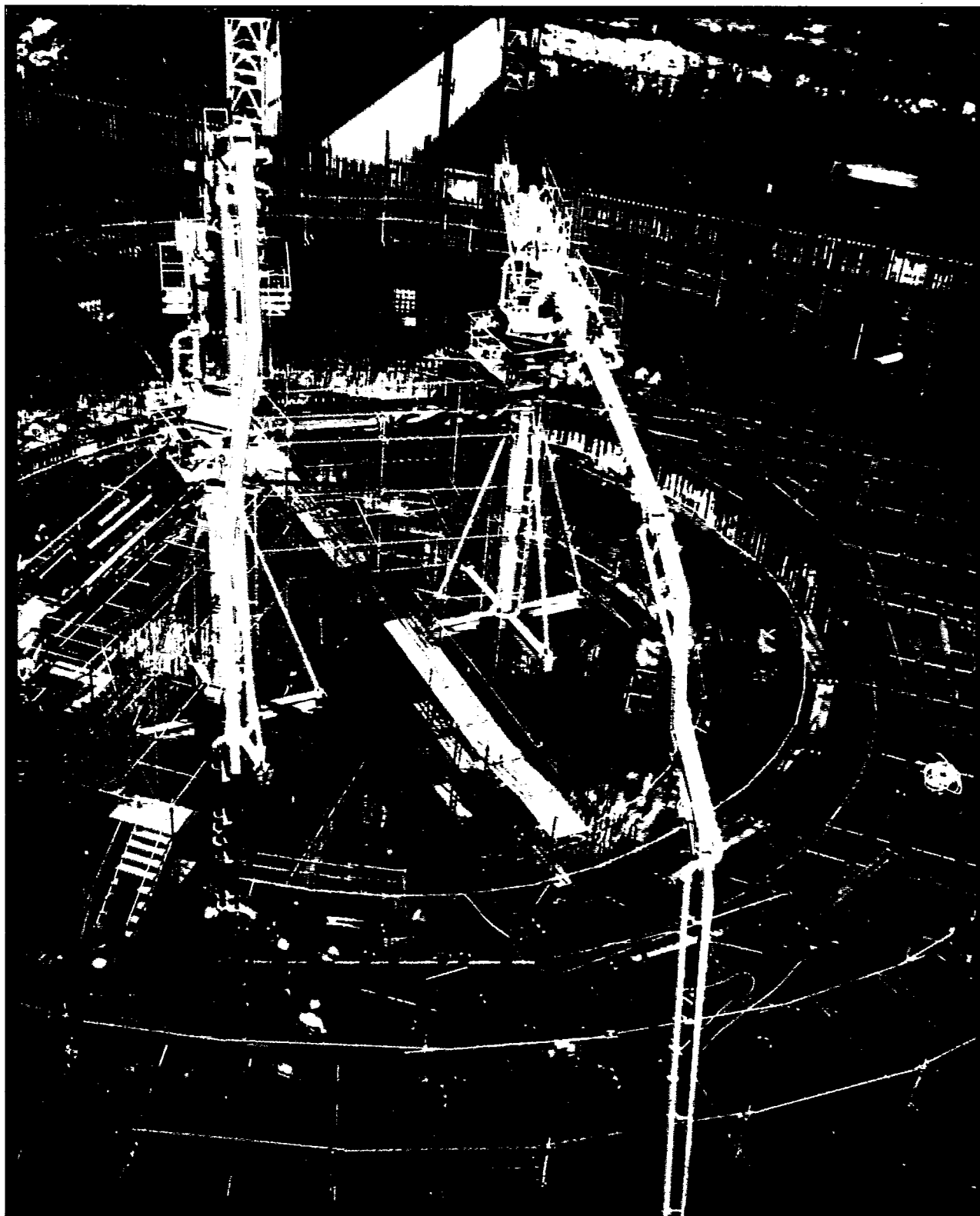


HIGH STRENGTH CONCRETE



Compilation 17

American Concrete Institute



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High Strength Concrete

ACI Compilation

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Preface

ACI Compilations combine material previously published in Institute periodicals to provide compact and ready reference on specific topics. The material in a compilation does not necessarily represent the opinion of an ACI technical committee — only the opinions of the individual authors. However, the information presented here is considered to be a valuable resource for readers interested in the subject.

Kenneth L. Saucier

Chairman, ACI Committee 363

High Strength Concrete

On the cover: Automation, including robotic technology, is now a major factor in the construction industry, especially in nations like Japan. As shown in this photo, automatically controlled cranes are frequently used in placing concrete on jobs where large quantities are necessary.

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

High-Strength Concrete

by H. G. Russell

"Imagine, for example, concrete with an available strength of 10,000 pounds per square inch. Smaller columns, thinner and lighter beams and slabs would at once result. Precast units, easy to handle, would be available. The present limiting heights of buildings, of spans of bridges would be at least double. A new basis of design, new codes and specifications would be required," said Professor S. C. Hollister, former Dean of Engineering at Cornell University during the ACI's 30th Annual Dinner in 1934.

As we all know, the "idle dreams" of Prof. Hollister have now been accomplished. We have 10,000 psi (69 MPa) compressive strength concrete readily available in many parts of North America and other major industrial nations. The development Prof. Hollister dreamed about has been accomplished over the years.

In the 1950s, we had 5000 psi (34 MPa) concrete. In the 1960s, we had the commercial use of 6000 and 7500 psi (41 and 52 MPa) concretes. In the early 1970s, we reached 9000 psi (62 MPa) concrete. And toward the end of the 1980s, we reached 20,000 psi (138 MPa) compressive strength concrete. As we look to the future and the year 2000, there is no doubt in my mind we will have a concrete with a compressive strength of at least 25,000 psi (172 MPa) and even 30,000 psi (207 MPa) produced commercially without using super-exotic materials.

The results of having these materials available will be just as Prof. Hollister predicted. Columns will become even smaller — to the point that we will begin to worry about the lateral stability of those members. Beams and slabs will become lighter, particularly in long-span structures such as cable-stayed bridges and prestressed girders. But, more importantly, we will see these materials used because of their enhanced performance characteristics.

In recent years, we have seen greater use of high-strength concrete in applications where durability is important — highway bridge decks, parking structures, industrial facilities, stilling basins, and offshore structures. Our next development is to combine the material characteristics with comparable structural performance so the two factors are used to their greatest advantage.

We will need to look constantly at the basis of our designs. Prof. Hollister said that new codes and specifications would be required, and as these higher strength materials have become available, it has been necessary to review our codes and specifications for their applicability to high-strength concretes.

In its report in 1988, ACI Committee 363, High Strength Concrete, identified numerous research needs for high-strength concrete, many of which were driven by a need for information with which to modify codes and specifications to accommodate new material characteristics. And we can't stop with the current research on 10,000 to 15,000

psi (69 to 103 MPa) concretes. We must look at the engineering characteristics of concretes with strengths of 15,000 to 25,000 psi (103 to 172 MPa) to validate or modify design methods.

As an industry, we have a great opportunity to capitalize on a material with strengths approaching those of structural steel; however, we have to advance in unison. At the present time, materials technology has outpaced engineering knowledge. Hopefully, this will change in the next few years and we will be able to pursue this opportunity before us.

To rephrase Prof. Hollister's dream, "Imagine concrete with a compressive strength of 30,000 psi (207 MPa). Smaller columns and lighter bridge girders will result. Taller buildings and longer span bridges will be built in concrete. Codes and specifications that support and include the use of these materials will be developed."

ACU Fellow **Henry G. Russell** is president of Construction Technology Laboratories, Inc., Skokie, Ill. He has authored numerous publications related to the



structural design of reinforced and prestressed concrete. He is a Director of ACI and a member of ACI Committees 363, High Strength Concrete; 223, Expansive Cement Concrete; and 358, Concrete Guideways; and the Fellows Nominating Committee, Planning Committee, and International Activities Committee.

Expanding Applications of Materials Technology

PCA Research on High-Strength Concrete

by Anthony E. Florato

Reviews Portland Cement Association-sponsored research on high-strength normal-weight concrete. This research has reflected construction-industry needs for technological information about high-strength concrete applications for prestressed members, building, and bridges, and in situations where durability rather than strength is the primary concern.

The development and use of high-strength concrete has been an evolutionary process. The objectives and direction of Portland Cement Association (PCA) research on high-strength concrete reflect this evolution.

The following review of PCA research was originally conceived as a convenient summary of one source of technical information on high-strength concrete. As work progressed, it was evident that this summary also provides a unique perspective on how developments in materials technology can meet changing needs of the construction industry.

Since this review is limited to PCA-sponsored research, it does not represent a comprehensive summary of the current state of the art. Many other organizations, researchers, and practitioners are involved in work on high-strength concrete, but their contributions are

not reflected here. This review also is limited to normal-weight concretes, as developments in light-weight concrete technology require more space than can be given here.

PCA is very heavily oriented toward applied, or "market driven" research; thus, work on high-strength concrete closely mirrors needs of the construction industry.

Needs of the prestressing industry led to some of the first applications of high-strength concrete, and the development of high-rise buildings was, and remains, a strong impetus for high-strength concrete research. The experience and technology gained from use of high-strength concrete in buildings has provided a base for such additional applications as bridges. Perhaps the most recent direction for high-strength concrete has been in applications where durability rather than strength is the primary design concern.

High-strength concrete for prestressed members

Early work on high-strength concrete was directed toward providing high early strengths for prestressing,^{1,2} where relatively high-strength concretes are required even though the high later-age strengths may not be used.

Work by Klieger in the 1950s defined mix proportion characteristics, consolidation techniques, and curing conditions to develop one-

day compressive strengths in excess of 2000 psi (13.8 MPa) for Type I cements and 4000 psi (27.6 MPa) for Type III cements.¹ Strengths at 28 days ranged from about 7500 to 8500 psi (51.7 to 58.6 MPa) for cylinders moist cured at room temperature. Mixes evaluated had water-cement ratios ranging from 0.29 to 0.41, with cement contents of 470 to 1128 lb/yd³ (279 to 670 kg/m³).

Accelerated curing at elevated temperatures provided one-day strengths up to 180 percent of those of equivalent moist-cured cylinders made from Type I cement. The mixes had very low slumps—less than 1.5 in. (38 mm)—and were made without the water reducers that are now common.

Based on his tests, Klieger recommended the following means for obtaining high-quality high early strength concrete:

- Use low water-cement ratio mixes.
- Use Type III cement.
- Use mechanical vibration to permit more aggregate per unit volume.
- Use saturated steam at atmospheric pressure at temperatures below the boiling point of water, together with insulation.
- Carefully control aggregate gradation, batch weights, mixing, compacting, and curing.
- Use water curing during early hours of hydration.

Keywords: bridges; buildings; concrete durability; high-strength concrete; prestressed concrete; reinforced concrete; research; structural design.

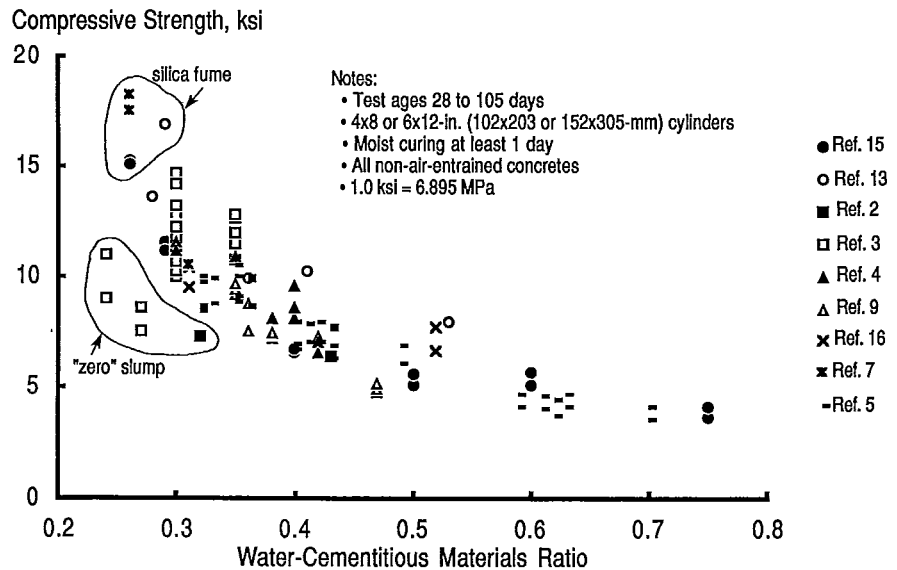


Fig. 1—Summary of strength data as a function of w/c.

Klieger also evaluated freeze-thaw and deicer-scaling resistance as well as creep and shrinkage characteristics of high early strength concretes for prestressing.² Air-entrained and nonair-entrained concretes with water-cement ratios from 0.30 to 0.50, cement contents from 564 to 799 lb/yd³ (335 to 474 kg/m³), and compressive strengths (28-day moist-cured) up to 7630 psi (52.6 MPa) were tested. For air-entrained concretes, air contents ranged from 2.2 to 3 percent for the higher cement content mixes to 3.9 to 6 percent for the lower cement content mixes.

Among other conclusions, these tests indicated:

- All concretes required intentionally entrained air for a high degree of freeze-thaw and deicer-scale resistance.
- Concretes made with Type I and Type III cements were equally durable.
- Curing at elevated temperatures did not impair durability of air-entrained concretes provided some drying followed prior to exposure.
- Richer low water-cement ratio mixes showed lower creep strains than leaner higher water-cement ratio mixes.

High-strength concrete for buildings

Buildings have probably attracted the most attention of any applica-

tion of high-strength concrete, and much of the work on development of high-strength concretes has been directed toward the need for such materials in high-rise structures. High-strength concrete is considered in design of buildings to achieve greater heights, to reduce column sections, and to provide greater stiffness.

Materials development

In 1973, Perenchio³ reported on factors that affect production of high-strength concrete. He considered a number of mix proportioning variables including cement composition and fineness; water-cement ratio; curing; aggregate composition, size, and gradation; and admixtures. At water-cement ratios ranging from 0.22 to 0.35, the low-slump mixes tested reached 28-day compressive strengths (moist cured) up to 13,240 psi (91.3 MPa).

Within this fundamental program, the following findings were highlighted:

- Cement paste was a primary factor in obtaining high strengths. Cements with higher percentages of tricalcium silicate produced higher strength pastes. At equivalent water-cement ratios, finer cements produced higher early age strengths, but fineness was of little significance at later ages.
- The lowest water-cement ratios consistent with workable mixes provided the highest strengths.

- Provision of moist curing, particularly within the first 24 hours, was very important to strength development.
- Use of saturated aggregates provided an internal source of curing water.
- Smaller maximum aggregate sizes provided higher strength concretes.
- Aggregate properties of importance to producing high-strength concrete were strength, bonding potential, and absorption.

As an extension of the fundamental program, Perenchio and Klieger⁴ reported on a study to evaluate some physical properties of high-strength concretes. Three aggregate combinations were used in zero-slump concretes prepared at water-cement ratios of 0.30, 0.35, and 0.40 by weight. Concretes were tested for compressive strength, modulus of elasticity, Poisson's ratio, creep, drying shrinkage, and resistance to freezing and thawing. Both air-entrained and nonair-entrained concretes were tested for freezing and thawing. Compressive strengths up to 11,580 psi (79.8 MPa) at 28 days (moist cured) were tested.

Significant conclusions from this investigation were:

- Compressive strength and modulus of elasticity increased with decreasing water-cement ratio.

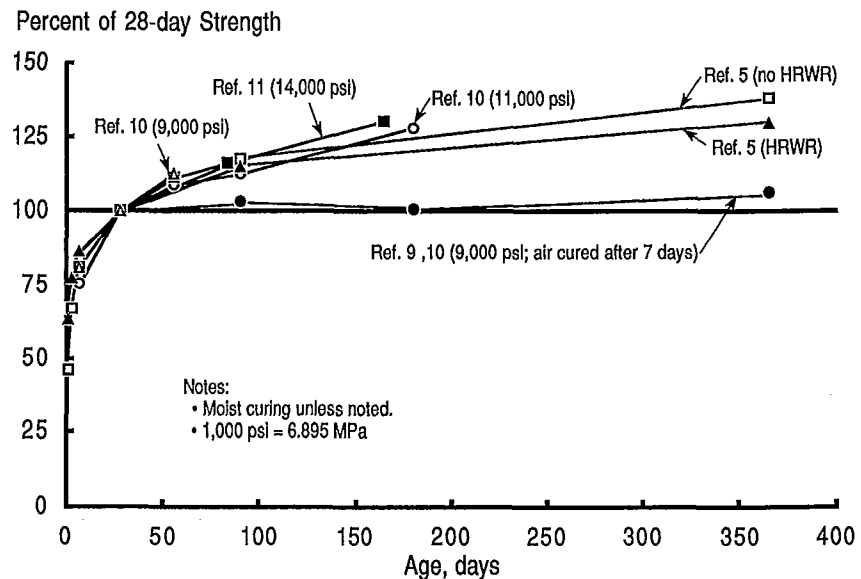


Fig. 2—Compressive strength development.

- Specific creep, in millionths per psi, decreased with decreasing water-cement ratio (increasing strength).
- Drying shrinkage was generally unaffected by change in water-cement ratio. Concretes with lower water-cement ratios appeared to require a longer drying period to reach equilibrium.
- Both air-entrained and nonair-entrained concretes had excellent freeze-thaw resistance. This was attributed to the low freezable water contents and increased tensile strengths of the low water-cement ratio mixes. It is also noted that these tests included a period of drying prior to freeze-thaw exposure.

The work of Klieger and Perenchio was supplemented by an investigation of high-range water reducers (superplasticizers) reported by Whiting in 1979.⁵ Although not specifically directed at high-strength concretes, the work included evaluation of concretes with water-cement ratios down to 0.32 and 28-day moist-cured compressive strengths up to 9100 psi (62.7 MPa). Nominal cement content for the high-strength concretes was 658 lb/yd³ (391 kg/m³). Slumps ranged from 1.7 to 3.2 in. (43 to 81 mm).

Results relevant to high-strength concrete were that low water-cement ratio mixes could be obtained effectively using high-range water reducers, which provided a means

of achieving workability at low water contents. The high-range water reducers lowered net water contents from 10 to 20 percent. Efficiency of water reduction increased with increasing cement content.

Structural design

As work on materials progressed, it was evident that information was also needed to update design criteria for higher strength concretes. One of the first issues that needed to be resolved was the flexural stress distribution for ultimate strength design. In 1977, Kaar, Hanson, and Capell reported on an investigation of stress-versus-strain characteristics of high-strength concrete.⁶

Tests included normal-weight concretes with compressive strengths from 6500 to 14,850 psi (44.8 to 102.4 MPa). Three aggregates were used. Results included stress-versus-strain curves, flexural constants for design, and moduli of elasticity for all concretes tested.

A primary result of this investigation was to define constants for design of high-strength concrete flexural members. In particular, the value of the constant that defines the fraction of the neutral axis depth that can be used for the ACI Building Code⁷ rectangular stress block depth was confirmed. This value is in Section 10.2.7.3 of the ACI Building Code.

A paper by Roller and Russell on the most recent research on struc-

tural design for high-strength concrete was directed toward gathering information on shear strength of beams with web reinforcement.* Beams made with nominal concrete strengths of 10,000 and 17,000 psi (69.0 and 117.2 MPa) were tested. Shear reinforcement ranged from the minimum amount required by ACI 318-83 to the maximum amount that can be assumed when calculating shear capacity.

Results of these tests indicate that for nonprestressed high-strength concrete members subject to shear and flexure only, minimum shear reinforcement must be increased with increased compressive strength. These results have confirmed proposed ACI Building Code revisions that would require increases in the minimum amount of web reinforcement for concretes with compressive strengths in excess of 10,000 psi (69.0 MPa).

Time-dependent deformations

In August 1966, a field investigation at Lake Point Tower, a 70-story, 645 ft (197 m) tall building in Chicago, was begun.⁸ Objectives of the investigation were to measure deformations on what, at the time, was an unusually tall structure, to relate the field measurements to companion laboratory data and to develop methods to predict time-de-

*Roller, J. J., and Russell, H. G., "Shear Strength of High-Strength Concrete Beams with Web Reinforcement," in print.

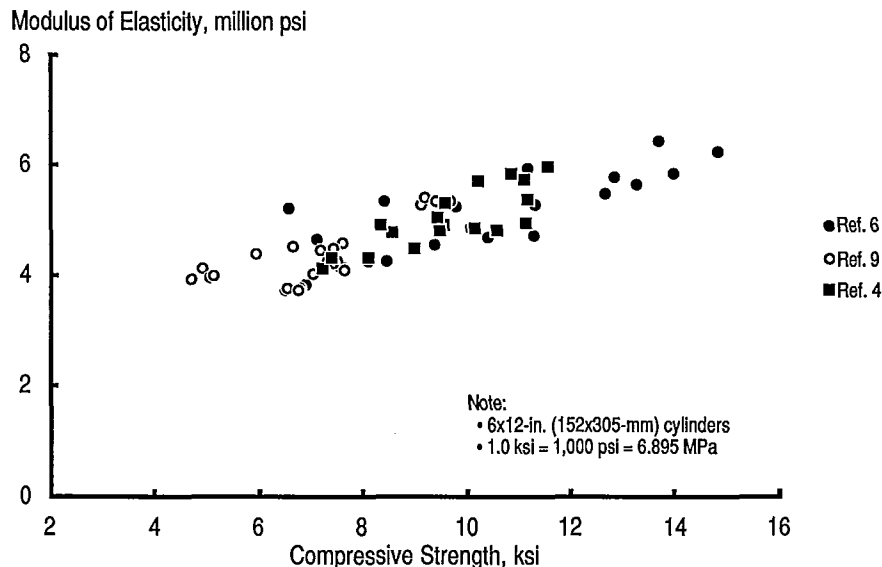


Fig. 3—Modulus of elasticity as a function of strength.

pendent deformations of the structure. Concrete design strengths ranged from 7500 to 3500 psi (51.7 to 24.1 MPa) over the height of the building.

The Lake Point Tower investigation demonstrated that it is possible to obtain reliable field measurements on a multistory structure during and after construction. It also identified the significant effects of creep and shrinkage on total deformations. Differential shortening between columns and core walls was found to increase uniformly from the first story to the midheight of the building, and then to remain relatively constant. An analytical procedure for calculation of deformations was presented that showed satisfactory agreement between calculated and measured deformations.

Subsequent to the work on Lake Point Tower, a combined laboratory and field investigation was undertaken of the time-dependent deformations of columns and walls in Water Tower Place, a 76-story, 859 ft (262 m) tall reinforced concrete building in Chicago.⁹

Laboratory tests were conducted on concretes obtained in the field to measure compressive strength, modulus of elasticity, coefficient of thermal expansion, creep, and shrinkage. Field measurements were made to evaluate vertical shortening of individual members and relative vertical displacements at se-

lected floor levels during and after construction of the building. Concrete design strengths ranged from 9000 psi (62.1 MPa) at the lower levels to 4000 psi (27.6 MPa) at the top of the building.

In addition to providing properties of high-strength concrete used in an actual structure, the Water Tower Place investigation extended the base of essential information on short- and long-term building deformations. Based on construction records that defined loading histories on instrumented columns and walls and on laboratory data on elastic properties, creep, and shrinkage, time-dependent shortening was calculated. These calculations accounted for load history, column and wall sizes, amount of reinforcement, and concrete properties. Calculated shortening compared well with measured values.

This work provided a quantitative basis for dealing with time-dependent movements in high-rise buildings and has since been extended to other buildings.^{10,11} In addition, a new publication provides an update to the Water Tower Place investigation and documents measurements continued for about 13 years after start of construction.¹²

Fire endurance

Another issue related to high-strength concrete structures that has been addressed in recent research is fire endurance. Fire tests were per-

formed on reinforced 3 x 3 x 0.33 ft (914 x 914 x 101 mm) slab specimens fabricated from conventional and high-strength concrete mixes, with and without silica fume.¹³ Commercially supplied concretes with water-cement ratios ranging from 0.28 to 0.53, with 56-day compressive strengths from 6990 to 17,490 psi (48.2 to 120.6 MPa) were used.

Specimens were fire tested for four hours and monitored for temperature rise and physical integrity. Fire endurance of all specimens were not significantly different. None of the specimens exhibited spalling of exposed surfaces.

Summary of selected properties

Fig. 1 summarizes compressive strength test results as a function of water-cementitious materials ratio from previously discussed references and from projects that will be described later. This figure confirms what by now is obvious: strength increases with decreasing water-cement ratio. However, the changes that have occurred in mix proportions over time are not directly evident.

The advent of conventional and high-range water reducers has permitted production of very low water-cement ratio concretes that are extremely workable. Supplementary cementitious materials (fly ash, slag, and silica fume) have been used

additions to cement in producing high-strength concretes. Silica fume, in particular, has been used in combination with high-range water reducers to increase achievable strength levels at very low water-cementitious materials ratios (Fig. 1).

Fig. 2 illustrates compressive strength development of high-strength concretes. For moist-cured specimens, strengths at 56 days are about 10 percent greater than 28-day strengths. Strengths at 90 days are about 15 percent greater than 28-day strengths. Mixes shown contain cement only or cement and fly ash.

While it is inappropriate to generalize such results, they do provide an overall indication of potential for strength gain at later ages. In addition, the effect of curing is evident from the curve shown for air-cured specimens, where only marginal strength increases were achieved after 28 days.

Fig. 3 summarizes modulus of elasticity test results as a function of compressive strength. These data confirm the increased stiffness obtained at higher strength levels.

High-strength concrete for bridges

Much of the work on high-strength concrete buildings has direct applicability to bridges. Generally, design requirements and economics have not yet led to the use of the same strength levels in bridges as are used in some buildings. However, there has been a growing recognition of the availability of high-strength concretes by the bridge design community. High-strength concretes are being considered to reduce dead weights (smaller sections), to permit longer spans or fewer girders for equivalent loading, and to provide improved durability.

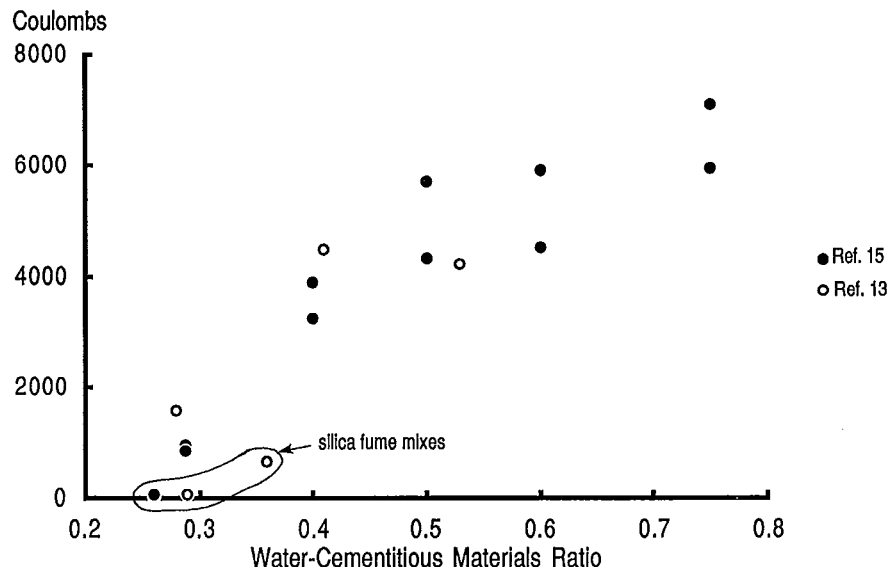


Fig. 4—Rapid chloride permeability test results.

In a study sponsored by the Federal Highway Administration, which is summarized in Reference 14, a limited investigation was made on the effect of concrete strength on span lengths. For standard precast prestressed bridge girders, increasing concrete compressive strength from 5000 to 7000 psi (34.5 to 48.3 MPa) increased span capabilities of AASHTO girders by about 15 percent.

High-strength concrete for durability

An interesting and relatively recent development has been the use of high-strength concretes where durability rather than strength is the primary design criteria. Durability of structures takes many forms, including resistance to corrosion, freezing and thawing, deicing chemicals, alkali-aggregate reactivity, chemical attack, sulfate attack, abrasion, and combination effects.

Permeability

One of the most important characteristics of durable concrete is low permeability. Low water-cement ratio, high-strength concretes have very low permeabilities relative to conventional strength concretes.

One of the problems in evaluating permeability is the difficulty of measurements for low water-cement ratio concretes. Whiting reported on a program to compare test procedures for permeability and

to establish effects of mix design variables, materials, and curing on permeability of selected concrete mixes.¹⁵

Test procedures included conventional steady-state flow tests for air and water permeability, rapid chloride permeability, helium porosity, and volume of permeable voids. Concretes tested had water-cement ratios ranging from 0.26 to 0.75, cement contents from 413 to 750 lb/yd³ (245 to 445 kg/m³), and slumps from 3.5 to 4.9 in. (89 to 124 mm). Some mixes included silica fume and high-range water reducers. Curing included a 7-day moist cure as well as simulation of cases with minimal curing. Compressive strengths ranged from 4120 to 15,100 psi (28.4 to 104.1 MPa) at 90 days (7-day moist cure).

Findings from this study:

- Confirmed the strong influence of water-cement ratio on permeability
- Confirmed the importance of adequate curing to achieve lower permeabilities, particularly at higher water-cement ratios
- Demonstrated that at water-cement ratios less than 0.3, especially when silica fume was used, concretes were virtually impermeable to water and chloride ions
- Demonstrated that rapid test procedures such as the rapid chloride permeability test (AASHTO T-277) and volume of permeable voids (ASTM C 642) can be used

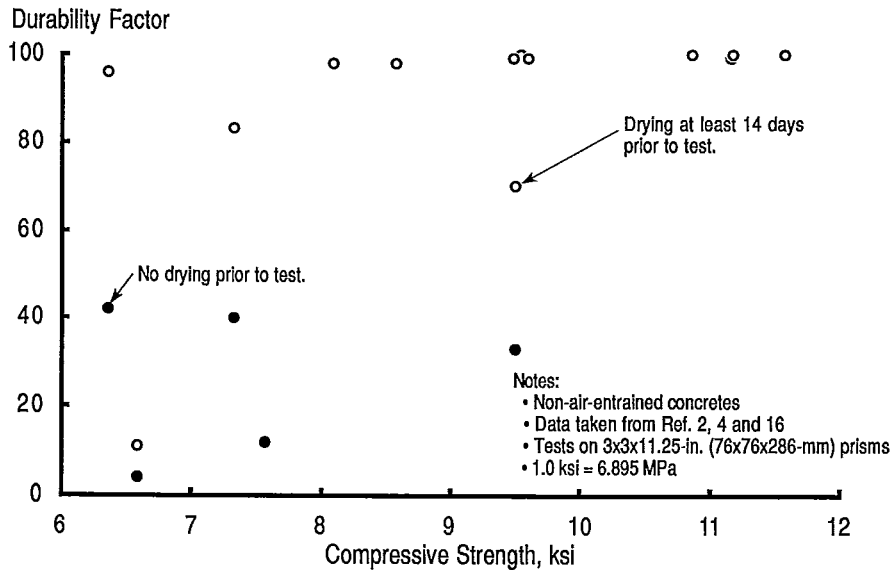


Fig. 5—Freeze-thaw test results

to estimate permeability in lieu of flow testing

Fig. 4 summarizes results of rapid chloride permeability measurements as reported in References 13 and 15. The reduction in permeability at lower water-cement ratios is readily evident. The addition of silica fume resulted in even greater reductions in permeability than would be anticipated based solely on water-cement ratio.

Freeze-thaw resistance

With many applications of high-strength concrete, freeze-thaw durability is not an issue because the concrete is used in elements that are not subject to freezing in a saturated condition. However, high-strength concretes are being used increasingly in elements such as parking or bridge decks and girders that are subject to severe freeze-thaw environments.

An important aspect related to freeze-thaw durability of high-strength concretes is entrained-air requirements. Addition of entrained air is not compatible with achieving high strengths; therefore, from the viewpoint of strength alone, it is preferable to minimize entrained air. The issue becomes whether the low permeabilities (lower probability of containing freezable water) and higher tensile strengths of high-strength concrete mixes offset the need for entrained air.

Several of the previously discussed investigations included tests on freezing and thawing and on deicer scaling. In addition to those investigations, a program was undertaken to address freeze-thaw durability specifically.¹⁶

The work included concretes with nominal 28-day compressive strengths of 6000, 8000, and 10,000 psi (41.4, 55.2, and 69.0 MPa) produced using representative commercial mixes with water reducers, high-range water reducers, and fly ash. Air contents were varied from no entrained air to 3 to 4 percent, 4 to 6 percent, and 7 to 9 percent by volume. Durability tests included rapid freezing and thawing in tap water and application of deicing agents. Some specimens were placed into testing immediately after moist curing for 28 days, while others were moist cured for 7 days and air dried for 21 days prior to exposure to freeze-thaw cycles.

The following findings were obtained from the tests:

- All nonair-entrained concretes performed poorly, regardless of curing, in both freeze-thaw and deicer-scaling tests.
- All air-entrained concretes performed satisfactorily when exposed to freezing and thawing in tap water. The only evidence of deterioration was weight loss (minor surface scaling), which decreased as nominal strength levels increased.

- For strength and air-content levels tested, the 8000 and 10,000 psi (55.2 and 69.0 MPa) concretes exhibited severe scaling in deicer tests, regardless of curing. The 6000 psi (41.4 MPa) concretes at recommended air-content levels performed satisfactorily.

Data on freeze-thaw tests reported in References 2, 4, and 16 all have indicated that air-entrained concretes perform well; however, results for nonair-entrained concretes appear inconsistent. Fig. 5 summarizes results of tests in terms of the durability factor as a function of compressive strength for nonair-entrained mixes. For this comparison, the durability factor was based on 300 cycles or the number of cycles at 60 percent of the original dynamic modulus, whichever was reached first.²

Results are divided into those in which there was no drying prior to freeze-thaw testing (solid circles) and those where at least 14 days of drying were permitted (hollow circles). The common thread in these results appears to be the level of drying after initial moist curing. Where drying was permitted, concretes generally performed better. This likely is related to the reduction of potentially freezable water during hydration and drying and to the ability of low-permeability concretes to resist resaturation under freeze-thaw conditions.

Because of uncertainties in initial curing and drying under actual field exposure conditions, the use of air entrainment is still recommended even in high-strength concretes where they are exposed to potential freezing and thawing under saturated conditions.

Data on deicer-scaling tests in References 2 and 16 indicate that nonair-entrained concretes exhibited severe scaling irrespective of strength level or conditions of curing (drying prior to tests).

ously, the higher strength air-entrained concretes included in Reference 16 did not perform well in deicer-scaling tests, and this anomaly has not been resolved. However, all tests indicate that air entrainment is required for deicer-scale resistance, even for high-strength concretes.

Current and future research

Current PCA projects cover both materials development and structural design issues. Materials work is directed toward quantifying cement characteristics that affect strength and toward mix proportions for concrete strengths in excess of 20,000 psi (137.9 MPa). Structural work is directed toward providing test data on development lengths for reinforcing steel in high-strength concrete.

Future programs will continue to address structural issues primarily related to providing data for dealing with high-strength concretes in the ACI Building Code. Work on axial load and reinforcement in columns, punching shear, ductility, shear strength of walls, shear strength of deep beams, transfer of column loads through slabs, torsion, and long-term deflections is being considered for future programs.

Research on materials is also planned for continuation, with emphasis on developing more information on properties of higher strength mixes. Additional data on strength development, modulus of elasticity, effects of cylinder size, and core-versus-cylinder strengths are needed, as is research on durability, particularly in the area of air-entrainment requirements. PCA is also working to develop joint industry programs to support nonproprietary research on high-strength concrete.

In conjunction with sponsored research programs, PCA continues to participate in activities and programs of ACI Committee 363, High-Strength Concrete; the Concrete Materials Research Council;

and the Reinforced Concrete Research Council, which are active in formulating and supporting high-strength concrete research.

It is apparent that concrete technology and use of high-strength concretes have come a long way since the 1950s. Design strengths achievable with commercially available concrete have almost tripled in some areas of the country. If industry needs continue at a similar pace in the future, it will provide a major challenge to concrete materials technology and engineering.

References

1. Klieger, Paul, "Early High-Strength Concrete for Prestressing," *Proceedings, World Conference on Prestressed Concrete*, San Francisco, July 1957, pp. A5-1-A5-14. Also, *Research Department Bulletin* No. 91, Portland Cement Association.
2. Klieger, Paul, "Some Aspects of Durability and Volume Change of Concrete for Prestressing," *Research Department Bulletin* No. 118, Portland Cement Association, Skokie, Nov. 1960, 12 pp.
3. Perenchio, William F., "An Evaluation of Some of the Factors Involved in Producing Very High-Strength Concrete," *Research and Development Bulletin* No. RD014.01T, Portland Cement Association, Skokie, 1973, 7 pp.
4. Perenchio, William F., and Klieger, Paul, "Some Physical Properties of High-Strength Concrete," *Research and Development Bulletin* No. RD056.01T, Portland Cement Association, Skokie, 1978, 7 pp.
5. Whiting, David, "Effects of High-Range Water Reducers on Some Properties of Fresh and Hardened Concretes," *Research and Development Bulletin* No. RD061.01T, Portland Cement Association, Skokie, 1979, 15 pp.
6. Kaar, P. H.; Hanson, N. W.; and Capell, H. T., "Stress-Strain Characteristics of High-Strength Concrete," *Douglas McHenry International Symposium on Concrete and Concrete Structures*, SP-55, American Concrete Institute, Detroit, 1978, pp. 161-185. Also, *Research and Development Bulletin* No. RD051.01D, Portland Cement Association.
7. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-83)," American Concrete Institute, Detroit, 1983, 111 pp.
8. Pfeifer, Donald W.; Magura, Donald D.; Russell, Henry G.; and Corley, W. G., "Time Dependent Deformations in a 70 Story Structure," *Designing for Effects of Creep, Shrinkage, and Temperature in Concrete Structures*, SP-27, American Concrete Institute, Detroit, 1971, pp. 159-185.
9. Russell, H. G., and Corley, W. G., "Time-Dependent Behavior of Columns in

Water Tower Place," *Douglas McHenry International Symposium on Concrete and Concrete Structures*, SP-55, American Concrete Institute, Detroit, 1978, pp. 347-373. Also, *Research and Development Bulletin* No. RD052.01B, Portland Cement Association.

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

11. Daniel, J. I., "Investigation of High-Strength Concrete at the Chicago Mercantile Exchange—Report to ACI Chicago Chapter, Chicago Committee on High-Rise Buildings, Concrete Reinforcing Steel Institute, Material Service Corporation, and Structural Engineers Association of Illinois," *Construction Technology Laboratories*, Skokie, Jan. 1983, 6 pp.

12. Russell, H. G., and Larson, S. C., "Thirteen Years of Deformations in Water Tower Place," *ACI Structural Journal*, V. 86, No. 2, Mar.-Apr. 1989, pp. 182-191.

13. Shirley, Scott T.; Burg, Ronald G.; and Fiorato, Anthony E., "Fire Endurance of High-Strength Concrete Slabs," *ACI Materials Journal*, V. 85, No. 2, Mar.-Apr. 1988, pp. 102-108.

14. Rabbat, Basile G. and Russell, Henry G., "Optimized Sections for Precast, Prestressed Bridge Girders," *Research and Development Bulletin* No. RD080.01E, Portland Cement Association, Skokie, 1982, 10 pp.

15. Whiting, D., "Permeability of Selected Concretes," *Permeability of Concrete*, SP-108, American Concrete Institute, Detroit, 1988, pp. 195-222.

16. Whiting, D., "Durability of High-Strength Concrete," *Concrete Durability—Katharine and Bryant Mather International Conference*, SP-100, American Concrete Institute, Detroit, 1987, pp. 169-186.

Selected for reader interest by the editors.

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Research and Application of High-Strength Concrete

10,000 psi Concrete

by James E. Cook

In the summer of 1981, the question was asked if concrete could be produced that would meet a specified designed strength of 10,000 psi (69 MPa) for InterFirst Plaza, the proposed Dallas Main Center Project. The response was that a definite yes would require an extensive laboratory investigation.

With approximately a two-year lead time, an investigation of local materials was started in August 1981 to determine material properties and establish mix proportions for the 10,000 psi (69 MPa) high-strength concrete. During an 18-month period a total of 100 laboratory trial batches were made, 92 yd³ (70.3 m³) of plant-batched concrete

was produced for field testing, and approximately 4,000 test specimens were made.

The structural system used in the 72-story building significantly advances the technology of tall building construction. At 921 ft (280.7 m), the building is Dallas' tallest and among the tallest in the world. The ratio of height to structural width of 7.24:1 makes it the most slender of the 12 tallest buildings. The World Trade Center Towers in New York City, in comparison, have height to width ratios of 6.49:1.

In tall slender buildings, the major design problem is to provide sufficient rigidity under lateral wind loading. The large height-to-width

ratio of the InterFirst Plaza meant that an unusually efficient structure was needed to give the slim building the necessary stiffness.

The owner wanted a building totally uninterrupted by perimeter columns, which ruled out the old reliable tube design. The architect and engineers met this requirement by supporting the entire weight of the structure on 16 exterior columns set 20 ft (6.1 m) back from the glass curtain wall. The setback was large enough that the architect could put an entire row of offices between the curtain wall and the column line. The exterior columns are located on 30 ft (9.1 m) centers and vary in size from 6 x 6 ft. (1.8

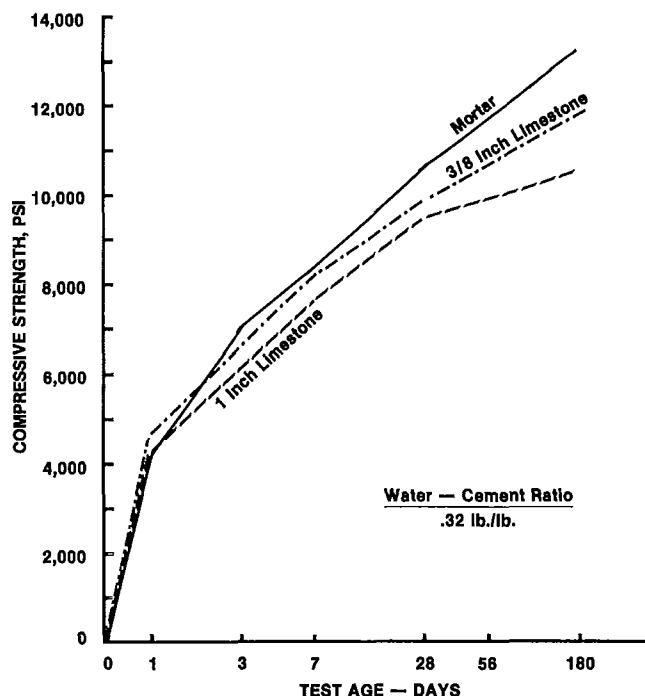


Fig. 1 — Compressive strength comparisons of mortar and concretes with two sizes of coarse aggregate at a constant water-cement ratio.

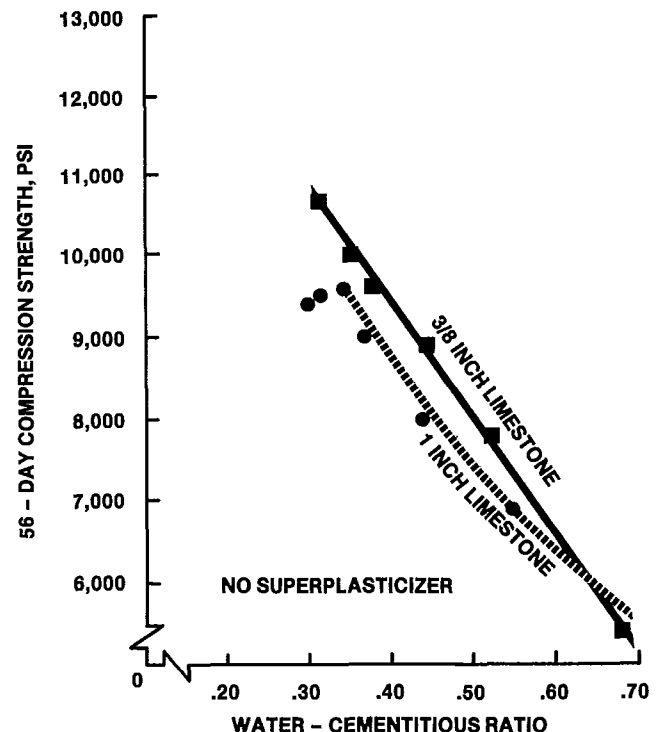


Fig. 2 — Relationship of water-cementitious ratio and compressive strength for two sizes of coarse aggregate.

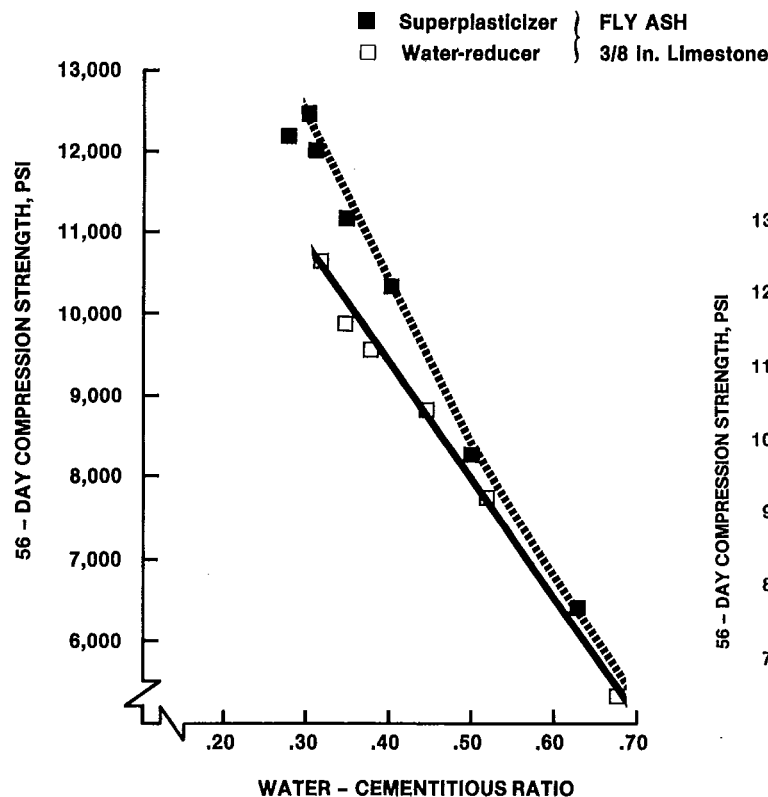


Fig. 3 — Relationship of water-cementitious ratio and compressive strength with and without a high-range water-reducing admixture for 3/8 in. aggregate concrete.

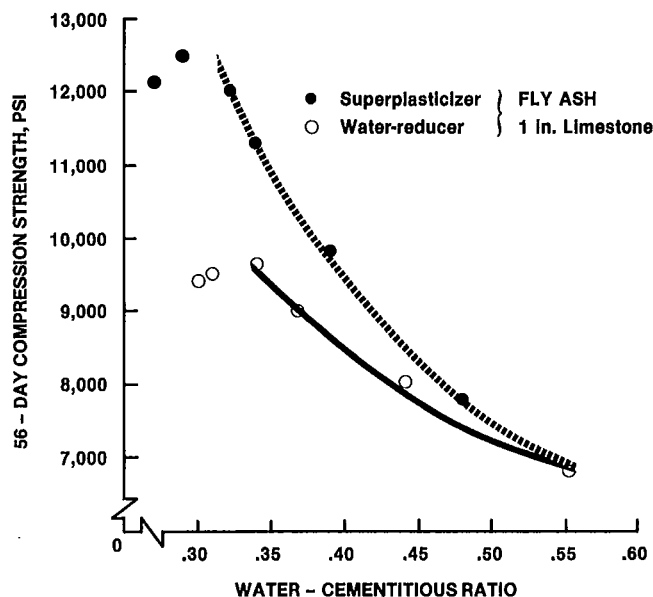


Fig. 4 — Relationship of water-cementitious ratio and compressive strength with and without a high-range water-reducing admixture for 1-in. aggregate concrete.

x 1.8 m) to 8 x 8 ft. (2.4 x 2.4 m).

The engineers first attempt at a structural design was an all-concrete, fairly conventional, shearwall structure. Although the frame was rigid and efficient, the contractor claimed that the concrete work would add seven months to the construction schedule.

The next attempt was an all steel building, but the steel weights came out to be a heavy and expensive 40 lb/ft² (18.1 kg/m²).

The solution was a combination of the two. The slim structure was designed to have very high rigidity under wind loading for minimum dynamic response in extreme winds. This was achieved by concentrating all vertical gravity loads on the 16 exterior columns using 10,000 psi (60 MPa) concrete up through the 62nd floor level.

The largest columns at the lower levels are reinforced with 96 No. 18 bars and steel columns embedded in the concrete columns provide the means for connecting the interior steel with the exterior columns. The steel weight for this combination structure was a respectable 25 lb/ft² (11.3 kg/m²).

The principal concern of the engineer in the design of this structure was the stiffness of the concrete; a high modulus of elasticity was needed. The rigidity of steel is five to seven times that of concrete per unit volume, but the cost is 20 to 30 times the cost per unit volume of cast-in-place reinforced concrete. The high-strength concrete with a high modulus of elasticity will provide six times as much stiffness per dollar in a column as steel.

Construction of the 72-story building and 16-story parking garage began in March 1983. The first columns using 10,000 psi (69 MPa) concrete were cast on July 13, 1983; the structure was topped out on July 3, 1984. A total of 84,700 yd³ (64,762 m³) of concrete was used in the project; 20,560 yd³ (15,720 m³) of the total was 10,000 psi (69 MPa) concrete and 1,800 yd³ (1,376 m³) was 8000 psi (55.2 MPa) for column footings.

The data presented are the results of extensive testing performed during the development of the high-strength concrete prior to the construction startup. Evaluation of compressive strength tests of sam-

ples of concrete taken in the field also are presented to show conclusively that 10,000 psi (69 MPa) concrete can be produced consistently while using locally available materials to meet the architectural, engineering, and economic demands of these structures.

Mix proportioning

Since stiffness of the concrete was the principal concern of the engineers in their design scheme, the selection of coarse aggregate was given careful consideration. As the characteristics of the coarse aggregate will significantly influence the structural properties of the concrete, a comprehensive study was made to determine the largest maximum-size aggregate that could be used for the 10,000 psi (69 MPa) designed strength concrete.

In general, the smallest size coarse aggregate produces the highest strength for a given water-cementitious material ratio as shown in Fig. 1 through 5. However, this data clearly shows that compressive strengths in excess of 10,000 psi (69 MPa) are feasible using a 1-in.

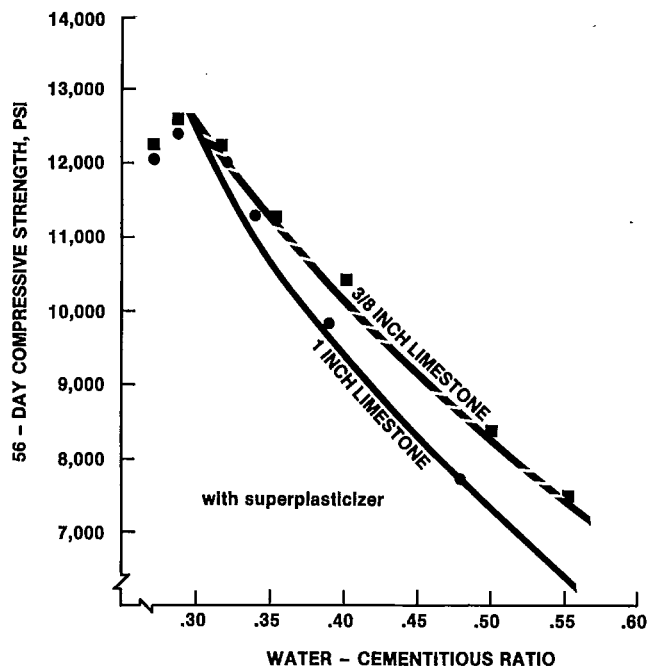


Fig. 5 — Comparison of relationships of water-cementitious ratio and compressive strength for concretes using two aggregate sizes and a high-range water-reducing admixture.

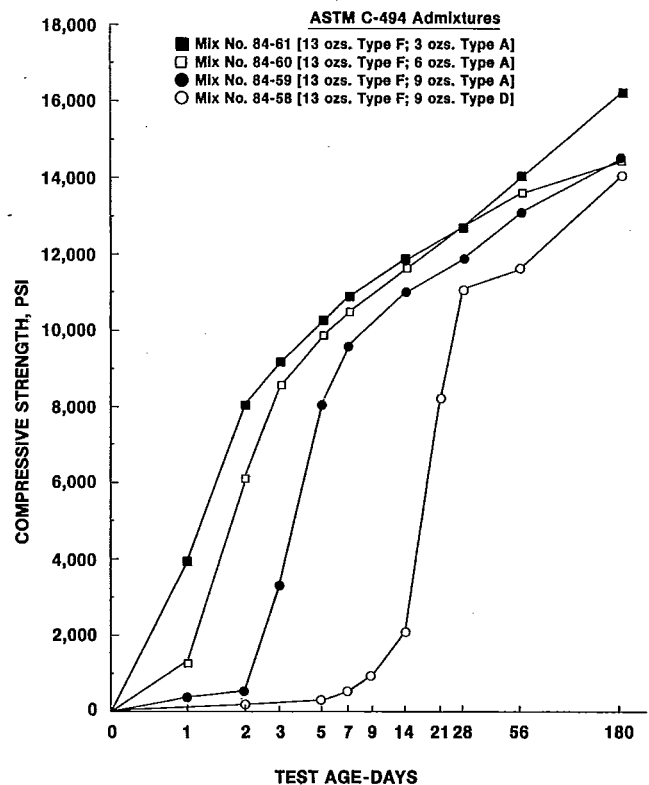


Fig. 6 — Effect of varying dosage rates of a normal/retarding water-reducing admixture on the strength development of concrete.

(25.4-mm) maximum-size aggregate when the mixture is proportioned with a high-range water-reducing admixture.

It can be seen in Fig. 4 that with a 1-in. (25.4-mm) coarse aggregate the concrete strength is increased approximately 3000 psi (20.7 MPa) by the additional dispersion of cement particles at a 0.30 water-cementitious ratio. This strength enhancement appears to occur only when the water-cementitious ratio drops below 0.50 for both aggregate sizes (Fig. 3 and 4).

The performance of the admixtures is influenced by the particular cement and other cementitious material used. For high-strength concrete, the actual dosage rate of the admixture will most likely be higher than the manufacturer's recommended dosage. Thus, the optimum dosage of an admixture or combination of admixtures should be determined by trial mixes using varying amounts of admixture.

Fig. 6 shows the effect of strength development of concretes when the dosage rate of normal and retarding water-reducing admixtures is increased while the high-range water-

reducing admixture dosage is held constant. In this case, Mix No. 84-61 contained optimum dosages of admixtures.

This investigation was made to study the effects on strength when the normal set and retarding admixtures were accidentally overdosed due to the equipment malfunction or batchman error, which is not uncommon in the ready-mixed concrete industry. Considering the results, the decision was made to discontinue the use of a retarding water-reducing admixture in conjunction with the high-range water-reducing admixture and additional precautions were taken to insure accurate dispensing of admixture.

Where slump loss may be a problem, the desired slump can be successfully restored by redosing with a high-range water-reducing admixture. A second dosage after a period of time generally restores the slump and results in increased strengths at nearly all test ages. This proved to be a great advantage for the high-strength concrete placed during the hot summer months in Dallas. Table 1 shows the effect of redosing with a high-range water-

reducing admixture during construction.

For the concrete batches given, the concrete batched at 12:32 p.m. received a normal dosage of 42 oz/yd³ dispensed at the central mix batch plant and 60 oz/yd³ added on arrival at the jobsite. The concrete batched at 8:27 a.m. received the same normal dosage as described above.

However, a breakdown of placement equipment prevented the concrete from being unloaded for a period of approximately 45 minutes, after which time the concrete received an additional dosage of 94 oz/yd³ (the amount required to restore the concrete to the desired placement consistency). It can be seen that the additional dosage of high-range water-reducing admixture resulted in increased strengths at all test ages when compared to the batch with a normal dosage.

Mix proportions selected for use on this project are given in Table 2. Also shown are mix properties, strength results, and modulus of elasticity values for the plant-batched trial batches. For this project, 6 x 12 in. Documents

Table 1 — Effect of redosing with high-range water-reducing admixture

Admixture dosage, oz/yd ³	102	196
Compressive strength, psi		
1 day	5,984	6,126
3 days	7,803	9,526
7 days	8,825	11,080
28 days	11,038	12,990
56 days	11,415	14,070
180 days	12,445	14,934
Time batched	12:32 p.m.	8:27 a.m.
Time sampled	12:55 p.m.	9:30 a.m.
Date sampled	July 26, 1983	July 26, 1983

Table 2 — Mix proportions and test results of 10,000 psi concrete field test mixes

Mix design number	GHS-119 (%in. Limestone)	GHS-120 (1-in. Limestone)
Materials/yd ³		
Type I cement, lb	606	602
Class C fly ash, lb	253	251
Stone - ASTM C-33, lb	1772	1994
Sand - ASTM C-33, lb	1151	1017
Water, lb	279	250
Type A admixture - ASTM C494, fl oz (4 fl oz/cwt)	34.3	34.2
Type F admixture - ASTM C494, fl oz (10 fl oz/cwt)	85.9	85.3
Mix properties		
Slump, in. (before/after addition of Type F admixture)	1 1/10	2 1/9
Unit weight, lb/ft ³	150.4	152.4
Water-cementitious ratio, lb/lb	0.33	0.29
Air content, percent volume	1.3	0.8
Fly ash, percent weight	29.5	29.4
Concrete temperature	84 F	84 F
Ambient temperature	67 F	67 F
Compressive strength, psi (moist cured)		
1-Day	1900	1560
3-Day	7099	6669
7-Day	9228	9277
28-Day	11522	11236
56-Day	12376	12448
90-Day	12901	13401
Flexural strength, psi (3rd point)		
3-Day	945	875
7-Day	1125	1005
28-Day	1445	1286
56-Day	1450	1395
Modulus of elasticity, $E \times 10^6$, psi		
7-Day	5.49	6.05
28-Day	6.33	6.49
56-Day	6.50	6.71
90-Day	6.71	6.94

Table 3 — Required average strength

Specified compressive strength f'_c , psi	Required average compressive strength f'_{cr} , psi
Less than 3000 psi	$f'_c + 1000$
3000 to 5000	$f'_c + 1200$
over 5000	$f'_c + 1400$

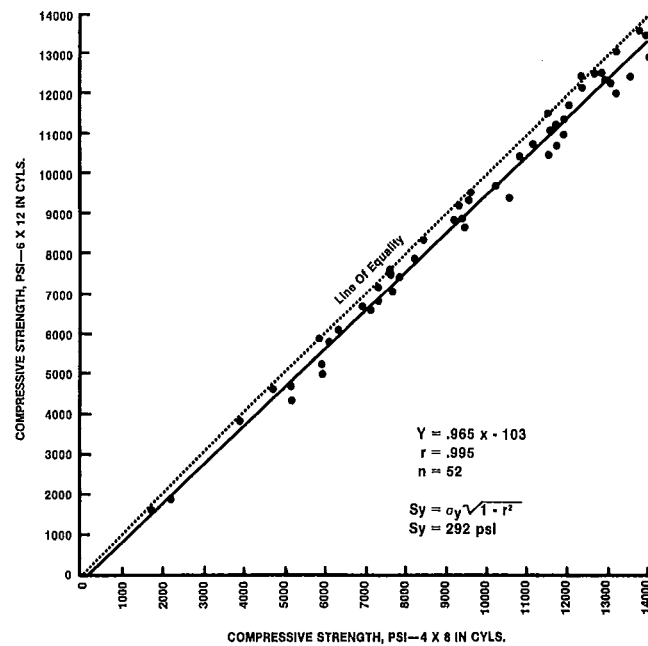


Fig. 7 — Relationship between the compressive strength of 4 x 8 in. cylinders and 6 x 12 in. cylinders.

Table 4 — Statistical evaluation of strength data for 10,000 psi strength tests

Test age	28 days	56 days	180 days
Commercial laboratory			
Number of test results, n	422	415	381
Average strength $\bar{X}$, psi	11,149	12,068	13,397
Standard Deviation s , psi	855	850	791
Coefficient of variation V , percent	7.7	7.1	6.0
Design strength met f'_{cm} , psi	10,003	10,929	12,333
Mode strength $Mo.$, psi	11,390	12,328	13,747
Skewness $\sqrt{\beta_1}$	-0.5645	-0.6120	-0.8846
Kurtosis β_2	4.095	3.676	3.962
Probability of a test falling below f'_c (area of normal curve)		1 in 132	
Probability of a test falling below f'_c (area of skewed curve)		1 in 1200	
Producer quality control laboratory			
Number of test results n	139	139	138
Average strength $\bar{X}$, psi	11,192	12,082	13,462
Standard deviation s , psi	678	682	724
Coefficient of variation V , percent	6.1	5.7	5.4
Design strength met f'_{cm} , psi	10,283	11,168	12,492
Mode strength $Mo.$, psi	11,152	12,109	13,584
Skewness $\sqrt{\beta_1}$	+0.1185	-0.0797	-0.3377
Kurtosis β_2	3.218	3.174	3.410
Probability of a test falling below f'_c (area of normal curve)		1 in 900	
Probability of a test falling below f'_c (area of skewed curve)		1 in 1400	

cylindrical specimens were used for compressive strength measurements. However, 4 x 8 in. (101.6 x 203.2 mm) cylindrical specimens were made during the laboratory study to determine the relationship between the two specimen sizes. Data plotted in Fig. 7 indicates that 4 x 8 in. (101.6 x 203.2 mm) cylinders exhibit approximately 5 percent higher strengths than 6 x 12 in. (152.4 x 304.8 mm) cylinders.

Regardless of specimen size, the size used to determine mix proportions should be consistent with the size specified for acceptance. Also, it should be noted that good success was experienced on the use of single-use rigid plastic molds on this project. A commercially available high-strength sulfur capping material was also successfully used, with cap thicknesses maintained at $\frac{1}{16}$ to $\frac{1}{8}$ in. (1.59 to 3.18 mm).

Modulus of elasticity

At present, there are two empirical formulas that serve as a guide for use when the static modulus of elasticity cannot be determined by test. They are the ACI 318-83 Building Code¹ formula

$$E_c = 57,000 \sqrt{f'_c} \text{ psi} \quad (1)$$

where $w_c = 145 \text{ lb/ft}^3$

and the ACI 363² Committee on High Strength Concrete formula

$$E_c = 40,000 \sqrt{f'_c} + 1.0 \times 10^6 \text{ psi} \quad (2)$$

where $w_c = 145 \text{ lb/ft}^3$

Using these formulas, the engineer could assume a modulus of elasticity value in the range of 5.0 to 5.7 x 10⁶ psi (35 to 39 GPa) for the 10,000 psi (69 MPa) design strength concrete depending on the formula used.

Since the elastic modulus of concrete was a critical design consideration for the tall slender structure, the engineer asked us to determine the elastic modulus of proposed concrete mixtures using the ASTM C 469 standard test method. Table 2 shows 56-day modulus of elasticity values of 6.5 to 6.7 x 10⁶ psi (45 to 46 GPa) for proposed concrete mixtures.

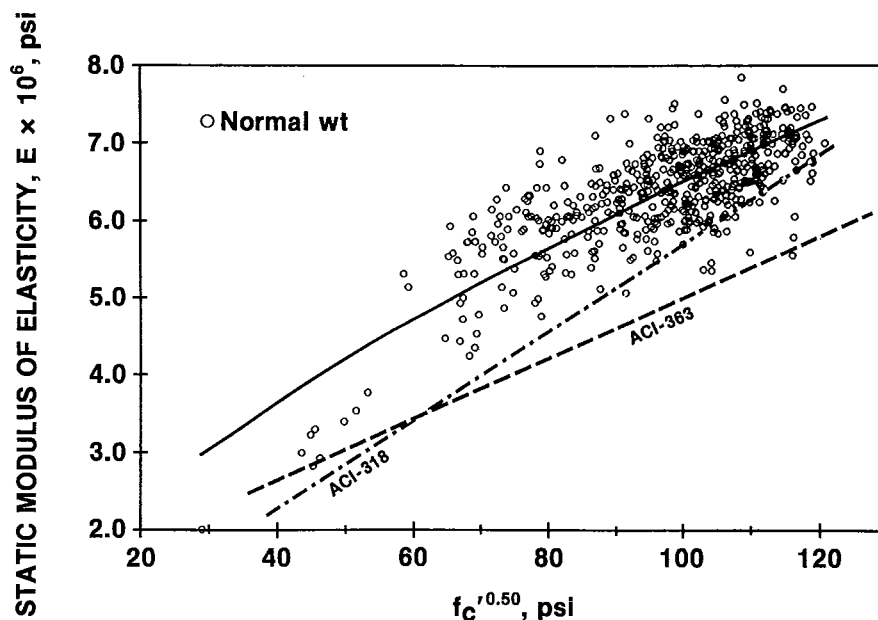


Fig. 8 — Modulus of elasticity versus concrete strength for normal weight concretes.

Fig. 8 gives a comparison of experimentally determined elastic modulus test values collected over the past seven years with those predicted by the expressions given in ACI 318 and ACI 363. The concretes tested contain aggregates from South Carolina, Tennessee, Texas, and Arizona. Aggregate sizes vary from $\frac{3}{8}$ to 1 in. (9.5 to 25.4 mm) and consist primarily of crushed limestones, granites, and native gravels.

It should be noted that tests of North Texas limestones have produced elastic modulus values of 8.8 to 10.0 x 10⁶ psi (61 to 69 GPa), with unconfined compressive strengths of rock core specimens ranging from 20,000 to 24,000 psi (138 to 165 MPa).

The power curve fit equation for experimental data presented is

$$E_c = w_c^{2.55} f'_c{}^{0.315} \text{ psi} \quad (3)$$

where $w_c = 151 \text{ lb/ft}^3$

The ACI formulas underestimate the modulus of elasticity for the majority of concretes observed. However, there are some granites in the southeast that are capable of producing concrete compressive strengths in excess of 9000 psi (62 MPa) but have modulus of elasticity of concrete values between 3.5 and 4.0 x 10⁶ psi (24 and 28 GPa).

A regression curve is still an estimate and the scatter of data could put your particular data far below or above the average.

If a structural property of concrete is part of an important design consideration, that property should be determined for the particular materials that will be used in the mix design just as is required by the ACI Building Code to verify a proposed mix design for compressive strength. The modulus of elasticity has become a more important consideration now with high-strength concrete becoming commonplace and verification requirements should be detailed in the body of the project specification.

Fig. 9 clearly demonstrates the influence of coarse aggregate on the modulus of elasticity of concrete. Concrete containing a 1 in. (25.4 mm) maximum-size aggregate at the same stress level had a modulus of elasticity of 6.05 x 10⁶ psi (42 GPa) and strain at maximum stress of 0.0020 in./in. Therefore, it can be concluded that, in general, the larger the amount of coarse aggregate with a high elastic modulus in a concrete mixture, the greater would be the modulus of elasticity of concrete.

Concrete produced for high-strength concrete research ten years ago was proportioned in such a manner as to minimize the

the coarse aggregate on the modulus of elasticity of the concrete. For example, low coarse aggregate contents and high sand and paste contents.

Required average strength

The ACI Building Code requires concrete mixtures to be proportioned based on field experience or by laboratory trial batches. To meet these requirements, the average compressive strength of field test results exceeds the specified designed compressive strength f'_c by an amount sufficiently high to minimize the number of test results below the specified strength value. The requirements for overdesign provide average strength values of not less than 1.34 standard deviations above specified strength.

When the concrete producer chooses to select mixture proportions based upon field experience, the required average strength (f'_{cr}) used as the basis for selection of concrete proportions should be the larger value calculated from the following equations

$$f'_{cr} = f'_c + 1.34s \quad (4)$$

or

$$f'_{cr} = 0.90 f'_c + 2.33s \quad (5)$$

where s = standard deviation

Eq. (5) is a modified version of Eq. (4-2) of ACI 318 since the code was established on the basis of concrete strengths in the range of 3000 to 6000 psi (21 to 41 MPa). To date, project specifications for high-strength concrete have modified the acceptance criterion to allow not more than one in 100 individual tests to fall below 90 percent of the specified strength.

When the concrete producer chooses to select concrete proportions on the basis of laboratory trial batches or when data is not available to establish a standard deviation, the required average strength f'_{cr} used for selection of concrete proportions is determined from Table 4.3.2.2 given in the ACI Building Code. The table is partially reproduced as Table 3.

A problem with this table is that the code assumes the average

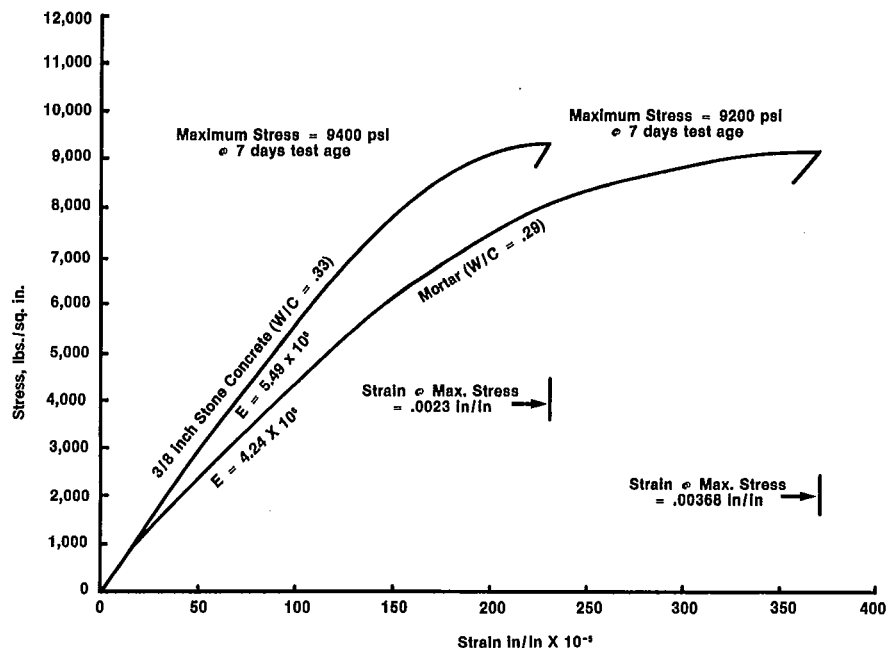


Fig. 9 — Stress-strain relation of high-strength mortar and concrete with $\frac{3}{8}$ in. maximum-size aggregate at constant rate of load.

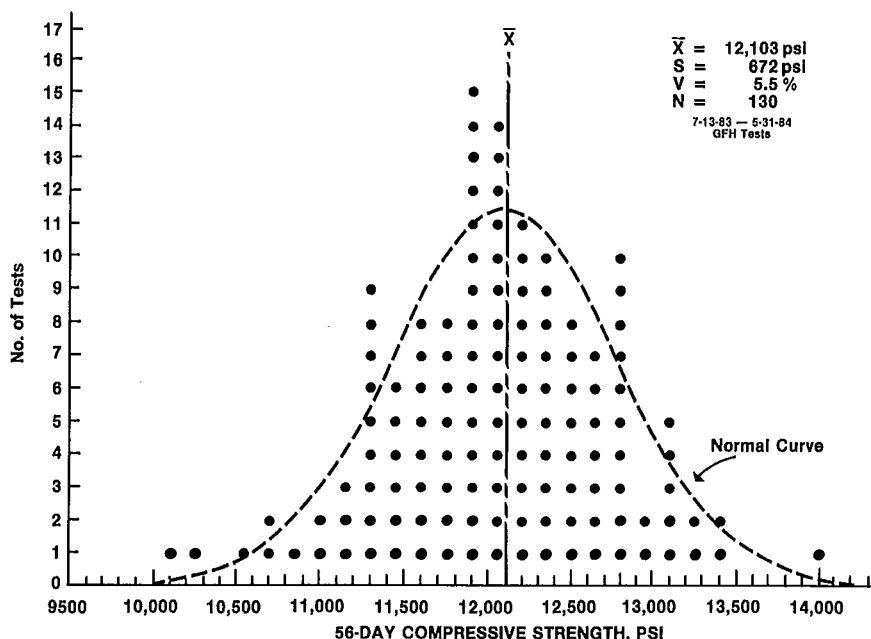


Fig. 10 — Frequency distribution of compressive strength test results for 10,000 psi designed strength concrete.

Table 5 — Summary of statistical measures of high-strength concrete

Test age	Commercial laboratory					Producer laboratory				
	n	$\bar{X}$, psi	s , psi	$\sqrt{\beta_1}$	β_2	n	$\bar{X}$, psi	s , psi	$\sqrt{\beta_1}$	β_2
3 days	386	7,373	980	-2.5900	17.250	112	7,463	689	-0.5100	6.000
7 days	421	9,059	721	-1.1500	7.410	139	9,063	596	+0.1900	3.830
28 days	419	11,149	855	-0.5645	4.095	139	11,192	678	+0.1185	3.218
56 days	411	12,068	850	-0.6120	3.676	139	12,082	682	-0.0797	3.174
180 days	377	13,397	791	-0.8846	3.962	138	13,462	724		

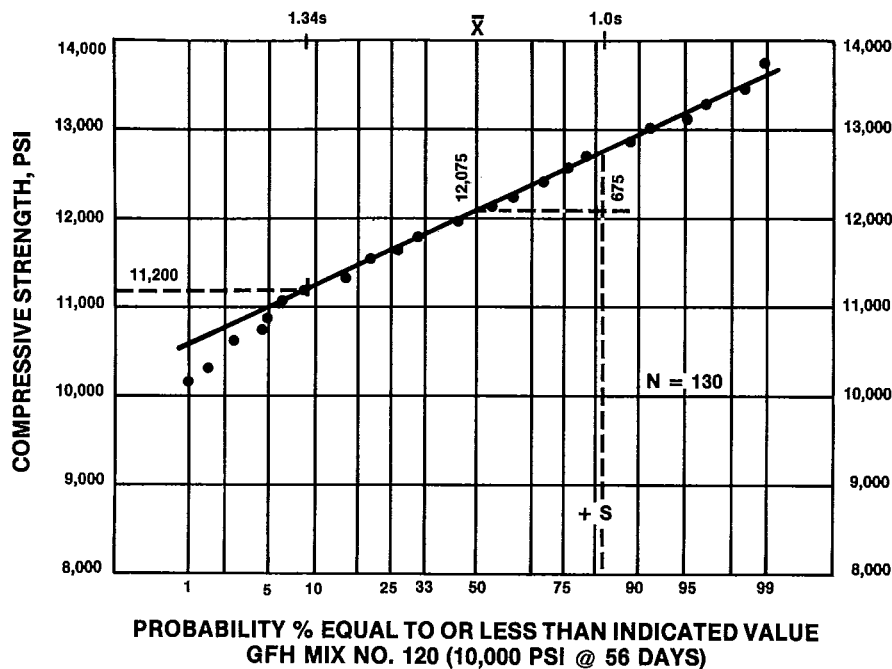


Fig. 11 — Graphic analysis of 10,000 psi concrete test results.

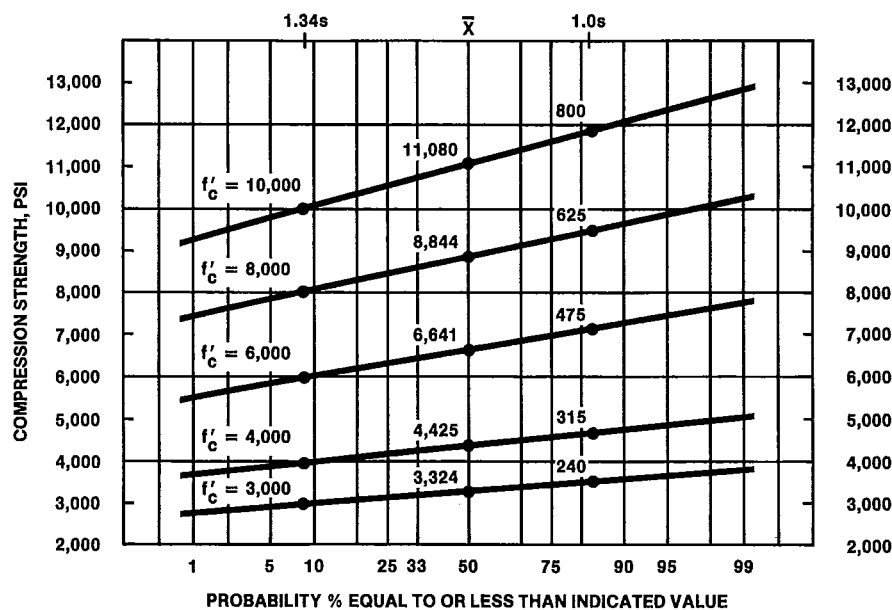


Fig. 12 — Graphic presentation of frequency distribution for various designed strength levels of concrete.

Table 6 — Standards of concrete control

Class of operation	Coefficient of variation for different control standards, percent				
	Excellent	Very good	Good	Fair	Poor
Overall variation					
General construction testing	<8	8 to 10	10 to 12	12 to 15	>15
Laboratory trial batches	<4	4 to 6	6 to 8	8 to 10	>10
Within-test variation					
Field control testing	<3	3 to 4	4 to 5	5 to 6	>6
Laboratory trial batches	<2	2 to 3	2 to 3	3 to 4	>5

strength of field production concrete will equal the strength of a laboratory trial batch. This assumption is not realistic since many factors can influence the variability of strengths and strength measurements in the field.

Experience has shown that approximately 90 percent or less of laboratory controlled strengths are attained under field conditions. Therefore, when selecting mixture proportions based on laboratory trial batch data, the following equation should be used

$$f'_{cr} = f'_c + 1400/0.90 \quad (6)$$

Considerations other than compressive strength may influence the selection of mixture proportions. For this project, other considerations included modulus of elasticity, creep and drying shrinkage, and heat of hydration.

Mixture proportions for the 10,000 psi (69 MPa) design strength concrete were based on an average laboratory strength of 13,250 psi (91 MPa) at 56-days test age. This strength is five percent higher than the required average strength f'_{cr} determined by Eq. (6) ($f'_{cr} = 11,400/0.90 = 12,667$ psi). The higher average strength was dictated by the high modulus of elasticity requirement.

Also, note that 56-day compressive strength test results of field test mixes given in Table 2 are 93 percent of the average laboratory trial batch strength.

Analysis of compressive strength test results

Statistical procedures in accordance with ACI 214-77³ were used to evaluate the performance of 10,000 psi (69 MPa) design strength concrete used in the project. The data presented are test results (average of two 6 x 12 in. [152.4 x 304.8 mm] specimens) of concrete delivered from one batch plant from July 13, 1983, through June 25, 1984. The acceptance of test results was based on a 56-day test age criterion.

The analysis of strength test results of concrete is presented in Table 4. The data presented are results of tests performed by the commercial laboratory hired by the owner and the concrete producer's quality control laboratory.

Table 7 — Column core strengths versus 6 X 12 in. cylinders*

Test age	7-Days		28-Days		56-Days		180-Days		1-Year	
Maximum size stone	¾ in	1 in	¾ in	1 in	¾ in	1 in	¾ in	1 in	¾ in	1 in
Compressive strength of 6 X 12 in. cylinders, psi										
Field-cured	8,596	8,139	10,177	10,204	10,775	10,542	11,514	11,546	12,444	11,772
Moist-cured	9,228	9,277	11,522	11,236	12,376	12,448	13,852	13,776	14,060	13,951
Compressive strength of cores, psi										
West face	9,407	9,312	11,118	10,743	11,598	10,964	12,970	12,400	15,080	13,775
Middle	8,660	8,959	9,674	9,724	9,833	9,656	11,635	10,720	13,404	13,003
East face	9,180	9,706	10,584	10,575	10,756	10,603	11,626	11,589	14,088	13,695
All cores	9,083	9,326	10,459	10,347	10,729	10,408	12,077	11,570	14,190	13,490
Cores/6 X 12 in. moist-cured cylinders, percent										
West face	102	101	97	96	94	88	94	90	107	99
Middle	94	97	84	87	79	78	84	78	95	93
East face	99	105	92	94	87	85	87	84	100	98
All cores	98	101	91	92	87	84	87	84	101	97

*Reported strengths are average of two specimens.

analysis of test data reported here, we believe the quality control techniques commonly used for low and moderate strength concretes may not necessarily be appropriate for all high-strength concretes.

In particular, this data does not support the common assumption that the standard deviation for a particular mix will remain nominally constant and independent of the test age and compressive strength. In addition, it is inappropriate to implicitly assume that even large amounts of test data are normally distributed.

As a practical matter, specifiers and producers of concrete typically assume test results are normally distributed with the classic bell-shaped curve as illustrated by the histogram of 56-day test results for producer laboratory data shown in Fig. 10. If the data is not symmetrical around the mean (i.e., it is skewed), or if distribution of the data is too peaked or flat (i.e., significant kurtosis exists), the data may not be normally distributed and any analysis presuming a normal distribution may be misleading rather than informative.

The term skewness refers to a lack of symmetry in the test data. Kurtosis refers to the peakedness or flatness of the frequency distribution curve.

If a frequency plot of data is positively skewed, the bulk of data is to the left of the sample mean (i.e., the sample mean strength is larger than the mode strength). If the data is negatively skewed, the bulk of data

is to the right of sample mean strength (i.e., the sample mean strength is smaller than the mode strength).

The skewness and kurtosis for a particular set of data may be evaluated analytically by

$$Sk = \sqrt{\beta_1} = \frac{\sum (X_i - \bar{X})^3/n}{s^3} \quad (7)$$

$$Ku = \beta_2 = \frac{\sum (X_i - \bar{X})^4/n}{s^4} \quad (8)$$

Using Eq. (7), a perfectly symmetrical normal distribution would have a value $\sqrt{\beta_1}$ equal to zero. Skewness values were calculated for the concrete test presented in Table 4. It is interesting to note that skewness values are higher on commercial laboratory data and values seem to increase with an increase in test age.

Interpretation of the significance of skewness is made somewhat simpler if we understand that the following relationship is approximately correct in slightly skewed distributions

$$\sqrt{\beta_1} = \frac{2(\bar{X} - Mo.)}{s} \quad (9)$$

Then, $\sqrt{\beta_1}$ is equal to twice the distance from the mean to the mode, expressed in units of the standard deviation. The commercial laboratory frequency distribution of 56-day test results has a $\sqrt{\beta_1}$ value of -0.6120 . This means that the mean strength is smaller than the mode, and that they are separated by an

amount equal to $\frac{1}{2}$ (0.6120) or 0.306 standard deviations.

Knowing the value of $\sqrt{\beta_1}$ is useful in that we can use it with tables of the areas under the skewed curve just as we use tables of areas under the normal curve to determine the probability of failure. It can be seen in Table 4 that correcting for the skewness in the distribution of test data significantly decreases the probability of a test falling below the 10,000 psi (69 MPa) design strength at 56-days test age. Had the skewness been positive rather than negative, the chances of a test falling below the design strength would have increased.

The term β_2 calculated through Eq. (8) represents a common measure of kurtosis. In the normal curve, β_2 equals 3. When it is greater than 3, the curve is more peaked than the normal curve, and when it is less than 3, the curve has a flatter top than the normal curve.

When $\sqrt{\beta_1}$ and β_2 differ very markedly from 0 and 3, respectively, we know that the frequency distribution curve is not normal. Then we have to turn to some other kind of frequency curve to describe the data. Table 5 presents a summary of five statistical measures of frequency distribution for the 10,000 psi (69 MPa) high-strength concrete at various test ages.

It can be seen that there are marked differences in frequency distribution with nearly identical mean strengths. The notable difference at early test ages should be considered when comparing pro-

portioned to meet a minimum strength requirement of 3000 to 4000 psi (21 MPa to 28 MPa) at a 2- to 3-day test age.

Fig. 11 is a graphic presentation of the same test results used in the histogram shown in Fig. 10. The results are graphed on probability-versus-arithmetic paper. Arithmetic divisions are used for the y-axis and the normal probability scale for the x-axis. If the sum of the area under the normal curve is plotted from left to right, the result is the summation of probability curve. If a straight line develops in the plotting, the data are said to have normal distribution. If a straight line does not develop in the plotting, the distribution is not normal.

It can be seen that the bulk of data points form a straight line and it is possible to determine graphically the mean strength and standard deviation from the mean as illustrated. The design strength met by the test data can also be determined by a perpendicular line erected at 1.34 standard deviations below the mean. The mean strength, standard deviation, and design strength met is nearly identical to calculated values shown for producer laboratory test results in Table 4.

Standards of concrete control

It should be noted that the adjectives used to describe the standards of control are prescribed according to the magnitude of the standard deviation as defined by ACI 214. I have consistently noted a higher level of standard deviation values for strengths above 4500 psi (31 MPa). Thus, the standard of control assigned by ACI 214 may not be a fair evaluation for the higher strength concretes.

The coefficient of variation is a unitless standard deviation expressed as a percentage of the average strength. This value will be less affected by the magnitude of the strengths obtained and is more useful in comparing the degree of control between higher strength concretes and lower strength concretes.

A good illustration of this statement is given in Figure 12. All of the concrete strength levels presented have the same degree of control. This is due to the fact that all

concretes have the same coefficient of variation (7.2 percent) and the same probability of a test falling below the specified strength (1 in 11, or 9 percent). Thus, a 10,000 psi (69 MPa) designed strength concrete with a standard deviation of 800 psi (6 MPa) has the same degree of control as 3000 psi (21 MPa) designed strength concrete with a standard deviation of 240 psi (2 MPa).

Table 6 lists suggested values of coefficient of variation for standards of control assigned by ACI 214 for all strength levels.

High-strength concrete core strengths

There has been little published about the relationship between strength of high strength cores and standard 6 x 12 in. (152.4 x 304.8 mm) cylinders. Therefore, an investigation was made to determine if the 85 percent of specified strength acceptance criterion specified in the ACI Building Code was applicable to high-strength concrete.

Columns measuring 30 x 30 in. (762 x 762 mm) were cast using field trial batches of concrete as shown in Table 2. The results of this investigation are presented in Table 7. The results indicated that the 85 percent criterion would be applicable to high-strength concrete. Also, this study confirms the belief that job-cured specimens do not give accurate measurements of in-place strength. The reason for lower strengths in the middle portion of columns is probably due to the measured high temperature rise (i.e., 100 F [38 C]) for the high-strength concrete mixtures.

Summary and conclusions

1. Locally available materials used in the production of ready-mixed concrete can produce compressive strengths in excess of 10,000 psi (69 MPa).

2. It is feasible to produce strengths in excess of 10,000 psi (69 MPa) using a 1-in. (25.4-mm) maximum-size aggregate when the mixture is proportioned with a high-range water-reducing admixture.

3. In general, the larger the amount of coarse aggregate with a high elastic modulus in a concrete mixture, the higher the modulus of elasticity of the concrete.

4. Quality control techniques commonly used for low to moderate strength concretes may not necessarily be appropriate for very high strength concretes.

5. A laboratory trial batch program seems to be the most effective method of determining concrete properties and establishing mixture proportions for high-strength concrete. Specimen size used for strength measurements of laboratory trial batches should be consistent with the size used by the specifier for acceptance on the project.

6. The coefficient of variation expression is a more useful term in comparing the degree of control between high-strength concrete and low to moderate strength concretes.

7. The availability of data on higher strength concretes requires a reassessment of current equations to determine their applicability with higher strength concretes.

Acknowledgments

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References

1. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-83)," American Concrete Institute, Detroit, 1983, 111 pp. Also, *ACI Manual of Concrete Practice*, Part 3.
2. ACI Committee 363, "State-of-the-Art Report on High Strength Concrete," (ACI 363R-84), American Concrete Institute, Detroit, 1984, 48 pp. Also, *ACI Manual of Concrete Practice*, Part 1.
3. ACI Committee 214, "Recommended Practice for Evaluation of Strength Results of Concrete (ACI 214-77)," American Concrete Institute, Detroit, 1977, 14 pp. Also, *ACI Manual of Concrete Practice*, Part 2.

Selected for reader interest by the editors.

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The Production and Delivery Of High-Strength Concrete

by Nathan L. Howard and David M. Leatham*

High-strength concrete is becoming increasingly popular in the greater Seattle area. Testing labs and consultants in the area claim that because of the high quality of aggregates available, concrete with a strength of 20,000 psi (140 MPa) should not be difficult to produce. Results of many laboratory tests have proven this to be true, but much of this research has gone on without fully considering the ready-mixed concrete supplier. They are very important to the team effort that is necessary to produce high-strength concrete, and therefore must be involved in all aspects of the job, including the planning.

Northwest contractors and engineers are working to advance the concrete industry by developing and specifying concrete with ultra high compressive strengths. This statement is as relevant now as it was in 1960 when the Washington State Highway Department specified 6000 psi (41 MPa) concrete for prestressed girders, allowing the WSDOT girders to be among the thinnest in the country. In 1961, Seattle's mono-rail track girder was specified with 7000 psi (48 MPa) prestressed concrete, and the Port of Seattle has

been specifying 8000 psi (55 MPa) concrete piles for many years.

In the 1960s, the same concrete constituents were being shipped to both the precast yards and the concrete ready-mixed companies, but city building codes were restricting strengths for cast-in-place concrete to 4000 psi (27.5 MPa). 564 lb (256 kg) of cement per yd^3 (0.76 m^3) of concrete was typical for the 3000 psi (20.5 MPa) concrete used then for much of Seattle's construction.

In the 1980s, the same 564 lb (256 kg) mix is used to produce 5000 psi (34.5 MPa) concrete. The current 28-day average strength is 6230 psi (43 MPa), probably just as it was in 1960. The only thing that changed besides the definition of high-strength was the specifications.

In the last five years, several building projects in downtown Seattle have specified high-strength concrete. The 76-story Columbia Center has reinforced structural columns of 9500 psi (65.5 MPa) concrete. The 30-story Century Square used 10,000 psi (69 MPa) concrete and the 12-story First & Stewart Building has 11,000 psi (76 MPa) concrete in its columns.

The structural engineer for the Bellevue Place project, Cary Kopczynski, designed 10,000 psi (69 MPa) cast-in-place concrete columns as a cost savings feature, as well as to provide additional tenant lease space in the building. He

claimed that some engineers add up to 8 percent extra steel to add compressive strength to the columns.

Citing an example of alternative column design on a high rise, Kopczynski pointed out that by specifying a 2000 psi (14 MPa) greater concrete strength, he eliminated 20 #11 reinforcing bars for a net savings of \$400 per column. He also reported that by increasing the concrete compressive strength from 5000 psi (34.5 MPa) to 10,000 psi (69 MPa) when designing for gravity loads, the cross sectional area of the column is cut by nearly half.

Doug Canfield, Editor of Pacific Building & Engineer, said in an article on high-strength concrete, "Tenants, in fact, are a large consideration in the economics of using high-strength concrete. The tenant/concrete equation goes like this: Tenants provide lease income. High-strength concrete provides smaller columns and more leasable space. More space, more tenants, more income."

Turner Construction's Frank Anderson stated of the 58-story Two Union Square Building in Seattle, which used 14,000 psi (96.5 MPa) concrete in its columns, "A typical steel-framed 58-story building would have 23 to 27 lb (10 to 12 kg) of steel per ft^2 (0.09 m^2) in it — this building will have only 12 lb (5.5 kg) of steel per ft^2 (0.09 m^2)."

In addition to the cost savings feature,

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Keywords: high-strength concretes; mixing; mix proportioning; placing; quality assurance; quality control; ready-mixed concrete.

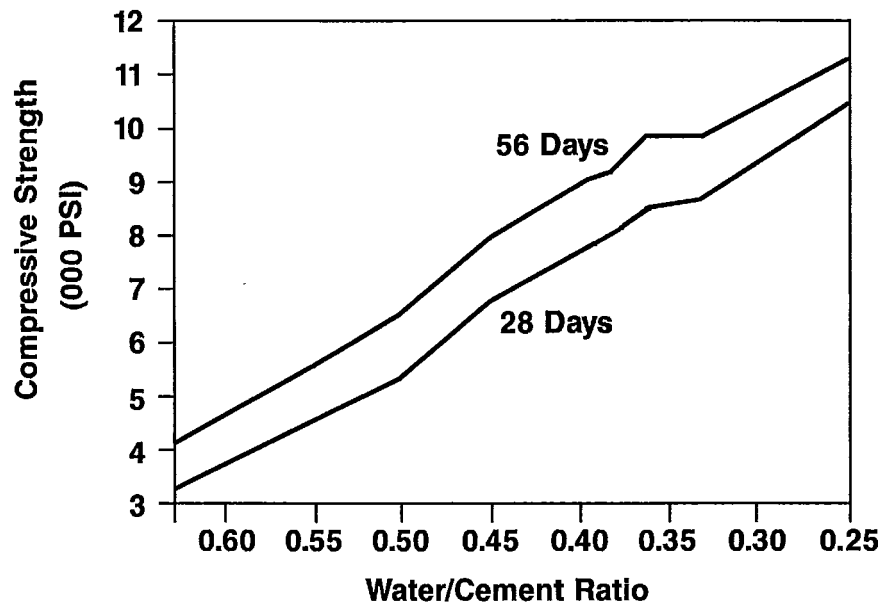


Fig. 1—High-strength w/c curves (mix contains 20 percent fly ash).

of concrete over steel, it is said that the Two Union Square Building has more leasable space than any building in Seattle.

The engineers for the Two Union Square Building, Skilling Ward Magnusson Barkshire, added to economic advantages a unique stiffening design against sway and earthquake accelerations. The engineers originally wanted to specify a 7.2 million psi (49.6 million kPa) modulus of elasticity, but decided upon a more conventional specification of compressive strength when compression tests from trial batch and mock-up tests reached 19,000 psi (131 MPa).

Testing

Tests were later run under the direction of Dr. Weston Hester (University of California at Berkeley) to verify the modulus of elasticity specimens cast on the jobsite. He explained that the state-of-the-art stress-strain equipment used for the tests measure strain and load measurements every one-half second. The data acquisition system plots and computes the stress/strain as the specimen is loaded by computer controlled rate of 35 psi (241 kPa) per second. Modulus of elasticity values of actual job samples have been as high as 7 million psi (48.3 million kPa) and greater.

These specification requirements for very-high-strength concrete have

not surprised ready-mixed concrete suppliers, at least not those who have been providing concrete to the high-rise marketplace for the last few years.

Some concrete producers have chosen to approach the demand for high-strength concrete with proprietary practices and mix proportions. Others have suggested certain mixes and mixing procedures to be approved later by the engineer of record for each project. In either case, the producer enters into the project as a member of the construction team responsible for his part in the production and delivery of very-high-strength concrete.

Most ready-mixed companies considering this type of work have a quality control organization, including experienced personnel and laboratory facilities capable of formulating trial batches to investigate the materials available to them in the design of high-strength concrete. Admixture companies and cement and fly ash suppliers can offer a great deal to the ready-mixed companies in the way of resources of expertise and testing facilities.

The prudent producer takes advantage of all resources available to compliment his technical staff. The producer needs to understand why concrete fails in compression so he may achieve the highest possible concrete strengths using the materials available to him.

There are three areas of compressive strength failure: in the mortar, in the mortar-aggregate interface, and in the aggregate. By simply reducing the water-cement ratio, a stronger mortar will be produced. Fig. 1 shows a water-cement ratio curve from actual trial batch data showing the relationship of water-cement ratios to strength.

Some researchers contend that water-cement ratios less than 0.30 are only theoretical values. They claim that effective water-cement ratios (the ratio calculated from the mass of cement particles that are actually hydrated) are somewhat greater. This value is obviously difficult to calculate precisely and until this can be better defined, mix designers must consider that each pound (kg) of water in the mix will decrease the concrete compressive strength by some degree.

Selection of Materials Cement

Presently, there is no set of scientific criteria that specifies the perfect cement for high-strength concrete. Most specifications select Type I or Type II ASTM C 150 as a base criteria, and leave it to the concrete producer to make the choice of cements through trial and error methods. Obviously the application of previous studies is helpful, where it is known that admixtures work most effectively with

higher values of C_3A . A cement should be tested with that parameter in mind for added workability and water reduction in the concrete.

If silica fume and/or fly ash is to be used, a cement that has the best pozzolanic reaction should be considered for testing, since strength at later ages is often an important characteristic in high-strength concrete. Uniformity is essential. Thus, once the brand of cement has been selected it should be used throughout the entire job. Monitoring its performance is as critical as taking concrete cylinders.

Aggregate

The ideal coarse aggregate for high-strength concrete must exhibit very high resistance to crushing; have a high modulus of elasticity, have a surface that creates a good bond to the mortar, and have a particle size gradation that minimizes the required mixing water and adds to the workability of the fresh concrete.

The first three characteristics involve complex testing equipment not readily available to most ready-mixed producers. However, there are many tests listed in ASTM C 33 that are available on a local level and tell a great deal about the strength-producing capabilities of aggregate.

Basis testing apparatus can be used to research the proper gradation, along with obtaining the highest possible unit weight of combined aggregates for maximum concrete density. Equal importance needs to be placed on the physical characteristics of the fine aggregate, just as with the coarse aggregate, such as particle shape, fineness modulus, and gradation. The large proportion of fines produced by high cement factors, and often pozzolanic materials, requires sharp angular sand, coarse gradation, and a fineness modulus of greater than 2.8.

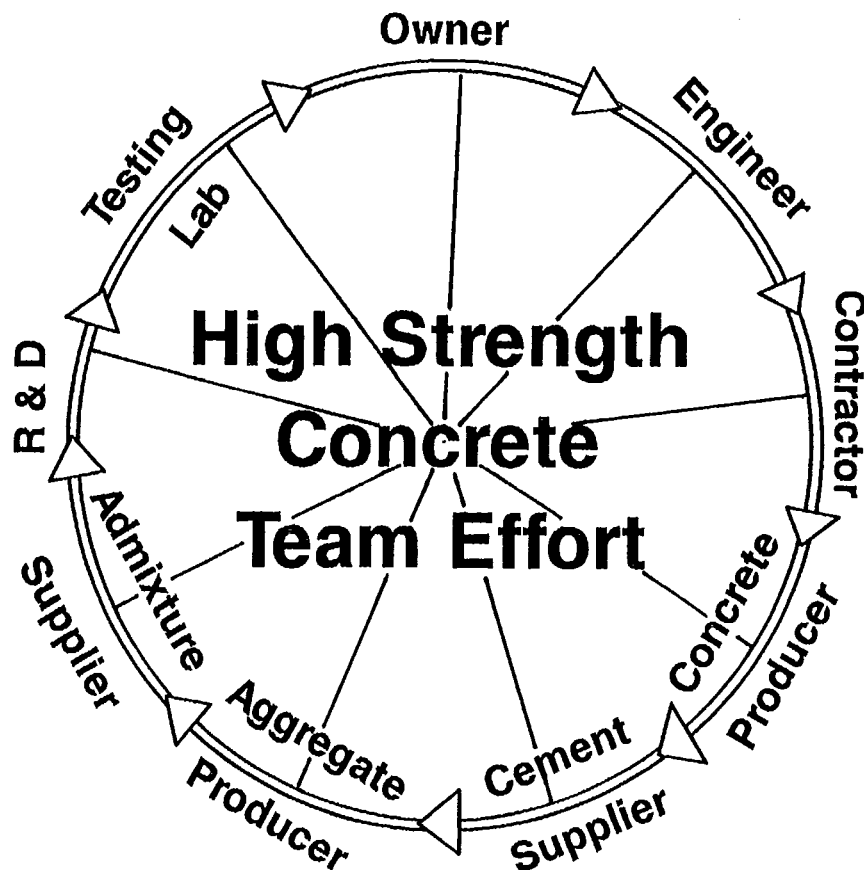


Fig. 2—High-strength concrete team effort.

Chemical admixtures

Chemical admixtures should meet the requirements of ASTM C 494. The compatibility of the admixtures with the choice of cement is a very important consideration so that no slump losses or any other undesirable effects in the concrete are created.

The combination of different types or even different brands of admixtures should not be ruled out when researching high-strength concrete. For instance, where flowable concrete is desired, a Type D retarder with a high-range water reducer may give the best water reduction and add to the extension of set time for better placability.

Mineral admixtures

Fly ash and silica fume vary in their strength-producing properties and significant research needs to be done to compare their compatibility with various cements. Quality control procedures with mineral admixtures are just as important as those used in the production of cement. The concrete producer needs to be

aware of any property changes in the material coming into the plant, and, in the case of silica fume, that the equipment used to store and dispense the material is functioning properly.

Because one of the main factors contributing to high-strength concrete is the reduction of the water-cement ratio, it is important to consider pozzolans when selecting a mix proportion. Fly ash and silica fume will reduce the mixing water demand, while adding to long-term strength; decreased permeability; increased chemical resistance; reduced bleeding; and other plastic concrete advantages.

Fly ash requires less mix water because of its particle shape. Silica fume admixtures, which come in several formulations, contain a high-range water reducer and other proprietary ingredients that aid in the reduction of mix water. Both of these materials react with the free lime in cement to form a new cementitious compound, calcium silicate hydrate.

Condensed silica fume is superior in its strength-producing ability.

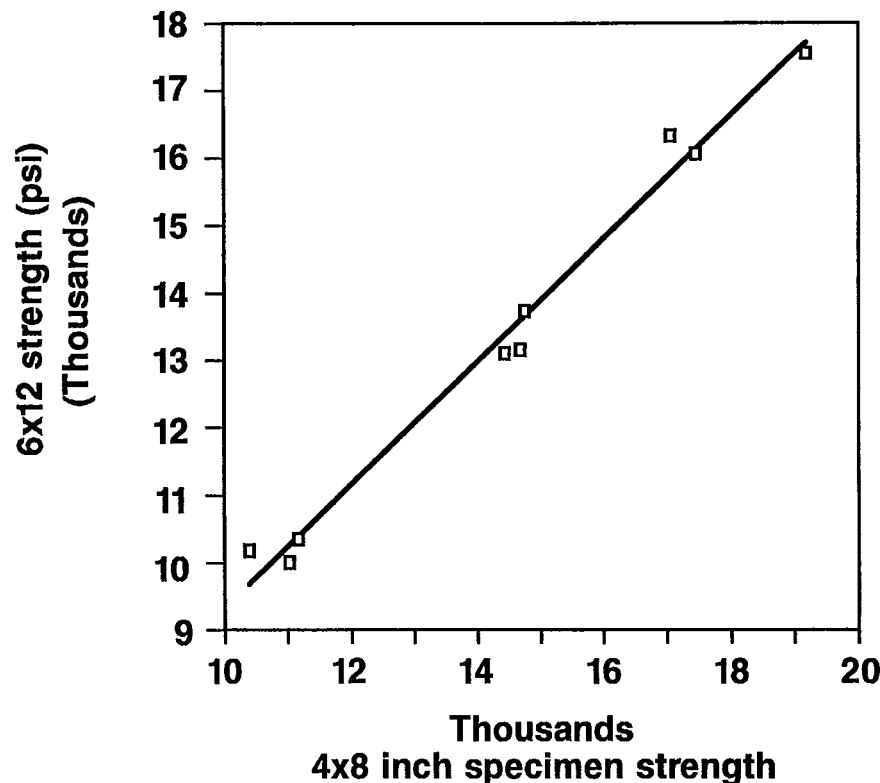


Fig. 3—Specimen size versus strength curve.

due to the surface area of 98,000 ft²/lb (20,000 m²/kg) as opposed to 102 ft²/lb (500 m²/kg) for fly ash. In addition, silica fume has an amorphous silica content of 93 to 98 percent, which may be as much as twice that present in available fly ash.

Quality assurance

A comprehensive quality assurance program is critical when producing high-strength concrete. In addition to preparing and regularly practicing his own quality control program, the ready-mixed producer must be a part of the construction team that implements a project manual of procedures, of which his becomes a part. Fig. 2 shows a listing of various members of the construction team, all of whom play a key role in producing high-strength concrete.

The quality assurance team can be made up of many members, but all involved must be willing to share in the responsibility with the other team members to the extent that their work pertains to the project. The strength of the whole program

depends largely on separate quality control programs implemented by each team member.

Each quality assurance program needs to include personnel training, procurement, material manufacture and handling, records, calibration, surveillance, and other pertinent information. Each program, before it becomes a part of the project quality assurance manual, needs to be reviewed and approved by the quality assurance manager, who may be the owner or his designate.

Quality control procedures

Before writing detailed procedures to be included in the quality assurance manual, the engineer, owner, contractor, testing lab, and concrete producer should meet to outline the parameters of the project and try to anticipate as many of the problems that might arise as possible.

It is also important to realize that every project is unique and that each has its own special needs. For example, the 7500 psi (52 MPa) concrete needed for the mat foundation on the Gateway Tower proj-

ect recently begun in Seattle is entirely different from the 7500 psi (52 MPa) mix used in the walls of the same building.

All aspects of the placing operation should be thoroughly researched before they actually occur. By conducting a full-scale mock-up, problems can be identified in time to enable further research to be done. It is important to have the best possible solution ready before the need arises, and that all parties agree that it is the best solution. Inevitably with projects of this nature, there is a great deal of anxiety and anything that can be done to relieve this anxiety will be beneficial to the entire project.

Mock-ups

For the Two Union Square Project, mock-ups were staged at the ready-mixed plant on two separate occasions. Variables such as pumpability, flowability, set times, temperature gain, slump loss, and strength characteristics were observed.

Several different types and combinations of admixtures were tried using a standard mix proportion that was previously selected based on nearly two years of laboratory testing. The concrete production facility, equipment, and personnel were also tested to rate their ability to produce high-strength concrete. Several other studies were conducted during the mock-up to aid in the preparation of the quality assurance manual.

At the first mock-up, several experiments relating to the compressive strength cylinders were conducted. These included comparative tests between 4 x 8 and 6 x 12 in. (10 x 20 and 15 x 30 cm) cylinders, cast in both plastic and steel molds.

These cylinders were either rod-ded, vibrated, or filled without compaction effort to determine the effect of the type of consolidation. Several cylinders were deliberately abused both immediately after cast-

ing and during transport to the testing lab. Cylinders were also cast at both the pump and truck discharges to determine the effect, if any, that the pressure and friction in the line had upon the slump and compressive strength.

Another consideration was the type of curing that these cylinders should receive: ambient cure, moist room, or saturated lime bath. Also, an attempt was made to identify the capping system that would yield the best results.

As a result of these tests, it was determined that although they were more sensitive to abuse, the 4 x 8 in. (10 x 20 cm) cylinders cast in steel molds and rodded by hand yielded the highest and most consistent results, while still being comparable to the standard 6 x 12 in. (15 x 30 cm) specimens (Fig. 3).

It was also decided that for optimum curing, the specimens were to be placed immediately after molding into a temperature-controlled saturated lime bath on the job. Both ends of the cylinders were ground smooth and plane before testing. The variations due to capping were minimized by this procedure.

While the first mock-up determined which test procedures were appropriate and that the basic mix was acceptable, several questions still lingered. This project required the mix to be pumped 58 stories, so a second mock-up was staged to see if more pumpable mixes could be found. Larger doses of first and second generation melamine-based superplasticizers were compared with their naphthalene counterparts to see if the slumps could be safely increased, and if there were any differences in their performance after being pumped under high pressure over a long distance.

While there were subtle differences, they were judged to be minimal and it was agreed that any remaining questions could be answered in the laboratory. Tests were conducted on the basic mix using differing types and brands of admixtures under conditions designed to imitate the effect of temperature

extremes that might occur as construction progressed through the seasons.

Quality assurance manual

The data accumulated from the mock-ups was combined with the information previously assembled to produce a joint quality assurance manual. The contractor, testing lab, and the concrete producer each submitted their own quality assurance/quality control procedure covering their respective aspects of the work. Next, a series of meetings was held to produce a final document that would assure the best possible product. This document was submitted to the owner, engineer, and city for review, and it then became part of the contract documents.

The quality assurance manual defined a quality team consisting of representatives of each of the participating companies to provide a forum to reconcile any quality related disputes. This teamwork was an essential part of making the document work because all involved were committed to the proper execution of the document.

Perhaps the most important aspect of the program was the section outlining the training and qualifications of the personnel involved in the production of the high-strength concrete. The drivers, batchmen, testing personnel, etc., received personalized training and instruction so they knew what problems to look out for, not only in their specific areas, but in other areas as well. By keeping the job personnel informed and involved, a genuine team spirit was established and it carried through to other aspects of the job.

As the project progressed, adjustments were made to the quality assurance program as needed, the most significant being the inclusion of a flow board to measure the mix consistency as well as the slump cone. Because of the small margin for error when using slumps in excess of 10 in. (25.5 cm), every drop of water needed to be accounted for. Tests for moisture content of both the sand and aggregate were

performed for every 50 yd³ (38 m³) of concrete produced. If a delay of 30 minutes or more occurred, the plants were drained of material and refilled to keep the moisture consistent within the bins.

Prior to each day's placement, water sprinkling systems were used to keep the surface of the sand and aggregate piles cool and moist. In addition to the slump controls at the plant, consistency was the key to control at the jobsite.

The high-strength mix required a second dosage of superplasticizer at the jobsite to achieve its final slump. The timing of the addition of the admixture to the truck was critical to achieve and maintain an extremely high slump without segregation or premature slump loss. As soon as the truck arrived at the jobsite, it was inspected for consistency and the superplasticizer was added as needed. Periodic checks for slump were made on any waiting trucks and subsequent admixture additions were made as required.

Before attempting to produce and place the high-strength concrete, it was imperative that enough tests were performed on the mix and any alternate proportions to be able to predict how they would behave under a variety of conditions. The key to success was being able to anticipate problems and have solutions readily available before they became critical to the project.

The current high-strength mixes are the result of an evolution within the industry, and current testing, batching, and placing practices need to evolve to keep up with this expanding technology. The rewards can be exaggerated, and so can the risks. Producing high-strength concrete has become a reality with the implementation of good quality assurance programs. When present day standards and specifications have lagged behind the current technology, they must be modified to be able to specify and produce quality high-strength concrete.

Proposed Mechanism for Strength Enhancement

A New Mineral Admixture for High-Strength Concrete

by A. M. Rosenberg and J. M. Gaidis

During research leading to the development of a new concrete admixture, insight was obtained on how silica fume enhances the strength of concrete. Silica fume does not densify concrete in the usual sense; it enhances the paste-aggregate bond, which is generally the weakest part of concrete. This comes about by a reduction in the amount of bleed water produced. Chemical and physical evidence is shown to prove these conclusions.

A new liquid admixture to improve the strength of concrete has been developed. Over the years the use of liquid admixtures has increased because of their beneficial effect on dispensing, handling, and mixing concrete. For these reasons, as well as the reduced cost of shipping and the elimination of the dust hazard, Grace has introduced this new mineral admixture. The product is a dispersion of silica fume (microsilica, or SF) in water with additives tailored to provide stability and

Keywords: aggregates; bleeding (concretes); bonding; calorimeters; chlorides; drying shrinkage; ettringite; high-strength concretes; mineral admixtures; permeability; porosity; silica fume; water-reducing agents; x-ray diffraction.

proper viscosity. Some of its physical properties are described in Table 1.

During the development work, data were collected that help to explain the mechanism for strength enhancement with this new product. Since the mechanism of how pozzolanic admixtures improve concrete strength is not clearly understood, findings in this also are presented. The influence of the silica fume on the paste-aggregate interface was found to be the dominant factor in the development of increased strength.

Use of silica fume in concrete and mortar

To realize the optimum benefit from this silica fume slurry, a high-range water reducing agent (HRWRA) must also be used to improve flow and reduce water content. Table 2 shows the benefits of adding the superplasticizer at the beginning, at the end, or splitting the addition at the beginning and the end of the batching operation. The importance of using a naphthalene-based HRWRA with limited molecular-weight range can be ob-

served by noting the effect of $w/(c + SF)$ on flow and strength. From this it can be seen that delaying the addition of the HRWRA enhances its effectiveness, as one would expect.

Strength, porosity, and permeability

Fig. 1 shows typical results of concrete with a type I cement using a 705 lb/yd³ (418 kg/m³) mix with ¾-in. (19-mm) maximum-size aggregate. The water-cement ratio (0.38 ± 0.01) was controlled with the superplasticizer to give a slump of 5 to 7 in. (127 to 178 mm). Changes in mix proportions can, of course, result in higher or lower strengths

Table 1 — Properties of new liquid admixture for high strength

Silica content of solids	>92 percent by weight
Viscosity* at 25 C	1000 CP at 0.6 rpm 200 CP at 60 rpm

*Typical viscosity measured with a Brookfield LVT Viscometer with a #3 spindle using a well stirred sample.

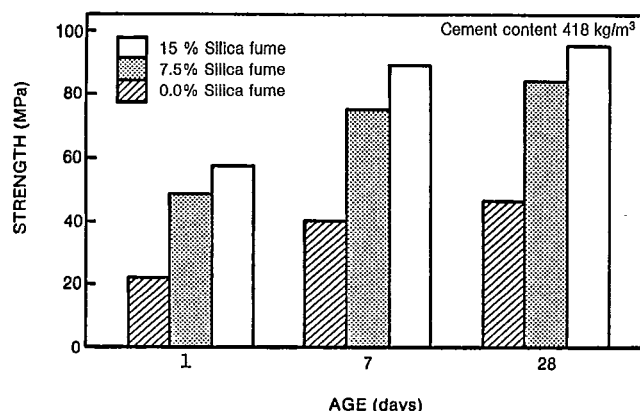


Fig. 1—Average compressive strengths.

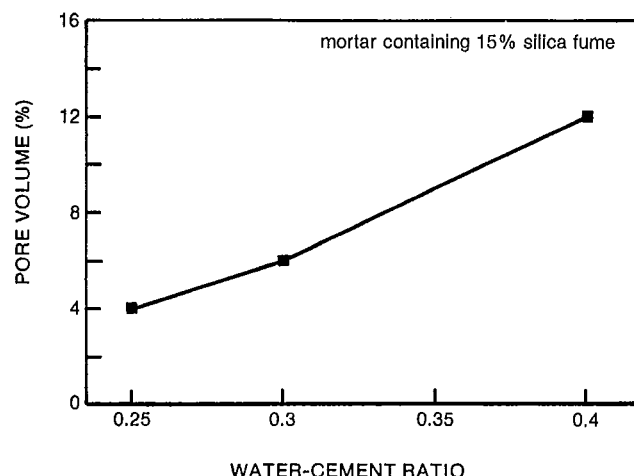


Fig. 2—Pore volume versus water-cement ratio.

Table 2 — Mortar batch with 15 percent substitution of silica fume by weight of cement with different HRWRA addition techniques

w/(c + SF)	1 percent s/s HRWRA, type of addition	Flow, percent	Air, percent	Compressive strength, MPa	
				14 Day	28 Day
0.45	Regular*	140	1.5	68.3	76.5
0.45	Split†	> 160	0.9	78.5	77.0
0.45	Delayed‡	> 160	1.0	77.4	84.0
0.40	Regular	116	3.1	78.3	85.5
0.40	Split	160	2.3	87.0	86.2
0.40	Delayed	> 160	1.0	87.9	91.0
0.35	Regular	85	5.0	89.7	89.1

*Regular: HRWRA added to mix water.

†Split: One-half of HRWRA added to mix water, the other one-half added as delayed.

‡Delayed: HRWRA added at the end of standard ASTM C305 mixing procedure and mixed an additional 1 minute.

NOTE: Type I cement was used. The mortar was made using concrete sand that passed ASTM C33. Sand/cement + SF ratio 2.0. Flow and air were determined by ASTM C185. Compressive strengths were determined using 50 mm cubes.

Table 3 — Mortar strength results using 10 percent by weight silica fume and different cement brands

Brand of cement	Percent s/s WRDA-19	Percent Silica fume addition	w/(c + SF)	Flow, percent	Air, percent	Compressive strength, MPa	
						14 days	28 days
A			0.6	83	3.3	32.4	36.2
B			0.6	52	3.3	35.9	40.0
C			0.6	79	1.7	32.4	39.3
D			0.6	60	4.3	33.8	35.2
A	1.0		0.45	18	9.3	37.2	38.6
C	1.0		0.45	18	6.4	40.0	40.0
A	1.0	10.0	0.45	93	5.0	58.6	65.2
B	1.0	10.0	0.45	41	4.9	60.3	61.7
C	1.0	10.0	0.45	81	5.9	60.0	62.7
D	1.0	10.0	0.45	50	5.8	63.4	64.1

Table 4 — Pore water volume in concrete

Silica fume, percent	Total intrusion volume,* cc/g	Bulk density, g/cc	Air, percent†	Pore water volume, percent
Blank	0.0398 ± 0.0041	2.4186 ± 0.0396	8.0	1.6
7.5	0.0399 ± 0.0094	2.4197 ± 0.1076	7.0	2.7
15	0.0335 ± 0.0040	2.4288 ± 0.0478	6.0	2.1

*Mercury porosimetry.

†Air determined on wet concrete.

NOTE: All concrete batches had w/c = 0.40. At least 5 samples were run on each total intrusion volume and bulk density measurement.

than reported here; however, the effects of this silica fume slurry do not appear to be sensitive to the brand of cement used. Table 3 shows results with different cements.

Because the use of this silica fume results in increased strength, one might expect a reduced porosity. The relationship between increased strength and reduced porosity is well known.¹ Fig. 2 shows the effect on the porosity of air-entrained mortar containing 15 percent silica fume as the water-cement ratio is varied. If the air content is subtracted, as it was in Fig. 2, it can be seen that there is an almost linear relation between porosity and water-cement ratio.

Contrary to the usual porosity-strength relationship for plain concrete,¹ reduced porosity is not generally found when silica fume is used (see Table 4^{2,3}) because porosity is a stronger function of the water-to-cement ratio. Indeed, this is what we found.

A careful look at the chemistry involved reveals that, in contrast to the usual cement hydration reactions, a reduction of total porosity by the addition of silica fume should not take place. We can demonstrate this by comparing the hydration of cement (for simplicity C₃S) with the reaction of silica fume with lime. The reaction products (C-S-H) are generally considered the same in both situations.⁴ The exact C-S-H composition may vary from C₂SH₂ to C_{1.5}SH₂.⁵ For example, in the hydration of cement

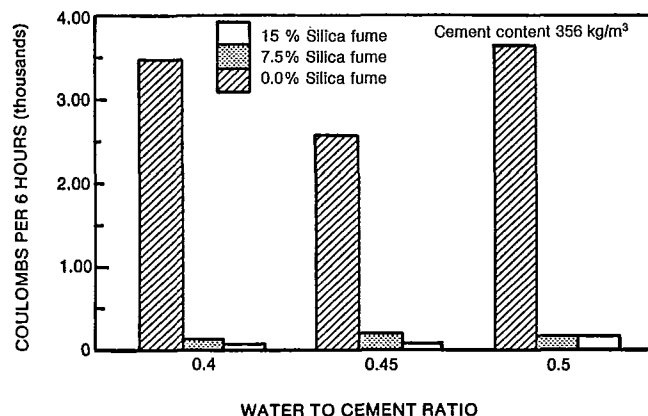


Fig. 3—Chloride permeability, FHWA/RD81/119.

Table 5 — Permeability of silica fume concrete (Gjörv⁸)

Cement content, kg/m ³	Silica fume content, kg/m ³	Permeability, kg/Pa·m·s
100	0	1.6×10^{-8}
100	10	4.0×10^{-11}
100	20	5.7×10^{-12}
250	0	4.8×10^{-12}
250	25	1.8×10^{-15}

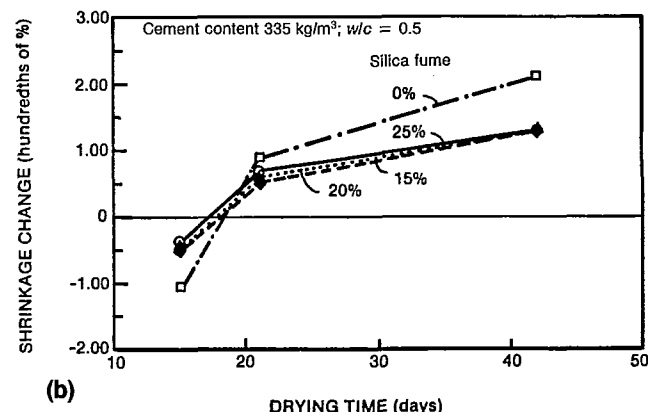
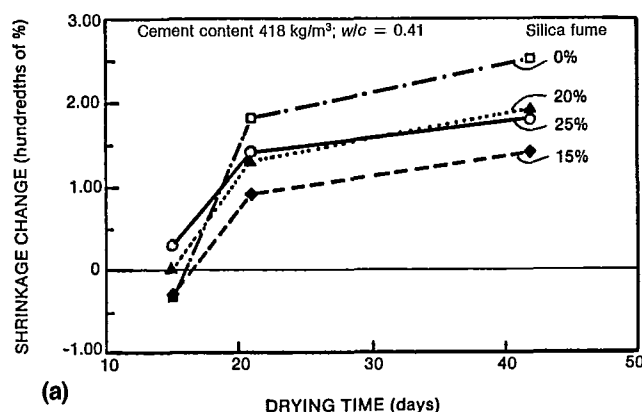
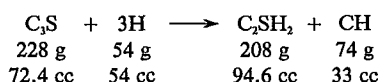


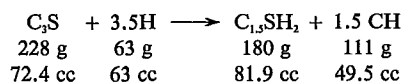
Fig. 4(a) and (b)—Drying shrinkage, modified ASTM C 157.

Case 1



or

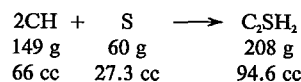
Case 2



The percent volume increase of solid in Case 1 is 76 percent (from 72.4 to 127.6 cm³) and in Case 2 81 percent (from 72.4 to 131.4 cm³). Such volume increase results in substantial decrease in the overall porosity.

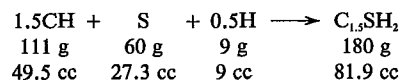
On the other hand, during the reaction of silica fume with lime, such large increases in the volume of solids do not occur (see Cases 3 and 4). Thus, decrease in total porosity is minimal.

Case 3



or

Case 4



Despite the small changes in total porosity, use of the silica fume admixture does reduce permeability. Fig. 3 shows the permeability of chloride in concrete determined by the electrochemical migration technique developed by the Federal Highway Administration in the U.S.⁶ It can be seen here that the water-to-cement ratio does not control, but that the addition of silica fume has a major effect on chloride movement.

These results are in agreement

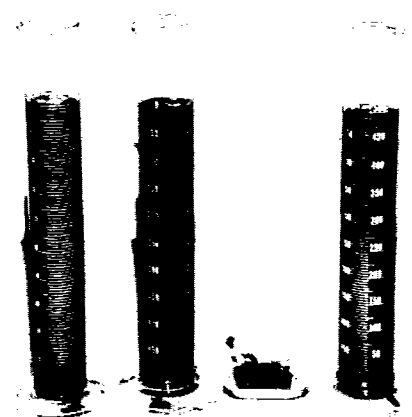


Fig. 5—Bleeding tests: (left) HRWRA 1 percent by weight of cement; $w/c = 0.45$; flow = 90 percent. (middle) No HRWRA, $w/c = 0.60$; flow = 64 percent. (right) HRWRA 1 percent by weight of cement; $w/c = 0.60$; flow = >160 percent. (All mortar samples contained 10 percent silica fume by weight of cement. The flow was determined by ASTM C 185. The dish contains the laitance from the graduate on the right. The photo was taken four hours after casting.)



Fig. 6—Molds used for concrete bond-strength test.

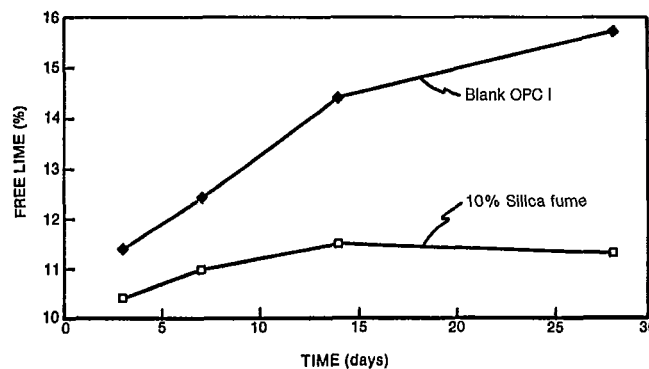


Fig. 7—Free lime content, ASTM C 114.

with the results reported by Gjrv⁷ for water permeability in silica fume-containing concrete (see Table 5). To explain the reduced permeability, Huang and Feldman² have proposed that silica fume increases the number of "ink-bottle" pores with narrow openings.

Drying shrinkage

In Fig. 4(a) and (b) the drying shrinkage of concrete containing different amounts of silica fume is shown at two different cement contents. It can be seen that less shrinkage occurs (over 40 days) when silica fume is used. Since the

aggregate effect on restraint of drying shrinkage in concrete⁸ is the same in all cases, the differences shown are more than likely due to slower water loss from concrete made with silica fume.

Bleeding and segregation

It is known that bleeding is normally reduced with the addition of a pozzolan; however, with this silica fume slurry, bleeding does not occur at all when the concrete is proportioned properly. Fig. 5 shows a case where we have deliberately used too much water, forcing the mix to segregate.

Instead of bleed water we obtained a laitance. This is because fine-particle silica holds water tenaciously. As will be seen later, this has a profound effect on bond strength since water in concrete migrates to aggregate and metal surfaces as the mix consolidates during the setting process.

Effect of aggregate strength

Table 6 shows the lack of importance of the aggregate strength on the strength of the mortar: the same strength was achieved when ceramic aggregates were substituted for regular concrete sand. However, when silica fume was added the strength was increased.

Table 6 — Ceramic aggregates in mortar

Mix number	Type of aggregate	Aggregate, volume percent used	Weight aggregate/cement*	w/c*	Flow, percent	Compressive strength 28 days, MPa
1	Concrete sand	100	2.00	0.30	40	78.4
2	Alumina sand - 8 mesh	10	2.70	0.30	83	78.2
	- 16 mesh	40				
	Alundum (Alpha Al ₂ O ₃)					
	30 Grit	40				
	70 Grit	10				
3	Same as #2 plus 15 percent silica fume substituted for cement by weight.		2.71	0.30	86	104.4

*Cement plus silica fume is used.

NOTE: HRWRA was used at 0.25 percent s/s (solid admixture to cement solids) regular addition and followed by 0.5 percent delayed addition.

Table 7 — Bond strength to steel with 10 percent by weight silica fume addition to concrete

Additive	w/c	Slump, mm	Approximate compressive strength, MPa	Bond strength,* MPa
None	0.48	165	35-40	26
Silica fume	0.42	216	55-60	37

*Values obtained by pushing pipe through concrete.

Cement paste-aggregate interface

Recently, attention has been focused on the cement paste-aggregate bond as being important to the strength development of concrete with silica fume. Bentur* has shown that water migrates to the aggregates surface where large crystals of Ca(OH)₂ precipitate, preventing the C-S-H from making good contact. Monteiro, Maso, and Ollivier⁹ have also confirmed by SEM and x-ray diffraction that both calcium hydroxide and ettringite are formed at the aggregate-paste interface. This same phenomenon was demonstrated via x-ray diffraction by Wang Jia et al.¹⁰

*Bentur, A., personal communication.

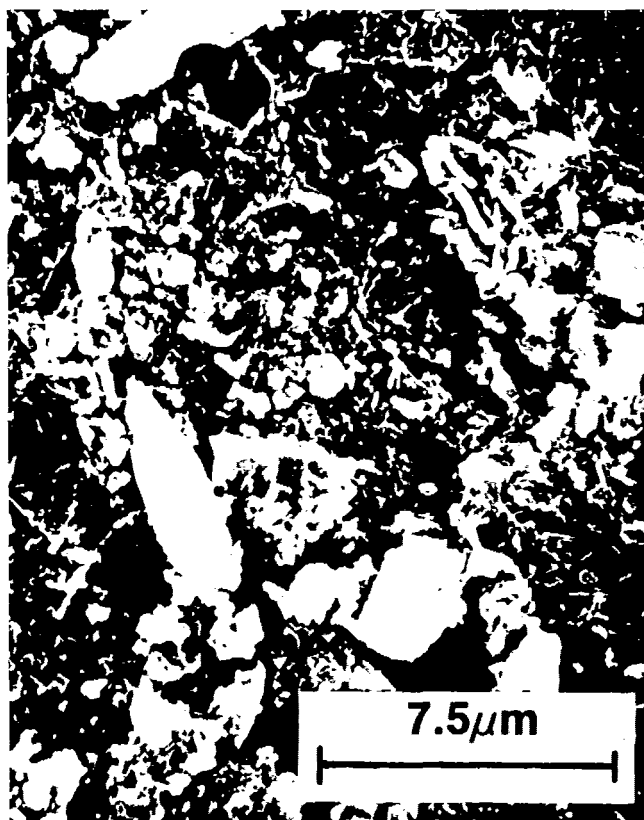


Fig. 8—Portland cement paste with silica fume at 14 days.

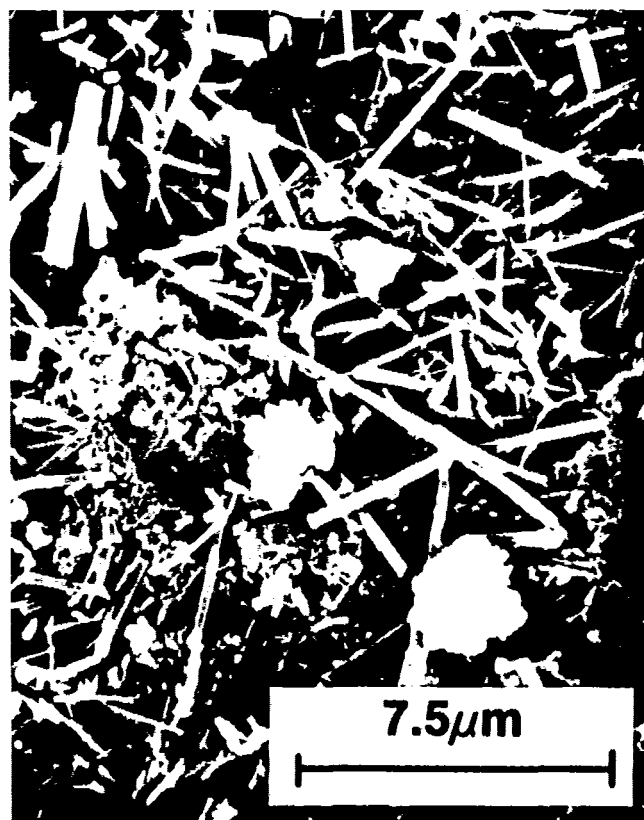


Fig. 9—Portland cement paste at 14 days.

In ordinary concrete, stress cracks tend to develop along the aggregate surfaces, whereas in high-strength concrete, fracture often goes through the aggregates. When we consider the aggregate, paste, and interface, the aggregate-paste interface is the weakest part in ordinary concrete.

The bond strength to steel for concrete with silica fume was examined. A steel pipe was placed in the center of a concrete cylinder and the force necessary to move it was determined.* Fig. 6 shows the apparatus. As can be seen in Table 7, the bond strength increased almost proportionally to the strength of the concrete.

It has been reported that the bond strength of concrete to steel does not increase with increasing compressive strength after the concrete reaches 4000 psi (27.6 MPa).¹¹ This must be related to the interface layer between the concrete and steel not changing in ordinary concrete while, when silica fume slurry was used, the bond strength in-

creased showing a definite effect on the steel-concrete interface.

Ettringite and calcium hydroxide formation

Chemical changes have been found in concrete when this silica fume slurry was used. In Fig. 7 it can be seen that the amount of Ca(OH)_2 formed in the presence of silica fume is reduced.

Fig. 8 and 9 are scanning electron photomicrographs of hydrating paste with and without silica fume. From these photographs it can be seen that ettringite (needle-like crystals) is not apparent in the case with silica fume. Since ettringite is an equilibrium compound formed from sulfate, calcium, and aluminate ions in solution, the reduction in lime would be expected to reduce ettringite formation. Space is also a consideration for ettringite formation.

Further, Fig. 10 shows the heat liberated by cement hydrating with and without silica fume. From this it can be seen that the added silica fume causes an increase in the early heat liberated. From the fact that

silica fume and lime, even with the addition of sodium hydroxide, do not liberate much heat alone, it would appear that silica fume actually accelerates the hydration of C_3S .

This acceleration of the cement hydration by silica fume was previously noted by Huang and Feldman.¹² The acceleration is most likely due to the reaction of silica fume with the Ca(OH)_2 , which is a product of the hydration of C_3S , thus encouraging further hydration. However, it is also possible that silica fume could provide seed crystals to facilitate precipitation of C-S-H.

Also, it has been shown¹³ that sulfate ions can be taken up in C-S-H as it is formed. The acceleration noted here would also help account for the reduced amount of ettringite found.

Conclusions

A new silica fume admixture in liquid form has been developed that is easier to handle than conventional powder forms. With this product, consistently high strength can be obtained regardless of portland cement source.

*Test procedure was obtained from W. Wiegand, W. R. Grace & Co., Cambridge, MA 02140.