweight.
‡Former West Germany.

1994). For the precast concrete columns of the City Center Parking Garage in White Plaines, NY, 10,000 psi (69 MPa) compressive strength concrete was used to reduce column weight.

## 9.3—Bridges

Table 9.2 lists some of the bridges around the world built with HSC. The first significant application of HSC in bridges occurred in Japan in the 1970s when compressive strengths up to 11,000 psi (76 MPa) were used in railway bridges. Nagataki (1978) reported that strengths of 11,400 psi (79 MPa) could be easily obtained in the field.

Before the mid-1990s, the use of HSC in bridges in the U.S. was very limited. Use began to increase as the result of the Federal Highway Administration (FHWA) initiative to implement the use of high-performance concrete in bridges. The FHWA program included the construction of demonstration bridges and dissemination of the technology and results at showcase workshops. Initially, a total of 18 bridges in 13 states were included in the program. Results from the projects have been compiled by the FHWA (2003). These demonstration bridges clearly indicate that HSC in prestressed concrete girders can result in longer span lengths, wider girder spacing, or shallower sections. Similar development programs have been used in France, Germany, and Japan.

The highest-strength concrete in a bridge in North America to date is 14,700 psi (101 MPa) used for the North Concho River Overpass in San Angelo, TX (Ralls 1998). Specified design strengths at 56 days ranged from 5800 to 14,700 psi (40 to 101 MPa), and specified strengths at strand release ranged from 8900 to 10,800 psi (61 to 74 MPa) according to the demand of the specific span. High-strength concrete was required to achieve a span length of 157 ft (47.9 m), with a simple span 54 in. (1.37 m) deep beam. A combination of straight pretensioned strands and draped post-tensioned strands was used to achieve the required prestress force.

The largest application of HSC in a bridge in North America is the Confederation Bridge, which crosses the Northumberland Strait between Prince Edward Island and New Brunswick, Canada (Sauvageot 1995). Completed in 1997, the bridge is designed for a service life of 100 years. The main portion of this bridge consists of 43 spans that are 820 ft (250 m) long. Each span consists of two precast, prestressed concrete variable-depth cantilevered girders and a drop-in span. The drop-in span is made continuous with the cantilever girders in alternate spans. All of the substructure and superstructure components for the main spans were precast and floated out and erected using a large floating crane. The main girder was precast segmentally using a balanced cantilever approach. Total mass for the 625 ft (190 m) long girder is approximately 8820 tons (8200 metric tons). Because of the aggressive environment in the Northumberland Strait, the majority of concrete used for the superstructure has low permeability and a compressive strength of 8000 psi (55 MPa). For some piers, the ice shields use concrete with a compressive strength of 11,600 psi (80 MPa) to resist abrasion damage.

High-strength concrete has also been used in cable-stayed bridges such as the East Huntington Bridge, Annacis Bridge, Normandy Bridge, Pertuiset Bridge, and the Elorn Bridge. High-strength concrete provides an efficient material to resist the longitudinal compressive force in the superstructure and the vertical compressive force in the pylons.

## 9.4—Offshore structures

In 1984, the Glomar Beaufort Sea I was placed in the arctic. This exploratory drilling structure contains about 12,000 yd$^3$ (9200 m$^3$) of high-strength, lightweight concrete with density of approximately 112 lb/ft$^3$ (1794 kg/m$^3$), and 56-day compressive strength of 9000 psi (62 MPa). The structure also contains about 6500 yd$^3$ (5000 m$^3$) of high-strength, normalweight concrete with densities of approximately 145 lb/ft$^3$ (2323 kg/m$^3$) and 56-day compressive strengths of approximately 10,000 psi (69 MPa) (Fiorato et al. 1984).

High-strength concrete was used in the Heidrun floating, tension-leg oil production platform for the North Sea (CEB 1994). A concrete cube compressive strength of 8700 psi (60 MPa) and a fresh concrete density of 122 lb/ft$^3$ (1950 kg/m$^3$) were specified. The Heidrun platform was the world's first tension-leg platform with a concrete hull. High-strength concrete was selected to resist the water pressure in direct compression. Lightweight concrete was selected to reduce the mass of the platform.

High-strength concrete was also used in the Hibernia offshore concrete platform constructed in Newfoundland, Canada (Hoff et al. 1994; Hoff 1998). The 364 ft (111 m) tall concrete structure contains approximately 216,000 yd$^3$ (165,000 m$^3$) of pumped concrete. Specified concrete compressive strengths were 7100 and 10,000 psi (49 and 69 MPa). Two types of concrete were used in the structures—a normal-density concrete and a modified-density concrete. The latter concrete had 42 to 45% by volume of the normalweight coarse aggregate replaced with structural lightweight aggregate. This reduced the hardened density by 6 to 7%. The Hibernia platform is the largest single use of HSC in North America, and also set a record for offshore concrete structures.

Other applications in offshore structures are reported by FIP-CEB (1990).

## 9.5—Other applications

In 1948, concrete with a specified compressive strength of 9000 psi (62 MPa) was used in precast panels for a powerhouse at Fort Peck Dam, MT (*Concrete* 1949). High-strength concrete was specified to provide a dense concrete that could withstand the harsh exposure. Actual compressive strengths of concrete were reported to be considerably higher than 9000 psi (62 MPa).

Anderson (1960) reports the use of HSC in piles for marine foundations in the northwestern U.S. 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 in dock structures in the northwestern U.S.

Skrastins (1970) describes using 10,000 psi (69 MPa) concrete for prestressed concrete poles produced by spinning. High-strength concrete was selected to reduce the size of the poles. CEB (1994) reports applications of 14,500 psi (100 MPa) compressive strength concrete in prestressed transmission poles.

Copen (1975) indicates that the use of 10,000 psi (69 MPa) concrete in thin-arch dams would usually result in greater economy by reducing the volume of concrete. High-strength concrete would tend to reduce deflections in a dam, and may improve the strength of construction joints and allow earlier removal of formwork. Disadvantages of HSC listed by Copen include development of stress concentrations, particularly in the foundation for the dam; tendency toward more cracking; increased temperature control problems; and complications involved with openings through the dam and railways over the dam.

Bobrowski and Bardhan-Roy (1971) describe the application of HSC in two grandstand roofs. Lightweight concrete with a density of 118 lb/ft$^3$ (1890 kg/m$^3$) and 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/ft$^3$ (1840 kg/m$^3$).

Wolsiefer (1984) reports on field placements of high-strength, low-permeability, and chemical-resistant concretes for industrial manufacturing applications.

Other special applications include 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. Munn (1998) described the use of HSC in security vaults.

In Australia, HSCs have been used in industrial facilities and pavements to provide greater abrasion resistance and increased flexural strength. Munn (1998) describes several of these applications. In Norway, HSC has also been used in pavements for abrasion resistance (FIB-CEB 1990; CEB 1994).

Another special column application was in the Reliant Stadium, Houston, TX. The entire retractable roof structure of this building is supported on four massive super columns. Each column is 153 ft (46.6 m) tall and contains 13,000 psi (90 MPa) compressive strength concrete (*Structure Magazine* 2003).

## CHAPTER 10—SUMMARY

This report presented state-of-the-art information on concrete with strengths in excess of 8000 psi (55 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 HSC should be carefully selected using all available techniques to ensure uniform success. Items to be considered in selecting materials include characteristics of cement and SCMs, aggregate size, aggregate strength, particle shape and texture, and the effects of chemical admixtures, particularly with respect to their water reduction and hydration controlling capabilities. Trial mixtures, both in the laboratory and field, are essential to ensure that required concrete strengths and other desired properties will be obtained and that all constituent materials are compatible.

Mixture proportions for HSC have generally 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 mixture proportions have included availability of constituent materials, desired workability, and effects of temperature rise. All materials should be optimized in concrete mixture proportioning to achieve maximum potential strength. High-strength concrete mixtures have usually used high cementitious materials contents, low *w/cm*, normal-density aggregate, chemical admixtures, and SCMs, including pozzolans and slag. Required strength, specified age, material characteristics, and type of application have strongly influenced mixture proportioning. High-strength concrete mixture proportioning has been found to be a more critical process than the proportioning of lower-strength concrete mixtures. Laboratory trial batches have been required to generate necessary data on mixture proportioning. In many cases, laboratory mixtures have been followed by field production trial batches.

Batching, mixing, transporting, placing, and control procedures for HSC are not essentially different from procedures used for lower-strength concretes. Special attention, however, is required to ensure 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. Delays 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 HSC cylinders because any deficiency will result in an apparent lower strength than that actually achieved by the concrete. Items deserving specific attention include manufacturing, 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.

Research data have indicated that the measured modulus of elasticity of HSC can vary significantly from calculated values based on unit weight and concrete compressive strength. Values of Poisson's ratio, however, appear to be in the expected range, based on lower-strength concretes. The modulus of rupture for HSCs is higher than would have been anticipated based on equations used to predict modulus of rupture for lower-strength concrete. The tensile splitting strength values, however, appear to be consistent with lower-strength concretes. Unit weight is slightly higher with HSC. 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 with lower-strength concrete, but at later ages, the difference is not as significant. Information on creep and shrinkage of HSC has indicated that the shrinkage is similar to that for lower-strength concrete. Specific creep, however, is much less for HSCs than for lower-strength concretes.

The use of HSC can have significant impacts on structural design, though changes in structural behavior generally occur gradually as concrete strength is increased. Modifications to standard design equations developed for lower-strength concretes are necessary for determining the strength of axially loaded columns, axial and flexural strength of eccentrically-loaded columns, loss of prestress in prestressed concrete beams, and minimum reinforcement requirements for flexure, shear, and torsion in reinforced concrete beams. Proposed modifications in each of these areas have been summarized in this document. Significant research has been completed, but consensus design equations have not yet been established for development length of bars in reinforced concrete, and for transfer and development length of prestressing strand. It has been established that the use of the ACI 318 equivalent rectangular stress block, without modification, is acceptable for under-reinforced HSC beams. Additionally, it is noted that long-term deformations are generally smaller and that confinement is generally less effective in HSC beams and columns.

The economic advantages of using HSC 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 8000 psi (55 MPa) has been used in the columns of high-rise buildings in cities throughout the world. Studies have also indicated that the use of HSC in prestressed concrete bridges can result in longer span lengths, wider girder spacing, or shallower sections. There have also been applications where high-compressive-strength concrete has been needed to satisfy special local requirements. These have included dams, prestressed concrete poles, grandstand roofs, marine foundations, offshore structures, parking garages, bridge deck overlays, heavy duty industrial floors, and industrial manufacturing applications.

Although HSC was once considered a relatively new material, it has become accepted in many parts of North America and throughout the world, as shown by the many examples of its usage. At the same time, material producers have responded to the demands for the material, and are learning new production techniques. As with many developments of new materials, research data supporting the growth have also increased. Some research projects are underway to satisfy the remaining needs. Further work, however, is still needed to fully utilize HSC and to affirm its capabilities. This report has documented existing knowledge of HSC so that the direction for future development may be ascertained.

# CHAPTER 11—REFERENCES

## 11.1—Referenced standards and reports

The standards and reports listed below were the latest editions at the time this document was prepared. Because these documents are revised frequently, the reader is advised to contact the proper sponsoring group if it is desired to refer to the latest version.

*American Association of State Highway and Transportation Officials (AASHTO)*

AASHTO LRFD Bridge Design Specifications

T277-07 Standard Test Method of Test for Rapid Determination of the Chloride Permeability of Concrete

*American Concrete Institute (ACI)*

209R Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures

210R Erosion of Concrete in Hydraulic Structures

211.1 Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete

211.4R Guide for Selecting Proportions for High-Strength Concrete Using Portland Cement and Other Cementitious Materials

212.3R Chemical Admixtures for Concrete

212.4R Guide for the Use of High-Range Water-Reducing Admixtures (Superplasticizers) in Concrete

214R Evaluation of Strength Test Results of Concrete

216.1 Code Requirements for Determining Fire Resistance of Concrete and Masonry Construction Assemblies

232.2R Use of Fly Ash in Concrete

233R Slag Cement in Concrete and Mortar

234R Guide for the Use of Silica Fume in Concrete

301/301M Specifications for Structural Concrete

304R Guide for Measuring, Mixing, Transporting, and Placing Concrete

305R Hot Weather Concreting

308R Guide to Curing Concrete

309R Guide for Consolidation of Concrete

318/318M Building Code Requirements for Structural Concrete and Commentary

363.2R Guide to Quality Control and Testing of High-Strength Concrete

435R Control of Deflection in Concrete Structures

441R Report on High-Strength Concrete Columns

*ASTM International*

C29/C29M Standard Test Method for Bulk Density (“Unit Weight”) and Voids in Aggregate

C31/C31M Standard Practice for Making and Curing Concrete Test Specimens in the Field

C33/33M Standard Specification for Concrete Aggregates

C94/C94M Standard Specification for Ready-Mixed Concrete

C109/C109M Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or [50-mm] Cube Specimens)

C136 Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates

C143/C143M Standard Test Method for Slump of Hydraulic-Cement Concrete

C150/C150M Standard Specification for Portland Cement

C172 Standard Practice for Sampling Freshly Mixed Concrete

C191 Standard Test Methods for Time of Setting of Hydraulic Cement by Vicat Needle

C192/192M Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory

C260 Standard Specification for Air-Entraining Admixtures for Concrete

C311 Standard Test Methods for Sampling and Testing Fly Ash or Natural Pozzolans for Use in Portland-Cement Concrete

C494/C494M Standard Specification for Chemical Admixtures for Concrete

C618 Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete

C666/C666M Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing

C672/C672M Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals

C684 Standard Test Method for Making, Accelerated Curing, and Testing Concrete Compression Test Specimens

| | |
|---|---|
| C917 | Standard Test Method for Evaluation of Cement Strength Uniformity From a Single Source |
| C918/C918M | Standard Test Method for Measuring Early-Age Compressive Strength and Projecting Later-Age Strength |
| C989 | Standard Specification for Slag Cement for Use in Concrete and Mortars |
| C1202 | Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration |
| C1240 | Standard Specification for Silica Fume Used in Cementitious Mixtures |
| C1602/C1602M | Standard Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete |

*Canadian Standards Association (CSA)*

CSA A23.3 Design of Concrete Structures

*Concrete Industry Eurocode 2 Group (CIEG)*

EN 13263 Silica Fume for Concrete

*Standards Association of New Zealand*

NZS 3101 Concrete Structures Standard

The above publications may be obtained from the following organizations:

American Association of State Highway and Transportation Officials
444 N. Capitol Street NW, Suite 249
Washington, DC 20001
www.transportation.org

American Concrete Institute
38800 Country Club Drive
Farmington Hills, MI 48331
www.concrete.org

ASTM International
100 Barr Harbor Drive
West Conshohocken, PA 19428
www.astm.org

Canadian Standards Association
5060 Spectrum Way
Mississauga, ON
L4W 5N6 Canada
www.csa.ca

Concrete Industry Eurocode 2 Group (CIEG)
The Concrete Centre, Riverside House
4 Meadows Business Park, Station Approach
Blackwater, Camberley
Surrey GU17 9AB
www.eurocode2.info

Standards Association of New Zealand
Standards New Zealand
Radio New Zealand House
Level 10, 155 The Terrace
Wellington 6011
http://www.standards.co.nz/

## 11.2—Cited references

AASHTO, 2004, "AASHTO LRFD Bridge Design Specifications," third edition, American Association of State Highway and Transportation Officials, Washington, DC, pp. 5-12 to 5-19.

Abdel-Fattah, H., and Ahmad, S. H., 1989, "Behavior of Hoop Confined High-Strength Concrete Under Axial and Shear Loads," *ACI Structural Journal*, V. 86, No. 6, Nov.-Dec., pp. 652-659.

Abrishami H. H., and Mitchell, D., 1993, "Bond Characteristics of Pretensioned Strand," *ACI Materials Journal*, V. 90, No. 3, May-June, pp. 228-235.

ACI Committee 105, 1933, "Reinforced Concrete Column Investigation—Tentative Final Report of Committee 105," ACI JOURNAL, *Proceedings* V. 29, No. 5, pp. 275-282.

ACI Committee 318, 1999, "Building Code Requirements for Structural Concrete (ACI 318-99) and Commentary (318R-99)," American Concrete Institute, Farmington Hills, MI, 391 pp.

ACI Committee 318, 2005, "Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary," American Concrete Institute, Farmington Hills, MI, 430 pp.

Aggoun, S.; Kheirbek, A.; Kadri, E. H.; and Duval, R., 2002, "Study of the Flow of Self-Compacting Concretes," First North American Conference on the Design and Use of Self-Consolidating Concrete, Nov. 12-13, pp. 259-265.

Ahlborn, T. M.; French, C. E.; and Leon, R. T., 1995, "Application of High Strength Concrete to Long-Span Prestressed Bridge Girders," *Transportation Research Record* No. 1476, Transportation Research Board, Washington, DC, pp. 22-30.

Ahmad, S. H., 1981, "Properties of Confined Concrete Subject to Static and Dynamic Loading," PhD thesis, University of Illinois at Chicago, Chicago, IL, p. 375.

Ahmad, S. H., and Shah, S. P., 1982a, "Stress-Strain Curves of Concrete Confined by Spiral Reinforcement," ACI JOURNAL, *Proceedings* V. 79, No. 6, June, pp. 484-490.

Ahmad, S., and Shah, S. P., 1982b, "Complete Triaxial Stress-Strain Curves for Concrete," *Proceedings*, ASCE, V. 108, No. ST4, Apr., pp. 728-742.

Ahmad, S. H., and Shah, S. P., 1985, "Structural Properties of High Strength Concrete and Its Implications on Precast and Prestressed Concrete," *PCI Journal*, Nov.-Dec., pp. 91-119.

Ahmad, S. H., and Zia, P., 1997, "High Early Strength Concrete for Prestressed Concrete Applications," *Proceedings of the PCI/FHWA International Symposium on High Performance Concrete*, New Orleans, LA, Oct. 20-22, pp. 194-205.

Aïtcin, P. C., and Neville, A., 1993, "High Performance Concrete Demystified," *Concrete International*, V. 15, No. 1, Jan., pp. 21-26.

Alca, N.; Alexander, S. D. B.; and MacGregor, J. G., 1997, "Effect of Size on Flexural Behavior of High-Strength Concrete Beams," *ACI Structural Journal*, V. 94, No. 1, Jan.-Feb., pp. 59-67.

Almeida. I. R., 1994, "Abrasion Resistance of High Strength Concrete with Chemical and Mineral Admixtures," *Durability of Concrete*, Proceedings of the Third CANMET/ACI International Conference, SP-145, V. M. Malhotra, ed., American Concrete Institute, Farmington Hills, MI, pp. 1099-1113.

American Concrete Institute, 2009, "ACI Concrete Terminology," American Concrete Institute, Farmington Hills, MI, http://terminology.concrete.org (accessed Sept. 9, 2009).

Anderson, A. R., 1960, "Research Answers Needed for Greater Utilization of High Strength Concrete," *PCI Journal*, V. 25, No. 4, July-Aug., pp. 162-164.

Angelakos, D.; Bentz, E. C.; and Collins, M. P., 2001, "Effect of Concrete Strength and Minimum Stirrups on Shear Strength of Large Members," *ACI Structural Journal*, V. 98, No. 3, May-June, pp. 290-300.

*Architectural Record*, 1976, "High-Strength Concrete Allows Bigger Loads on Smaller Columns," V. 159, No. 7, June, pp. 133-135.

Attard, M. M., and Stewart, M. G., 1998, "A Two Parameter Stress Block for High-Strength Concrete," *ACI Structural Journal*, V. 95, No. 3, May-June, pp. 305-317.

Azizinamini, A.; Kuska, S.; Brungardt, P.; and Hatfield, E., 1994, "Seismic Behavior of Square High-Strength Concrete Columns," *ACI Structural Journal*, V. 91, No. 3, May-June, pp. 336-345.

Azizinamini, A.; Darwin, D.; Eligehausen, R.; Pavel, R.; and Ghosh, S. K., 1999a, "Proposed Modification to ACI 318-95 Tension Development and Lap Slice for High-Strength Concrete," *ACI Structural Journal*, V. 96, No. 6, Nov.-Dec., pp. 922-926.

Azizinamini, A.; Pavel, R.; Hatfield, E.; and Ghosh, S. K., 1999b, "Behavior of Lap-Spliced Bars Embedded in High-Strength Concrete," *ACI Structural Journal*, V. 96, No. 5, Sept.-Oct., pp. 826-835.

Barnes, R. W., and Burns, N. H., 2000, "Anchorage Behavior of 15.2 mm (0.6 in.) Prestressing Strand in High Strength Concrete," *Symposium Proceedings*, PCI/FHWA/FIB International Symposium on High Performance Concrete, Orlando, FL, Precast/Prestressed Concrete Institute, Chicago, IL, pp. 484-493.

BASF, 2008, "Viscosity-Modifying Admixtures," BASF Web site reference: http://www.basf-admixtures.com/en/products/viscositymodifying/.

Bennett, E. W., and Muir, S. E., 1967, "Some Fatigue Tests on High Strength Concrete in Uniaxial Compression," *Magazine of Concrete Research* (London), V. 19, No. 59, pp. 113-117.

Berke, N. S.; Dallaire, M. P.; and Hicks, M. C., 1992, "Plastic, Mechanical, Corrosion, and Chemical Resistance Properties of Silica Fume (Microsilica) Concretes," *Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete*, Proceedings of the Fourth ACI/CANMET International Conference, SP-132, V. M. Malhotra, ed., American Concrete Institute, Farmington Hills, MI, pp. 1125-1149.

Bickley, J. A., 1999, "North American Trends in the Development of High-Strength Concrete," *Symposium Proceedings*, Fifth International Symposium on the Utilization of High-Strength/High Performance Concrete, V. 1, Sandefjord, Norway, pp. 1-13.

Bickley, J. A., and Payne, J. C., 1979, "High Strength Cast-in-Place Concrete in Major Structures in Ontario," paper presented at ACI Convention, Milwaukee, WI, Mar.

Bing, L.; Park, R.; and Tanaka, H., 2000, "Constitutive Behavior of High-Strength Concrete under Dynamic Loads," *ACI Structural Journal*, V. 97, No. 4, July-Aug., pp. 619-629.

Bjerkeli, L.; Tomaszewica, A.; and Jensen, J. J., 1990, "Deformation Properties and Ductility of High-Strength Concrete," *High-Strength Concrete*, Second International Symposium, SP-121, American Concrete Institute, Farmington Hills, MI, pp. 215-238.

Blick, R. L., 1973, "Some Factors Influencing High-Strength Concrete," *Modern Concrete*, V. 36, No. 12, Apr., pp. 38-41.

Bloom, R., and Bentur, A., 1995, "Free and Restrained Shrinkage of Normal and High-Strength Concrete," *ACI Materials Journal*, V. 92, No. 2, Mar.-Apr., pp. 211-217.

Bobrowski, J., and Bardhan-Roy, B. K., 1971, "Structural Assessment of Lightweight Aggregate Concrete," *Concrete* (London), V. 5, No. 7, July, pp. 229-234.

Brewe, J., and Myers, J. J., 2005, "Particle Size Optimization for Reduced Cement Content High Strength Concretes," *Proceedings for the PCI-NBC on Bridges for Life 2005*, Palm Springs, CA, Oct. 16-19, 14 pp. (CD-ROM)

Bruce, R. N.; Russell, H. G.; Roller, J. J.; and Martin, B. T., 1994, "Feasibility Evaluation of Utilizing High-Strength Concrete in Design and Construction of Highway Bridge Structures," *Research Report* No. FHWA/LA-94-282, Louisiana Transportation Research Center, Baton Rouge, LA, 168 pp.

Buckner, C. D., 1995, "A Review of Strand Development Length for Pretensioned Concrete Members," *PCI Journal*, V. 40, No. 2, Mar.-Apr., pp. 84-105.

Burg, R. G., and Ost, B. W., 1994, "Engineering Properties of Commercially Available High-Strength Concretes (Including Three-Year Data)," *Research and Development Bulletin RD104*, Portland Cement Association, Skokie, IL, 58 pp.

Burg, R. G.; Caldarone, M. A.; Detwiler, G.; Jansen, D. C.; and Willems, T. J., 1999, "Compression Testing of HSC: Latest Technology," *Concrete International*, V. 21, No. 8, Aug., pp. 67-76.

Burgess. A. J.; Ryell, J.; and Bunting, J., 1970, "High Strength Concrete for the Willows Bridge," ACI JOURNAL, *Proceedings* V. 67, No. 8, Aug., pp. 611-619.

Burns, N. H.; Gross, S. P.; and Byle, K. A., 1997, "Instrumentation and Measurements—Behavior of Long-Span Prestressed High Performance Concrete Bridges," *Proceedings of the PCI/FHWA International Symposium on High Performance Concrete*, Precast/Prestressed Concrete Institute (PCI), New Orleans, LA, pp. 566-577.

Caldarone, M. A., 2008, *High-Strength Concrete: A Practical Guide*, Routledge, London, pp. 34-35.

Caldarone, M. A.; Gruber, K. A.; and Burg, R. G., 1994, "HRM: A New Generation Mineral Admixture," *Concrete International*, V. 16, No. 11, Nov., pp. 37-40.

Carino, N. J.; Guthrie, W. F.; and Lagergren, E. S., 1994, "Effects of Testing Variables on the Measured Compressive Strength of High-Strength Concrete," *NIST Report* No. NISTIR 5405 141 pp.

Carrasquillo, P. M., 1994, "Concrete Strength Testing," *Significance of Tests and Properties of Concrete and Concrete-Making Materials*, STP-169C, ASTM International, West Conshohocken, PA, pp. 123-139.

Carrasquillo, R. L.; Slate, F. O.; and Nilson, A. H., 1981, "Microcracking and Behavior of High Strength Concrete Subject to Short-Term Loading," ACI JOURNAL, *Proceedings* V. 78, No. 3, Mar., pp. 179-186.

Carrasquillo, R. L.; Nilson, A. H.; and Slate, F. O., 1982, "Properties of High Strength Concrete Subjected to Short-Term Loads," ACI JOURNAL, *Proceedings* V. 78, No. 3, May-June 1981, pp. 171-178, and discussion, V. 79, No. 2, Mar.-Apr., pp. 162-163.

Castrodale, R. W.; Kreger, M. E.; and Burns, N. H., 1988, "A Study of Pretensioned High-Strength Concrete Girders in Composite Highway Bridges—Design Considerations," *Research Report* 381-4F, Center for Transportation Research, University of Texas at Austin, Austin, TX, 306 pp.

CEB (Comite Euro-International du Beton), 1991, "CEB-FIP Model Code 1990," Final Draft, Lausanne, Switzerland.

CEB (Comite Euro-International du Beton), 1994, "Application of High Performance Concrete" *Bulletin d'Information* No. 222, Nov., Lausanne, Switzerland, 66 pp.

Cetin, A., and Carrasquillo, R. L., 1998, "High Performance Concrete: Influence of Coarse Aggregates on Mechanical Properties," *ACI Materials Journal*, V. 95, No. 3, May-June, pp. 252-261.

CCHRB (Chicago Committee on High-Rise Buildings), 1977, "High-Strength Concrete in Chicago High-Rise Buildings," *Task Force Report* No. 5, Feb., Chicago, IL, 63 pp.

Cohen, M.; Zhou, Y.; and Dolch, W., 1992, "Non-Air-Entrained High Strength Concrete—Is It Frost Resistant?" *ACI Materials Journal*, V. 89, No. 4, July-Aug., pp. 406-415.

Collins, M. P., and Kuchma, D., 1999, "How Safe Are Our Large, Lightly Reinforced Concrete Beams, Slabs, and Footings?" *ACI Structural Journal*, V. 96, No. 4, July-Aug., pp. 282-290.

Collins, M. P.; Mitchell, D.; and MacGregor, J. G., 1993, "Structural Design Considerations for High-Strength Concrete," *Concrete International*, V. 15, No. 5, May, pp. 27-34.

Cook, J. E., 1982, "Check and Application of High-Strength Concrete Using Class C Fly Ash," *Concrete International*, V. 4, No. 7, July, pp. 72-80.

Cook, J. E., 1989, "Research and Application of High-Strength Concrete: 10,000 psi Concrete," *Concrete International*, V. 11, No. 10, Oct., pp. 67-75.

*Concrete*, 1949, "Unusual Strengths Attained in Precast Slabs Used for Facing Power House Walls," V. 57, No. 5, May 10, pp. 9-10.

Copen, M. D., 1975, "Problems Attending Use of Higher Strength Concrete in Thin Arch Dams," ACI JOURNAL, *Proceedings* V. 72, No. 4, Apr., pp. 138-140.

Cordon, W. A., and Gillespie, A. H., 1963, "Variables in Concrete Aggregates and Portland Cement Paste which influence the Strength of Concrete," ACI JOURNAL, *Proceedings* V. 60, No. 8, pp. 1028-1050; and discussion, Mar. 1964, pp. 1981-1998.

Coulombe, L. G., and Ouellet, C., 1995, "Construction of Two Experimental Bridges Using High-Performance Air-Entrained Concrete," Paper 95-1060, Transportation Research Board, Washington, DC, Jan., 26 pp.

Cousins, T. E.; Johnston, D. W.; and Zia, P., 1990, "Transfer and Development Length of Epoxy Coated and Uncoated Prestressing Strand," *PCI Journal*, V. 35, No. 4, July-Aug., pp. 92-103.

Cusson, D., and Paultre, P., 1994, "High-Strength Concrete Columns Confined by Rectangular Ties," *Journal of Structural Engineering*, ASCE, V. 120, No. 3, pp. 783-804.

deLarrard, F., and Belloc, A., 1997, "The Influence of Aggregate on the Compressive Strength of Normal and High Strength Concrete," *ACI Materials Journal*, V. 94, No. 5, Sept.-Oct., pp. 417-426.

Dewar, J. D., 1964, "The Indirect Tensile Strength of Concretes of High Compressive Strength," *Technical Report* No. 42.377, Cement and Concrete Association, Wexham Springs, Mar., 12 pp.

Ekenel, M.; Myers, J. J.; and Sides, B., 2004, "Optimization & Durability of High Strength/High Performance Concrete Produced Using Locally Available Materials in The State of Missouri," *Report* No. 02-33, Center for Infrastructure Engineering Studies, Missouri S&T, Rolla, MO, Feb., 124 pp.

ELKEM—SPIGERVERKET a/s, 1980, *ELKEM SILICA Technical Bulletin*, Fiskaa Verk, Vaayobygd, Norway.

*Engineering News-Record*, 1981, "Concrete Beats Steel by 29%," V. 206, May 14, p. 16.

Ernzen, J. J., and Carrasquillo, R. L., 1992, "Resistance of High Strength Concrete to Cold Weather Environments," *Research Report 481-7*, Center for Transportation Research, University of Texas at Austin, Austin, TX.

Esfahani, M. R., and Rangan, B. V., 1998a, "Local Bond Strength of Reinforcing Bars in Normal Strength and High-Strength Concrete (HSC)," *ACI Structural Journal*, V. 95, No. 2, Mar.-Apr., pp. 96-106.

Esfahani, M. R., and Rangan, B. V., 1998b, "Bond Between Normal Strength and High-Strength Concrete (HSC) and Reinforcing Bars in Splices in Beams," *ACI Structural Journal*, V. 95, No. 3, May-June, pp. 272-279.

Fafitis, A., and Shah, S. P., 1985, "Lateral Reinforcement for High-Strength Concrete Columns," *High-Strength Concrete*, SP-87, American Concrete Institute, Farmington Hills, MI, pp. 213-232.

Fagerlund, G., 1975, "Studies of the Destruction Mechanisms at Freezing of Porous Materials," *Proceedings of the Sixth International Congress on the Problems Raised by Frost Action*, LeHarve, France, pp. 166-196.

Fagerlund, G., 1994, "Frost Resistance of High Performance Concrete—Some Theoretical Considerations, Durability of

High Performance Concrete," *Proceedings of the RILEM International Workshop*, Feb., pp. 112-140.

Fang, I. K.; Wang, C. S.; and Hong, K. L., 1994, "Cyclic Behavior of High-Strength Concrete Short Beams with Lower Amount of Flexural Reinforcement," *ACI Structural Journal*, V. 91, No. 1, Jan.-Feb., pp. 10-18.

Farny, J. A., and Panarese, W. C., 1994, "High-Strength Concrete," *PCA Engineering Bulletin* No. 114, Portland Cement Association, Skokie, IL, 14 pp.

Feldman, R. F., 1981, "Pore Structure Formation During Hydration of Fly Ash and Slag Cement Blends," *Effects of Fly Ash Incorporation in Cement and Concrete*, Materials Research Society, Pittsburgh, PA, pp. 24-133.

Fentress, B., 1973, "Slab Construction Practice Compared Wear Tests," ACI JOURNAL, *Proceedings* V. 70, No. 7, July, pp. 486-491.

FHWA, 1996, "Study Tour of Northumberland Strait Crossing Project (NSCP)," Report Facilitator: H. G. Russell, *Report* No. FHWA-PL-96-022, Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 70 pp.

FHWA, 2003, "High Performance Concrete," Version 3.0, Federal Highway Administration, U.S. Department of Transportation, Washington, DC, Feb. (CD-ROM)

Fiorato, A. E., 1989, "PCA Research on High-Strength Concrete," *Research and Development Bulletin RD093.01T*, Portland Cement Association, Skokie, IL, 7 pp.

Fiorato, A. E.; Person, A.; and Pfeifer, D. W., 1984, "The First Large-Scale Use of High Strength Lightweight Concrete in the Arctic Environment," *Second Symposium on Arctic Offshore Drilling Platforms*, Apr., Houston, TX.

FIP-CEB, 1990, "High Strength Concrete: State-of-the-Art Report," *Bulletin d'Information* No. 197, Aug., Lausanne, Switzerland, 91 pp.

Fletcher, J., 1997, "Project Outline and Economic Analysis: West River East Side Bridge, Pictou County, Nova Scotia," *Proceedings*, Concrete Canada Technology Transfer Day, Toronto, ON, Canada, pp. 49-56.

Freedman, S., 1971, "High-Strength Concrete," *Modern Concrete*, V. 34, No. 6, Oct. 1970, pp. 29-36; No. 7, Nov. 1970, pp. 28-32; No. 8, Dec. 1970, pp. 21-24; No. 9, Jan. 1971, pp. 15-22; and No. 10, Feb., pp. 16-23.

Freyne, S. F., 2000, "High Performance Concrete in Oklahoma Designed for Precast/Prestressed Bridge Beams," dissertation, University of Oklahoma, Norman, OK.

Freyne, S. F.; Russell, B. W.; Bush, T. D.; and Hale, W. M., 2004, "Comparing Different Cements in High Performance Concrete," *ACI Materials Journal*, V. 101, No. 6, Nov.-Dec., pp. 436-441.

Gagne, R.; Pigeon, M.; and Aïtcin, P. C., 1991, "Deicer Salt Scaling Resistance of High-Strength Concretes Made with Different Cements," *Durability of Concrete*, SP-126, V. M. Malhotra, ed., American Concrete Institute, Farmington Hills, MI, pp. 185-199.

Gaynor, R. D., 1968, "High Strength Air-Entrained Concrete," *Joint Research Laboratory Publication* No. 17, National Sand and Gravel Association/National Ready Mixed Concrete Association, Silver Spring, MD, 19 pp.

Gaynor, R. D., 1980, "Mix Design Submission Under ACI 318 and ACI 301—(or Which Test Record Should I Use?)," *NRMCA Technical Information Letter* No. 372, National Ready Mixed Concrete Association, Silver Spring, MD, May 8, 7 pp.

Gaynor, R. D., and Meininger, R. C., 1983, "Evaluating Concrete Sands: Five Tests to Estimate Quality," *Concrete International*, V. 5, No. 12, Dec., pp. 53-60.

Gross, S. P., 1998, "Field Performance of Prestressed High Performance Concrete Highway Bridges in Texas," PhD dissertation, University of Texas at Austin, Austin, TX, pp. 127-132, 134-139, 152-164, 167-170, 330-332.

Gross, S. P., 2000, "Camber of High Strength Concrete Bridge Girders," *HPC Bridge Views* No. 8, National Concrete Bridge Council, Mar.-Apr., pp. 3-4.

Gross, S. P., and Burns, N. H., 1995, "Transfer and Development Length of 15.2 mm (0.6 in.) Diameter Prestressing Strand in High Performance Concrete: Results of the Hoblitzell-Buckner Beam Tests," *Research Report 580*-2, Center for Transportation Research, University of Texas at Austin, Austin, TX, June, 106 pp.

Gross, S. P., and Burns, N. H., 1999, "Field Performance of Prestressed High-performance Concrete Highway Bridges in Texas," *Preliminary Research Report 580/589-3*, Center for Transportation Research, University of Texas at Austin, Austin, TX, 691 pp.

Gross, S. P., and Burns, N. H., 2000, "Implementation of High Strength-High Performance Concrete in Two Texas Highway Overpass Structures," *Transportation Research Record* No. 1692, V. 2, Washington, DC, pp. 179-187.

Gustaferro, A.; Hillier, M. A.; and Janney, J. R., 1983, "Performance of Prestressed Concrete on the Illinois Tollway After 25 Years of Service," *PCI Journal*, V. 28, No. 1, pp. 50-67.

Hadchiti, K. M., and Carrasquillo, R. L., 1988, "Abrasion Resistance and Scaling Resistance of Concrete Containing Fly Ash," *Research Report 481-3*, Center for Transportation Research, University of Texas at Austin, Austin, TX, Aug., 206 pp.

Hale, W. M., and Russell, B., 2000, "The Need for Air Entrainment in High Performance Concrete," *PCI/FHWA/FIB International Symposium on High Performance Concrete*, L. S. Johal, ed., Sept., pp. 206-214.

Haque, M. N.; Kayyali, O. A.; and Gopalan, M. K., 1992, "Fly Ash Reduces Harmful Chloride Ions in Concrete," *ACI Materials Journal*, V. 89, No. 3, May-June, pp. 238-241.

Harris, A. J., 1969, "High-Strength Concrete: Manufacture and Properties," *The Structural Engineer* (London), V. 47, No. 11, pp. 441-446.

Hester, W. T., 1977, "High-Strength Air-Entrained Concrete," *Concrete Construction*, V. 22, No. 2, Feb., pp. 77-82.

Hester, W. T., 1980, "Field Testing High-Strength Concretes: A Critical Review of the State-of-the-Art," *Concrete International*, V. 2, No. 12, Dec., pp. 27-38.

Hester, W. T., and Leming, M., 1989, "High Strength Concretes: The Two Edged Sword," *NRMCA Publication* No. 176, National Ready Mixed Concrete Association, Silver Spring, MD.

Hoff, G. C., 1992, "High Strength Lightweight Aggregate Concrete for Arctic Applications—Parts 1, 2 and 3," *Structural Lightweight Aggregate Concrete Performance*, SP-136, T. A. Holm and A. M. Vaysburd, eds., American Concrete Institute, Farmington Hills, MI., pp. 1-245.

Hoff, G. C., 1998, "The Hibernia Offshore Platform—A Major Application of High Performance Concrete," *Proceedings of the International Conference on High-Performance High-Strength Concrete*, Perth, Australia, Aug., pp. 51-74.

Hoff, G. C.; Walum, R.; Elimov, R.; and Woodhead, H. R., 1994, "Production of High-Strength Concrete for the Hibernia Offshore Concrete Platform," *High-Performance Concrete*, Proceedings of the ACI International Conference, SP-149, V. M. Malhotra, ed., American Concrete Institute, Farmington Hills, MI, pp. 37-62.

Holland, T., 2005, *Silica Fume User's Manual*, Silica Fume Association, Lovettsville, VA, 193 pp.

Holm, T. A., 1980, "Physical Properties of High Strength Lightweight Aggregate Concretes," *Proceedings*, 2nd International Congress on Lightweight Concrete, Ci8O, Construction Press, Lancaster, pp. 187-204.

Hooton, R. D., 1986, "Permeability and Pore Structure of Cement Based Pastes Containing Fly-Ash, Slag and Silica Fume," *Blended Cements*, ASTM STP-897, G. Frohnsdorff, ed., ASTM, West Conshohocken, PA, pp. 128-143.

Hooton, R. D.; Pun, P.; Kojundic, T.; and Fidjestøl, P., 1997, "Influence of Silica Fume on Chloride Resistance of Concrete," *Proceedings of PCI/FHWA International Symposium on High Performance Concrete*, New Orleans, LA, pp. 245-256.

Huo, X., and Tadros, M. K., 2000, "Structural Design of High-Performance Concrete Bridge Beams," *Transportation Research Record* No. 1692, V. 2, Transportation Research Board, Washington, DC, pp. 171-178.

Hwang, S. J.; Leu, Y. R.; and Hwang, H. L., 1996, "Tensile Bond Strengths of Deformed Bars of High-Strength Concrete," *ACI Structural Journal*, V. 93, No. 1, Jan.-Feb., pp. 11-20.

Ibrahim, H. H. H., and MacGregor, J. G., 1996, "Flexural Behavior of Laterally Reinforced High Strength Concrete Sections," *ACI Structural Journal*, V. 93, No. 6, Nov.-Dec., pp. 674-684.

Ibrahim, H. H. H., and MacGregor, J. G., 1997, "Modification of the ACI Rectangular Stress Block for High-Strength Concrete," *ACI Structural Journal*, V. 94, No. 1, Jan.-Feb., pp. 40-48.

Iravani, S., 1996, "Mechanical Properties of High-Performance Concrete," *ACI Materials Journal*, V. 93, No. 5, Sept.-Oct., pp. 416-426.

Iravani, S., and MacGregor, J. G., 1998, "Sustained Load Strength and Short-Term Strain Behavior of High-Strength Concrete," *ACI Materials Journal*, V. 95, No. 5, Sept.-Oct., pp. 636-647.

Jansen, D. C.; Shah, S. P.; and Rossow, E. C., 1995, "Stress Strain Results of Concrete from Circumferential Strain Feedback Control Testing," *ACI Materials Journal*, V. 92, No. 4, July-Aug., pp. 419-428.

Johnson, M. K., and Ramirez, J. A., 1989, "Minimum Shear Reinforcement in Beams with Higher Strength Concrete," *ACI Structural Journal*, V. 86, No. 4, July-Aug., pp. 376-382.

Kaar, P. H.; Hanson, N. W.; and Capell, H. T., 1978, "Stress-Strain Characteristics of High-Strength Concrete," *Douglas McHenry International Symposium on Concrete and Concrete Structures*, SP-55, American Concrete Institute, Farmington Hills, MI, pp. 161-185.

Kahn, L. F.; Dell, J. C.; and Reutlinger, C. G., 2002, "Transfer and Development Length of 15-mm Strand in High Performance Concrete Girders," *Journal of Structural Engineering*, V. 128, No. 7, pp. 913-921.

Kaplan, M. F., 1959a, "Flexural and Compressive Strength of Concrete as Affected by the Properties of the Coarse Aggregates," ACI JOURNAL, *Proceedings* V. 55, No. 11, May, pp. 1193-1208.

Kaplan, M. F., 1959b, "Ultrasonic Pulse Velocity, Dynamic Modulus of Elasticity, Poisson's Ratio and the Strength of Concrete Made with Thirteen Different Coarse Aggregates," *RILEM Bulletin* (Paris), New Series No. 1, Mar., pp. 58-73.

Klieger. P., 1957, "Early High Strength Concrete for Prestressing," *Proceedings*, World Conference on Prestressed Concrete, San Francisco, CA, pp. A5-1 to A5-14.

Kodur, V. K. R., 2000, "Spalling in High Strength Concrete Exposed to Fire—Concerns, Causes, Critical Parameters and Cures," *ASCE Structures Congress Proceedings*, May. (CD-ROM)

Kong, P. Y. L., and Rangan, B. V., 1998, "Shear Strength of High-Performance Concrete Beams," *ACI Structural Journal*, V. 95, No. 6, Nov.-Dec., pp. 677-688.

Kopczynski, C., 2008, "High-Strength Rebar Expanding Options in Concrete Towers," *Structure Magazine*, National Council of Engineers Association, Apr., pp 30-31.

Kosmatka, S. H.; Kerkhoff, B.; and Panarese, W. C., 2001, *Design and Control of Concrete Mixtures*, fourteenth edition, Portland Cement Association, Skokie, IL, 307 pp.

Koutchoukali, N. E., and Belarbi, A., 2001, "Torsion of High-Strength Reinforced Concrete Beams and Minimum Reinforcement Requirement," *ACI Structural Journal*, V. 98, No. 4, July-Aug., pp. 462-469.

Kowalsky, M. J.; Zia, P.; Wagner, M. C.; and Warren, B. A., 2002, "The Behavior of Prestressed High Performance Concrete Bridge Girders for US Highway 401 over the Neuse River in Raleigh, NC," *Report* No. FHWA/NC/2002-003, North Carolina Department of Transportation, Raleigh, NC, 140 pp.

Legeron, F., and Paultre, P., 2000, "Prediction of the Modulus of Rupture of Concrete," *ACI Materials Journal*, V. 97, No. 2, Mar.-Apr., pp. 193-200.

Leming, M. L.; Schemmel, J. J.; Ahmad, S. H.; and Zia, P., 1993a, "High-Performance Concrete: North Carolina Field Installation Results," *Transportation Research Record* No. 1382, National Research Council, Washington, DC, June, pp. 78-81.

Leming, M. L.; Ahmad, S. H.; Zia, P.; Schemmel, J. J.; and Elliott, R. P., 1993b, "Mechanical Behavior of High Performance Concrete," *Production of High Performance Concrete*, V. 2, SHRP-C-362, Strategic Highway Research Program, National Research Council, Washington, DC, Nov., 92 pp.

Leslie, K. E.; Rajagopalan, K. S.; and Everard, N. J., 1976, "Flexural Behavior of High-Strength Concrete Beams," ACI JOURNAL, *Proceedings* V. 73, No. 9, Sept., pp. 517-521.

Leubkeman, C. H.; Nilson, A. H.; and Slate, F. O., 1985, "Sustained Load Deflection of High-Strength Concrete Beams," *Research Report* No. 85-2, Department of Structural Engineering, Cornell University, Ithaca, NY, 164 pp.

Li, Y.; Langan, B. W.; and Ward, M. A., 1994, "Freezing and Thawing: Comparison between Non-Air-Entrained and Air-Entrained High-Strength Concrete," *High Performance Concrete*, Proceedings of the ACI International Conference, SP-149, V. M. Malhotra, ed., American Concrete Institute, Farmington Hills, MI, pp. 545-560.

Lobo, C. L.; Mullings, G. M.; and Gaynor, R. D., 1994, "Effect of Capping Materials and Procedures on the Measured Compressive Strength of High-Strength Concrete," *Cement, Concrete, and Aggregates*, V. 16, No. 2, Dec., pp. 173-180.

Luciano, J. J.; Nmai, C. K.; and Delgado, J., 1991, "A Novel Approach to Developing High-Strength Concrete," *Concrete International*, V. 13, No. 5, May, pp. 25-29.

Malhotra, V. M.; Zhang, M.; Read, P. H.; and Ryell, J., 2000, "Long-Term Mechanical Properties and Durability Characteristics of High-Strength/High-Performance Concrete Incorporating Supplementary Cementing Materials under Outdoor Exposure Conditions," *ACI Materials Journal*, V. 97, No. 5, Sept.-Oct., pp. 518-525.

Manmohan, D., and Metha, P. K., 1981, "Influence of Pozzolanic, Slag, and Chemical Admixtures on Pore Size Distribution and Permeability of Hardened Cement Pastes," *Cement, Concrete, and Aggregates*, V. 3, No. 1, pp. 63-67.

Mansur, M. A.; Chin, M. S.; and Wee, T. H., 1997, "Flexural Behavior of High-Strength Concrete Beams," *ACI Structural Journal*, V. 94, No. 6, Nov.-Dec., pp. 663-674.

Marchand, J.; Sellevold, E. J.; and Pigeon, M., 1994, "The Deicer Salt Scaling Deterioration of Concrete—An Overview," *Durability of Concrete*, Proceedings of the Third CANMET/ACI International Conference, SP-145, V. M. Malhotra, ed., American Concrete Institute, Farmington Hills, MI, pp. 1-46.

Marsh, B. K.; Day, R. L.; and Bonner, D. G., 1985, "Pore Structure Characteristics Affecting the Permeability of Cement Paste Containing Fly Ash," *Cement and Concrete Research*, V. 15, No. 6, Nov., pp. 1027-1038.

Martinez, S.; Nilson, A. H.; and Slate, F. O., 1982, "Spirally-Reinforced High-Strength Concrete Columns," *Research Report* No. 82-10, Department of Structural Engineering, Cornell University, Ithaca, NY, Aug., pp. 432-441.

Martinez, S.; Nilson, A. H.; and Slate, F. O., 1984, "Spirally-Reinforced High-Strength Concrete Columns," ACI JOURNAL, *Proceedings* V. 81, No. 5, May, pp. 431-442.

Mather, K., 1965, "High Strength, High Density Concrete," ACI JOURNAL, *Proceedings* V. 62, No. 8, Aug., pp. 951-962.

Mindess, S.; Young, J. F.; and Darwin, D., 2003, *Concrete*, Prentice Hall Inc., Englewood Cliffs, NJ, pp. 80-85, 176-181, 459-462, 477-478, 542-543.

Mitchell, D.; Cook, W. D.; Khan, A. A.; and Tham, T., 1993, "Influence of High Strength Concrete on Transfer and Development Length of Pretensioning Strand," *PCI Journal*, V. 38, No. 3, May-June, pp. 52-66.

Mokhtarzadeh, A., and French, C., 2000a, "Mechanical Properties of High-Strength Concrete with Consideration for Precast Applications," *ACI Materials Journal*, V. 97, No. 2, Mar.-Apr., pp. 136-148.

Mokhtarzadeh, A., and French, C., 2000b, "Time-Dependant Properties of High-Strength Concrete with Consideration for Precast Applications," *ACI Materials Journal*, V. 97, No. 3 May-June, pp. 263-271.

Mokhtarzadeh, A.; Kriesel, R.; French, C.; and Snyder, M., 1995, "Mechanical Properties and Durability of High Strength Concrete for Prestressed Bridge Girders," *Transportation Research Record* No. 1478, Washington, DC, pp. 20-29.

Moreno, J., 1998, "High-Performance Concrete: Economic Considerations," *Concrete International*, V. 20, No. 3, Mar., pp. 68-70.

Mphonde, A. G., and Frantz, G. C., 1984, "Shear Tests of High and Low Strength Concrete Beams without Stirrups," ACI JOURNAL, *Proceedings* V. 81, No. 4, July-Aug., pp. 350-357.

Mphonde, A. G., and Frantz, G. C., 1985, "Shear Tests of High- and Low-Strength Concrete Beams with Stirrups," *High-Strength Concrete*, SP-87, American Concrete Institute, Farmington Hills, MI, pp. 179-196.

Munn, R. L., 1998, "Development, Production and Application of High Performance Concrete in Australia," *Proceedings of the International Conference on High Performance High Strength Concrete*, Perth, Australia, Aug., pp. 123-140.

Myers, J. J., 1999, "How to Achieve a Higher Modulus of Elasticity," *HPC Bridge Views*, No. 5, Sept.-Oct., p. 3.

Myers, J. J., 2001, "Permeability Performance of High Performance Concrete Subjected to Various Curing Regimes," *Transportation Research Record* No. 1775, National Research Council, Washington, DC, pp. 77-89.

Myers, J. J., and Carrasquillo, R. L., 1998, "The Production and Quality Control of High Performance Concrete in Texas Bridge Structures," *Report* No. FHWA/TX-05/9-580/589-1, Center for Transportation Research, University of Texas at Austin, Austin, TX, Dec., pp. 32-33, 226-231, 424.

Myers, J. J., and Carrasquillo, R. L., 2000, "Influence of Hydration Temperature on the Durability and Mechanical Property Performance of HPC Prestressed/Precast Beams," *Journal of the Transportation Research Board*, No. 1696, V. 1, Publication 5B0038, National Research Council, Washington, DC, Apr., pp. 131-142.

Myers, J. J., and Yang, Y., 2004, "Field and Laboratory Performance of Prestressed High Performance Concrete Girders for Missouri Bridge Structures," *Report* No. 04-49, Center for Infrastructure Engineering Studies, Missouri University of Science and Technology, Rolla, MO, June, pp. 96-98, 102.

Myers, J. J.; Touma, W.; and Carrasquillo, R. L., 1997, "Permeability of High Performance Concrete: Rapid Chloride

Ion Test vs. Chloride Ponding Test," *Proceedings for the PCI/FHWA International Symposium on High Performance Concrete, Advanced Concrete Solutions for Bridges and Transportation Structures*, New Orleans, LA, Oct., pp. 268-282.

Nagataki, S., 1978, "On the Use of Superplasticizers," *Seminar on Special Concretes*, 8th FIP Congress (London, 1978), Fédération Internationale de la Précontrainte, Wexham Springs, 15 pp.

Nagataki, S., and Yonekura, A., 1978, "Studies of the Volume Changes of High Strength Concretes with Superplasticizer," *Journal of the Japan Prestressed Concrete Engineering Association*, V. 20, Tokyo, pp. 26-33.

National Crushed Stone Association, 1975, "High Strength Concrete," *Manual of Concrete Materials—Aggregates*, Washington, DC, Jan., 16 pp.

Nawy, E. G., 2001, *Fundamentals of High Performance Concrete*, second edition, John Wiley & Sons, New York, 410 pp.

Nawy, E. G., 2003, *Reinforced Concrete: A Fundamental Approach*, fifth edition, Pearson Education, Upper Saddle River, NJ, pp. 49-51.

Nebesar, B., and Carette, G. G., 1986, "Variations in the Chemical Composition, Surface Area, Fineness and Pozzolanic Activity of a Canadian Condensed Silica Fume," Project MRP-3.6.0.4.65, CANMET, Ottawa, Canada, pp. 42-45.

Nedderman, H., 1973, "Flexural Stress Distribution in Very-High Strength Concrete," MS thesis, University of Texas at Arlington, Arlington, TX, 182 pp.

Neville, A. M., 1981, *Properties of Concrete*, Pitman Books, 779 pp.

Neville, A. M., 1996, *Properties of Concrete*, fourth edition, John Wiley & Sons, New York, 844 pp.

Neville, A. M., 1997, "Aggregate Bond and Modulus of Elasticity of Concrete," *ACI Materials Journal*, V. 94, No. 1, Jan.-Feb., pp. 71-74.

Ngab, A. S.; Slate, F. O.; and Nilson, A. H., 1980, "Behavior of High-Strength Concrete Under Sustained Compressive Stress," *Research Report* No. 80-2, Department of Structural Engineering, Cornell University, Ithaca, NY, Feb., 201 pp.

Ngab, A. S.; Nilson, A. H.; and Slate, F. O., 1981, "Shrinkage and Creep of High Strength Concrete," ACI JOURNAL, *Proceedings* V. 78, No. 4, Apr., pp. 255-261.

Nmai, C. K.; Schlagbaum, T.; and Violetta, B., 1998, "A History of Mid-Range Water-Reducing Admixtures," *Concrete International*, V. 20, No. 4, Apr., pp. 45-50.

Norges Standardiseringsfund, 1992, "Concrete Structures—Design Rules," NS 3473, Oslo, Norway.

Obla, K. H.; Hill, R.; Cunningham, R.; Thomas, M. D. A.; Balaguru, P. N.; and Cook, J., 2001, "High Strength High Performance Concrete Containing Ultra Fine Fly Ash," *Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete*, Proceedings of the Seventh CANMET/ACI International Conference, Supplementary Papers, Chennai, India, July 22-27.

Ozbakkaloglu, T., and Saatcioglu, M., 2004, "Rectangular Stress Block for High-Strength Concrete," *ACI Structural Journal*, V. 101, No. 4, July-Aug., pp. 475-483.

Ozcebe, G.; Ersoy, E.; and Tankut, T., 1999, "Evaluation of Minimum Shear Reinforcement Requirements for Higher Strength Concrete," *ACI Structural Journal*, V. 96, No. 3, May-June, pp. 361-368.

Ozyildirim, C., 1998, "Effects of Temperature on the Development of Low Permeability in Concretes," VTRC 98-R14, Virginia Transportation Research Council, Charlottesville, VA, 11 pp.

Ozyildirim, C., and Gomez, J., 2000, "HPC in Virginia's Bridge Structures," Symposium Proceedings, PCI/FHWA/FIB International *Symposium on High Performance Concrete*, Orlando, FL, Precast/Prestressed Concrete Institute, Chicago, IL, pp. 741-750.

Parrott, L. J., 1969, "The Properties of High Strength Concrete," *Technical Report* No. 42.417, Cement and Concrete Association, Wexham Springs, 12 pp.

Pastor, J. A.; Nilson, A. H.; and Slate, F. O., 1984, "Behavior of High Strength Concrete Beams," *Research Report* No. 84-3, Department of Structural Engineering, Cornell University, Ithaca, NY, 311 pp.

Paulson, K. A.; Nilson, A. H.; and Hover, K. C., 1989, "Immediate and Long-Term Deflection of High Strength Concrete Beams," *Research Report* No. 89-3, Department of Structural Engineering, Cornell University, Ithaca, NY, 230 pp.

Peece, M., and Fabbrocino, G., 1999, "Plastic Rotation Capacity of Beams in Normal and High-Performance Concrete," *ACI Structural Journal*, V. 96, No. 2, Mar.-Apr., pp. 290-296.

Pendyala, R. S., and Mendis, P., 2000, "Experimental Study on Shear Strength of High-Strength Concrete Beams," *ACI Structural Journal*, V. 97, No. 4, July-Aug., pp. 564-571.

Pendyala, R.; Mendis, P.; and Patnaikuni, I., 1996, "Full-Range Behavior of High-Strength Concrete Flexural Members: Comparison of Ductility Parameters of High and Normal-Strength Concrete Members," *ACI Structural Journal*, V. 93, No. 1, Jan.-Feb., pp. 30-35.

Perenchio, W. F., 1973, "An Evaluation of Some of the Factors Involved in Producing Very High-Strength Concrete," *Research and Development Bulletin* No. RD014, Portland Cement Association, Skokie, IL, 7 pp.

Perenchio, W. F., and Klieger, P., 1978, "Some Physical Properties of High Strength Concrete," *Research and Development Bulletin* No. RD056.01T, Portland Cement Association, Skokie, IL, 7 pp.

Pfeifer, D. W.; Magura, D. D.; Russell, H. G.; and Corley, W. G., 1971, "Time Dependent Deformations in a 70 Story Structure," *Designing for Effects of Creep, Shrinkage, Temperature in Concrete Structures*, SP-37, American Concrete Institute, Farmington Hills, MI, pp. 159-185.

Pfeifer, D. W., and Ladgren, J. R., 1981, "Energy Efficient Accelerated Curing of Concrete—A Laboratory Study for Plant-Produced Prestressed Concrete," *Technical Report* No. 1, Prestressed Concrete Institute, Chicago, IL, Dec., pp. 159-186.

Phan, L. T., and Carino, N. J., 2000, "Fire Performance of High Strength Concrete: Research Needs," *2000 ASCE Structures Congress Proceedings*, May. (CD-ROM)

Philleo, R. E., 1986, "Freezing and Thawing Resistance of High-Strength Concrete," *NCHRP Synthesis of Highway*

*Practice* No. 129, Transportation Research Board, Washington, DC, 31 pp.

Pinto, R. C. A., and Hover, K. C., 2001, "Frost and Scaling Resistance of High Strength Concrete," RD 122, Portland Cement Association, Skokie, IL, 75 pp.

Powers, T. C.; Copeland, L. E.; Hayes, J. C.; and Mann, H. M., 1954, "Permeability of Portland Cement Paste," ACI JOURNAL, *Proceedings* V. 51, No. 11, pp. 285-298.

Prestressed Concrete Institute (PCI), 2004, *PCI Design Handbook*, sixth edition, Chicago, IL, pp. 4-84 to 4-87.

Price, W. H., 1951, "Factor Influencing Concrete Strength," ACI JOURNAL, *Proceedings* V. 47, No. 6, Feb., pp. 417-432.

Radain, T. A.; Samman, T. A.; and Wafa, F. F., 1993, "Mechanical Properties of High-Strength Concrete," *Proceedings of the Third International Symposium on Utilization of High-Strength Concrete*, V. 2, Lillehammer, Norway, pp. 1209-1216.

Ralls, M. L., 1998, "Texas High-Performance Concrete Bridges—How Much Do They Cost?" *Concrete International*, V. 20, No. 3, Mar., pp. 71-74.

Ramachandran, V. S.; Malhotra, V. M.; Jolicoeur, C.; and Spirato, N., 1998, "Superplasticizers: Properties and Applications in Concrete," *Publication* No. MTL-14, CANMET National Resources, Ottawa, ON, Canada.

Rangan, B. V.; Saunders, P.; and Seng, E. J., 1991, "Design of High-Strength Concrete Columns," *Evaluation and Rehabilitation of Concrete Structures and Innovation in Design*, SP-128, V. M. Malhotra, ed., American Concrete Institute, Farmington Hills, MI, pp. 851-862.

Rasmussen, L. J., and Baker, G., 1995, "Torsion in Reinforced Normal and High-Strength Concrete Beams," *ACI Structural Journal*, V. 92, No. 1, Jan.-Feb., pp. 55-63.

Razvi, S. R., and Saatcioglu, M., 1994, "Strength and Deformability of Confined High-Strength Concrete Columns," *ACI Structural Journal*, V. 91, No. 6, Nov.-Dec., pp. 678-687.

Richart, F. E., and Brown, R. L., 1934, "An Investigation of Reinforced Concrete Columns," *Bulletin* No. 267, University of Illinois Engineering Experiment Station, Urbana, IL, 94 pp.

Roberts-Wollman, C.; Cousins, T.; Gomez, J.; and Ozyildirim, C., 2000, "Use of High Performance Concrete in a Bridge Structure in Virginia," *Symposium Proceedings*, PCI/FHWA/FIB International Symposium on High Performance Concrete, Orlando, FL, Precast/Prestressed Concrete Institute, Chicago, IL, pp. 687-696.

Rüsch, H., 1960, "Researches Toward a General Flexural Theory for Structural Concrete," ACI JOURNAL, *Proceedings* V. 57, No. 1, pp. 1-28.

Russell, B. W., and Burns, N. H., 1996, "Measured Transfer Lengths of 0.5 and 0.6 in. Strands in Pretensioned Concrete," *PCI Journal*, V. 41, No. 5, Sept.-Oct., pp. 44-65.

Russell, H. G., 1994, *High Performance Concretes and Applications*, Chapter 9, Edward Arnold, London, pp. 313-340.

Russell, H. G., 1997, "High-Performance Concrete—From Buildings to Bridges," *Concrete International*, V. 19, No. 8, Aug., pp. 62-63.

Russell, H. G., and Corley, W. G., 1978, "Time-Dependent Behavior of Columns in Water Tower Place," *Douglas McHenry International Symposium on Concrete and Concrete Structures*, SP-55, American Concrete Institute, Farmington Hills, MI, pp. 347-373.

Russell, H. G., and Roller, J. J., 1990, "Shear Strength of High-Strength Concrete Beams," *ACI Structural Journal*, V. 87, No. 2, Mar.-Apr., pp. 191-198.

Russell, H. G.; Voltz, J. S.; and Bruce, R. N., 1997, "Optimized Sections for High-Strength Concrete Bridge Girders," *Report* No. FHWA-RD-95-180, FHWA, U.S. Department of Transportation, 156 pp.

Sammam, T. A.; Mirza, W. H.; and Wafa, F. F., 1996, "Plastic Shrinkage Cracking of Normal- and High-Strength Concrete: A Comparative Study," *ACI Materials Journal*, V. 93, No. 1, Jan.-Feb., pp. 36-40.

Sanchez, A., and Hester, W. T., 1990, "Designer-Contractor Dialogue and High-Strength Concrete Construction," *The Construction Specifier*, V. 43, No. 12, pp. 56-62.

Sarsam, K. F., and Al-Musawi, J. M. S., 1992, "Shear Design of High- and Normal-Strength Concrete Beams with Web Reinforcement," *ACI Structural Journal*, V. 89, No. 6, Nov.-Dec., pp. 658-664.

Saucier, K. L., 1968, "Evaluation of Spiral-Blade Concrete Mixer, Shelbyville Reservoir Project, Shelbyville, IL," *Miscellaneous Paper* No. 6-975, U.S. Army Engineer, Waterways Experiment Station, Vicksburg, MS, Mar., 17 pp.

Saucier, K. L.; Tynes, W. O.; and Smith, E. F., 1965, "High-Compressive Strength Concrete—Report 3, Summary Report," *Miscellaneous Paper* No. 6-520, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, Sept., 87 pp.

Sauvageot, G., 1995, "Northumberland Strait Crossing, Canada," *Conference Proceedings, Fourth International Bridge Engineering Conference*, San Francisco, CA, Aug. 28-30, V. 1, Transportation Research Board, Washington, DC, pp. 238-248.

Schmidt, W., and Hoffman, E. S., 1975, "9,000-psi Concrete: Why? Why Not?" *Journal of Civil Engineering*, ASCE, V. 45, No. 5, May, pp. 52-55.

Schmidt, M., and Fehling, E., 2004, "Ultra-High-Performance Concrete: Research, Development and Application in Europe," *Proceedings for International Symposium on Ultra-High Performance Concrete*, Kassel, Germany, Sept., 32 pp.

Sengul, Ö.; Tasdemir, C.; and Tasdemir, M. A., 2002, "Influence of Aggregate Type on Mechanical Normal and High Strength Concretes," *ACI Materials Journal*, V. 99, No. 6, Nov.-Dec., pp. 528-533.

Shah, S. P., 1981, "High Strength Concrete: A Workshop Summary," *Concrete International*, V. 3, No. 5, May, pp. 94-98.

Shah, S. P.; Gokoz, U.; and Ansari, F., 1981, "An Experimental Technique for Obtaining Complete Stress-Strain Curves for High Strength Concrete," *Cement, Concrete and Aggregates*, V. 3, No. 1, pp. 21-27.

Shahawy, M. A.; Issa, M.; and Batchelor, B., 1992, "Strand Transfer Lengths in Full Scale AASHTO

Prestressed Concrete Girders," *PCI Journal*, V. 37, No. 3, May-June, pp. 84-96.

Sheikh, S. A., and Uzumeri, S. M., 1980, "Strength and Ductility of Tied Concrete Columns," *Journal of the Structural Division*, ASCE, V. 106, No. ST5, pp. 1079-1102.

Shideler, J. J., 1957, "Lightweight-Aggregate Concrete for Structural Use," ACI JOURNAL, *Proceedings* V. 54, No. 4, Oct., pp. 299-328.

Shin, S. W.; Ghosh, S. K.; and Moreno, J., 1989, "Flexural Ductility of Ultra High-Strength Concrete Members," *ACI Structural Journal*, V. 86, No. 4, July-Aug., pp. 394-400.

Shing, P. B.; Cooke, D. E.; Frangopol, D. M.; Leonard, M. A.; McMullen, M. L.; and Hutter, W., 2000, "Strand Development and Transfer Length Tests on High Performance Concrete Box Girders," *PCI Journal*, V. 45, No. 5, Sept.-Oct., pp. 96-109.

Skrastins, J. I., 1970, "Toward High-Strength Concrete," *Modern Concrete*, V. 34, No. 1, May, pp. 44-48.

Smadi, M. M.; Slate, F. O.; and Nilson, A. H., 1982, "Time-Dependent Behavior of High-Strength Concrete Under High Sustained Compressive Stresses," *Research Report* No. 82-16, Department of Structural Engineering, Cornell University, Ithaca, NY, 264 pp.

Smadi, M. M.; Slate, F. O.; and Nilson, A. H., 1985, "High-, Medium-, and Low-Strength Concretes Subject to Sustained Overloads—Strains, Strengths, and Failure Mechanisms," ACI JOURNAL, *Proceedings* V. 82, No. 5, Sept.-Oct, pp. 657-664.

Smadi, M. M.; Slate, F. O.; and Nilson, A. H., 1987, "Shrinkage and Creep of High-, Medium-, and Low-Strength Concretes, Including Overloads," *ACI Materials Journal*, V. 84, No. 3, May-June, pp. 224-234.

Smith, E. F.; Tynes, W. O.; and Saucier, K. L., 1964, "High-Compressive-Strength Concrete, Development of Concrete Mixtures," *Technical Documentary Report* No. RTD TDR 63 3114, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, Feb., 44 pp.

Strehlow, R. W., 1973, *Concrete Plant Production*, Concrete Plant Manufacturers Bureau, Silver Spring, MD, 112 pp.

*Structure Magazine*, 2003, "Outstanding Project, Buildings over $25M in Construction Value, Reliant Stadium," Dec. 2003-Jan. 2004, p. 28.

Swamy, R. N., and Anand, K. L., 1973, "Shrinkage and Creep of High Strength Concrete," *Civil Engineering and Public Works Review* (London), V. 68, No. 807, Oct., pp. 859-865, 867-868.

Tabatabai, H., and Dickson, T. J., 1993, "The History of the Prestressing Strand Development Length Equation," *PCI Journal*, V. 38, No. 6, Nov.-Dec., pp. 64-75.

Tadros, M. K.; Huo, X.; Ma, Z.; and Baishya, M., 1999, "Structural Design of High-Performance Concrete Bridges," *High-Performance Concrete*, SP-189, American Concrete Institute, Farmington Hills, MI, pp. 9-36.

Tadros, M. K.; Al-Omaishi, N.; Seguirant, S. J.; and Gallt, J. G., 2003, "Prestress Losses in Pretensioned High-Strength Concrete Bridge Girders," *NCHRP Report 496*, Transportation Research Board, Washington, DC, pp. 7-13, 17-30, 38-45.

Tazawa, E., ed., 1999, *Autogenous Shrinkage of Concrete*, Taylor & Francis Books Ltd., Jan., 424 pp.

Teychenné, D. C.; Parrott, L. J.; and Pomeroy, C. D., 1978, *Current Paper CP 23/78*, Building Check Establishment, Garston, Watford, 11 pp.

Thoman, W. H., and Raeder, W., 1934, "Ultimate Strength and Modulus of Elasticity of High Strength Portland Cement Concrete," ACI JOURNAL, *Proceedings* V. 30, pp. 231-238.

Tikalsky, P. J., and Carrasquillo, R. L., 1989, "The Effect of Fly Ash on the Sulfate Resistance of Concrete," *Research Report* 481-5, Center for Transportation Research, University of Texas at Austin, Austin, TX, Aug., pp. 79-80.

Tomosawa, F., and Noguchi, T., 1993, "Relationship between Compressive Strength and Modulus of Elasticity of High-Strength Concrete," *Proceedings of the Third International Symposium on Utilization of High-Strength Concrete*, V. 2, Lillehammer, Norway, pp. 1247-1254.

USACE, 1995, EM 1110-2-2002, Evaluation and Repair of Concrete Structures, Department of the Army, U.S. Army Corps of Engineers, Washington, DC, pp. 1-5.

Vallenas, J.; Bertero, V. V.; and Popov, E. P., 1977, "Concrete Confined by Rectangular Hoops and Subjected to Axial Loads," *Report* No. UCB/EERC-77/13, Earthquake Engineering Research Center, University of California, Berkeley, CA, 128 pp.

Vichit-Vadakan, W.; Carino, N. J.; and Mullings, G. M., 1998, "Effect of Elastic Modulus of Capping Material on Measured Strength of High-Strength Concrete Cylinders," *Cement, Concrete, and Aggregates*, V. 20, No. 2, Dec., pp. 227-234.

Walker, S., and Bloem, D. L., 1960, "Effects of Aggregate Size on Properties of Concrete," ACI JOURNAL, *Proceedings* V. 57, No. 9, Sept., pp. 283-298.

Walsh, J. T., 1977, personal correspondence to I. G. Cantor, Department of Buildings, New York, Aug. 11.

Wang, P. T.; Shah, S. P.; and Naaman, A. E., 1978a, "Stress-Strain Curves of Normal and Lightweight Concrete in Compression," ACI JOURNAL, *Proceedings* V. 75, No. 11, Nov., pp. 603-611.

Wang, P. T.; Shah, S. P.; and Naaman, A. E., 1978b, "High Strength Concrete in Ultimate Strength Design," *Proceedings*, ASCE, V. 104, No. ST11, pp. 1761-1773.

Whiting, D., and Kulman, L., 1987, "Curing and Chloride Permeability," *Concrete International*, V. 9, No. 4, Apr., pp. 18-21.

Wiegrink, K.; Marikunte, S.; and Shah, S. P., 1996, "Shrinkage Cracking of High-Strength Concrete," *ACI Materials Journal*, V. 93, No. 5, Sept.-Oct., pp. 409-415.

Wills, M. H. Jr., 1967, "How Aggregate Particle Shape Influences Concrete Mixing Water Requirement and Strength," *Journal of Materials*, V. 2, No. 4, pp. 843-865.

Wischers, G., 1978, "Applications and Effects of Compressive Loads on Concrete," *Betontechnische Berichte*, Betone Verlag GmbH, Düsseldorf, pp. 31-56. (in German)

Wolsiefer, J., 1984, "Ultra High-Strength Field Placeable Concrete with Silica Fume Admixtures," *Concrete International*, V. 6, No. 4, Apr., pp. 25-31.

Xiao, Y., and Ma, R., 1998, "Seismic Behavior of High Strength Concrete Beams," *International Journal of the Structural Design of Tall Buildings*, V. 7, No. 1, pp. 73-90.

Yerlici, V. A., and Özturan, T., 2000, "Factors Affecting Anchorage Bond Strength in High-Performance Concrete," *ACI Structural Journal*, V. 97, No. 3, May-June, pp. 499-507.

Yong, Y. K.; Nour, M. G.; and Nawy, E. G., 1988, "Behavior of Laterally-Confined High-Strength Concrete Under Axial Loads," *Journal of Structural Engineering*, ASCE, V. 114, No. 2, pp. 332-351.

Yoon, Y. S.; Cook, W. D.; and Mitchell, D., 1996, "Minimum Shear Reinforcement in Normal, Medium, and High-Strength Concrete Beams," *ACI Structural Journal*, V. 93, No. 5, Sept.,-Oct., pp. 576-584.

Young, J. F., 1988, "A Review of the Pore Structure of Cement Paste and Concrete and Its Influence on Permeability," *Permeability of Concrete*, SP-108, D. Whiting and A. Walitt, eds., American Concrete Institute, Farmington Hills, MI, pp. 1-18.

Zakka, Z. A., 1989, "Effects of High Range Water Reducers on the Properties of Fresh and Hardened Concrete," master's thesis, Department of Civil Engineering, University of Texas at Austin, Austin, TX, Oct., 300 pp.

Zia, P., 1977, "Structural Design with High Strength Concrete," *Report* No. PZIA-77-01, Civil Engineering Department, North Carolina State University, Raleigh, NC, Mar. 65 pp.

Zia, P., 1983, "Review of ACI Code for Design with High-Strength Concrete," *Concrete International*, V. 5, No. 8, Aug., pp. 16-20.

Zia, P., and Mostafa, T., 1977, "Development Length of Prestressing Strands," *PCI Journal*, V. 22, No. 5, Sept.-Oct., pp. 54-65.

Zia, P.; Schemmel, J. J.; and Tallman, T. E., 1989, "Structural Applications of High-Strength Concrete," *Report* No. FHWA/NC/89-0006, Center for Transportation Engineering Studies, North Carolina State University, Raleigh, NC, June, 330 pp.

Zia, P.; Ahmad, S. H.; Leming, M. L.; Schemmel, J. J; and Elliott, R. P., 1993a, "Mechanical Behavior of High Performance Concrete," *High Early Strength Concrete*, V. 4, SHRP-C-364, Strategic Highway Research Program, National Research Council, Washington, DC, Dec., 179 pp.

Zia, P.; Ahmad, S. H.; and Leming, M. L., 1993b, "High Performance Concrete—A State of the Art Report (1989-1994)," FHWA-RD-97-030, Strategic, Washington, DC. (CD-ROM)

American Concrete Institute®
*Advancing concrete knowledge*

As ACI begins its second century of advancing concrete knowledge, its original chartered purpose remains "to provide a comradeship in finding the best ways to do concrete work of all kinds and in spreading knowledge." In keeping with this purpose, ACI supports the following activities:

- Technical committees that produce consensus reports, guides, specifications, and codes.
- Spring and fall conventions to facilitate the work of its committees.
- Educational seminars that disseminate reliable information on concrete.
- Certification programs for personnel employed within the concrete industry.
- Student programs such as scholarships, internships, and competitions.
- Sponsoring and co-sponsoring international conferences and symposia.
- Formal coordination with several international concrete related societies.
- Periodicals: the *ACI Structural Journal* and the *ACI Materials Journal*, and *Concrete International*.

Benefits of membership include a subscription to *Concrete International* and to an ACI Journal. ACI members receive discounts of up to 40% on all ACI products and services, including documents, seminars and convention registration fees.

As a member of ACI, you join thousands of practitioners and professionals worldwide who share a commitment to maintain the highest industry standards for concrete technology, construction, and practices. In addition, ACI chapters provide opportunities for interaction of professionals and practitioners at a local level.

**American Concrete Institute**
**38800 Country Club Drive**
**Farmington Hills, MI 48331**
**U.S.A.**
**Phone: 248-848-3700**
**Fax: 248-848-3701**

**www.concrete.org**

# Report on High-Strength Concrete