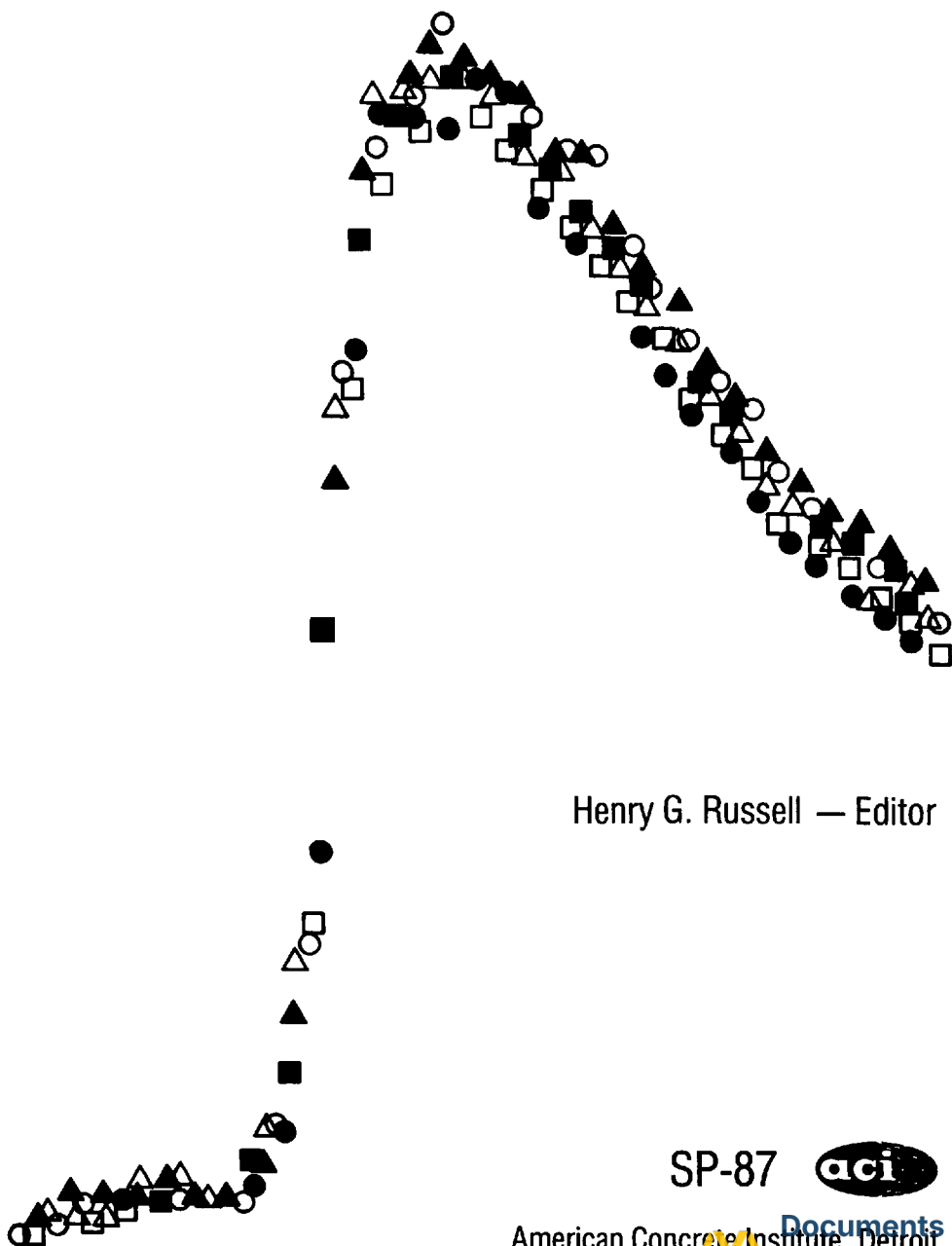


High-Strength Concrete



High-Strength Concrete



Henry G. Russell — Editor

SP-87



American Concrete Institute, Detroit
AA Documents
Document Sharing Hub

DISCUSSION of individual papers in this symposium may be submitted in accordance with general requirements of the ACI Publications Policy to ACI headquarters at the address given below. Closing date for submission of discussion is April 1, 1986. All discussion approved by the Technical Activities Committee will be published in the September/October 1986 ACI JOURNAL together with appropriate closing remarks by the authors.

The Institute is not responsible for the statements or opinions expressed in its publications. Institute publications are not able to, nor intended to, supplant individual training, responsibility, or judgment of the user, or the supplier, of the information presented.

The papers in this volume have been reviewed under Institute publication procedures by individuals expert in the subject areas of the papers.

Copyright © 1985
 AMERICAN CONCRETE INSTITUTE
 P.O. Box 19150, Redford Station
 Detroit, Michigan 48219

All rights reserved including rights of reproduction and use in any form or by any means, including the making of copies by any photo process, or by any electronic or mechanical device, printed or written or oral, or recording for sound or visual reproduction or for use in any knowledge or retrieval system or device, unless permission in writing is obtained from the copyright proprietors.

Printed in the United States of America

Editorial production: Patricia Kost

LIBRARY OF CONGRESS CATALOG CARD NUMBER 85-71930

PREFACE

High-strength concretes are used frequently in applications requiring slender members to carry large loads or span long distances. The purpose of this publication is to highlight applications of high-strength concretes, innovative approaches to their use, design implications, and research results. This volume is divided into two parts. Part 1 deals with applications of and innovations with high-strength concrete. Part 2 contains papers that describe design implications and research.

RECENT ACI ACTIVITIES

In 1979, ACI Committee 363--High-Strength Concrete was organized with the mission of studying and reporting information on high-strength concrete. As a first step toward accomplishing its mission, the "State-of-the-Art Report on High-Strength Concrete" was published in the July/August 1984 ACI JOURNAL, with discussion appearing in the January/February 1985 JOURNAL. The committee also sponsored a session on high-strength concrete at the 1982 Annual Convention and co-sponsored with Committee E703 a seminar on high-strength concrete at the 1983 Annual Convention. In 1984, a session entitled "High-Strength Concrete: Applications and Design Considerations," was held at the Fall Convention; and in 1985, two sessions on "Research and Experience with High-Strength Concrete" were sponsored at the Fall Convention.

Papers presented at the 1984 and 1985 Fall Conventions form the basis of this special publication. Fifteen papers were presented at the technical sessions, of which 14 are published in this symposium volume. Contributions were received from four different countries.

APPLICATIONS AND INNOVATIONS

Early applications of high-strength concrete emphasized its use to reduce column dimensions. More recently, high-strength concrete has been used to meet special project objectives such as large composite columns and stiffer structures. In turn, the use of high-strength concrete has prompted the application of more stringent quality control requirements.

In the paper 75-Story Texas Commerce Plaza, Houston--the Use of High-Strength Concrete, high-strength concrete as a composite material with structural steel is discussed. The author suggests that a composite section with high-strength concrete permitted an accelerated construction schedule, smaller column sizes, a more economical structural frame, and a greater stiffness in the structure as a whole.

Producers of prestressed, precast concrete products have traditionally used relatively high-strength concretes to reduce prestressing losses, obtain early release from formwork, and reduce member sizes. But, more recently, high-strength concretes are being used as a means of increasing the capacity of long-span members, controlling deflection, and in other more sophisticated applications. Some of these are discussed in Impact of High-Strength Concrete on Design and Service Behavior of Prestressed Precast Members.

The standard ACI procedures for establishing mix proportions may not be appropriate for high-strength concretes produced with higher than normal cement contents and high performance water-reducing admixtures. In geographic areas where high-strength concretes have not been used previously for large structures, it is important to evaluate available materials carefully and to identify expected levels of performance. In High-Strength Concrete in Seattle, the series of tests used to evaluate alternate mix materials, determine heat of hydration, and obtain mechanical properties of the concrete are described.

Development and research on high-strength concrete in France are described in the paper entitled High-Strength Concrete: How to Use it Every Day. In Canada, the use of silica fume and other special materials to produce a 13,000 psi concrete in the field are reviewed in Development and Experimental Use of a 90 MPa Field Concrete. Mechanical properties of the concrete are reported. The market for high-strength concrete has evolved gradually as a result of cooperative effort between the designers, contractors, concrete suppliers, and testing laboratories. In Statistical Controls for High-Strength Concrete, some methods of evaluating the laboratory test results of the compressive strength of high-strength concrete are discussed.

DESIGN IMPLICATIONS AND RESEARCH

For design strengths above 6,000 psi, the designer must be aware that many of the existing standards and codes are based on lower strength concretes. Engineering judgement may be necessary for utilization of higher strength concretes. Information needed by the design and construction professional will come from ongoing research. Part 2 of this symposium volume describes some of the completed and ongoing research. The applicability of existing design criteria for higher strength concretes is evaluated.

Current equations relating modulus of elasticity, tensile strength, and other mechanical properties of concrete to compressive strength are largely derived on the basis of data gathered from low- and moderate-strength concretes. The potential significance of the mechanical properties for design are critically examined in Design Implications of Current Research on High-Strength Concrete. Particular attention is given to areas where present ACI code provisions may not apply. In High-Strength Concrete--Material Properties and Structural Behavior, the properties and uses of high-strength concrete at early and late ages are discussed in detail.

Structural properties of high-strength concrete members are examined in the papers Structural Bending Properties of Higher Strength Concrete, Shear Tests of High- and Low-Strength Concrete Beams with Stirrups, and Reinforced Corbels of High-Strength Concrete. In all three papers, comparisons are made between laboratory test results and existing design procedures.

To ensure adequate ductility in columns, confinement of the concrete is necessary. In Lateral Reinforcement for High-Strength Concrete Columns, equations for predicting the performance of confined high-strength concrete elements are presented.

One alternate way of enhancing ductility is through the effective use of fibers. The principles and results for this approach are discussed in Properties of High-Strength Fiber Reinforced Concrete. The biaxial strength envelope and failure mechanism for high-strength concretes are discussed in detail in Behavior of High-Strength Concrete under Uniaxial and Biaxial Compression.

Committee 363 wishes to thank all authors who have contributed to this volume and to the staff of ACI who helped organize the symposiums and prepare this publication. Thanks are also due to the members of Committee 363 and all reviewers of papers. Without their cooperation, publication of this symposium volume would not have been possible.

Henry G. Russell
Chairman, ACI Committee 363

ACI COMMITTEE 363
High-Strength Concrete

Henry G. Russell*
Chairman

Jaime Moreno
Secretary

Pierre-Claude Aitcin
F. David Anderson
Roger W. Black
Irwin G. Cantor
Ramon L. Carrasquillo*
James E. Cook
Kingsley D. Drake
Gregory C. Frantz
Tom G. Guennewig
Weston T. Hester*
Anthony N. Kojundic

Brian R. Mastin
William C. Moore
Arthur H. Nilson
William F. Perenchio*
Kenneth L. Saucier
Surendra P. Shah
J. Craig Williams
Marshall S. Williams
John Wolsiefer, Sr.
J. Francis Young
Paul Zia

Corresponding Members

Charles D. Pomeroy

Jack O. Banning

Consulting Member

Antoine E. Naaman

*Members responsible for symposium volume.

CONTENTS

PREFACE.....iii

PART 1 - APPLICATIONS AND INNOVATIONS

75-STORY TEXAS COMMERCE PLAZA, HOUSTON--THE USE OF HIGH-STRENGTH CONCRETE
By Joseph P. Colaco.....1

IMPACT OF HIGH-STRENGTH CONCRETE ON DESIGN AND SERVICE BEHAVIOR OF PRESTRESSED PRECAST CONCRETE MEMBERS
By Alex Aswad and Weston T. Hester.....9

HIGH-STRENGTH CONCRETE IN SEATTLE
By Kingsley D. Drake.....21

HIGH-STRENGTH CONCRETE: HOW TO USE IT EVERY DAY
By R. Lacroix and P. Jaughey.....35

DEVELOPMENT AND EXPERIMENTAL USE OF A 90 MPa (13,000 psi) FIELD CONCRETE
By Pierre-Claude Aitcin, Pierre Laplante, and Claude Bedard.....51

STATISTICAL CONTROLS FOR HIGH-STRENGTH CONCRETE
By F. David Anderson.....71

PART 2 - DESIGN IMPLICATIONS AND RESEARCH

DESIGN IMPLICATIONS OF CURRENT RESEARCH ON HIGH-STRENGTH CONCRETE
By Arthur H. Nilson.....85

HIGH-STRENGTH CONCRETE--MATERIAL PROPERTIES AND STRUCTURAL BEHAVIOR
By R. N. Swamy.....119

STRUCTURAL BENDING PROPERTIES OF HIGHER STRENGTH CONCRETE
By S. E. Swartz, A. Nikaeen, H. D. Narayan Babu, N. Periyakaruppan, and T. M. E. Refai.....147

SHEAR TESTS OF HIGH-AND LOW-STRENGTH CONCRETE BEAMS WITH STIRRUPS
By Andrew G. Mphonde and Gregory C. Frantz.....179

REINFORCED CORBELS OF HIGH-STRENGTH CONCRETE
By Y. K. Yong, Douglas H. McCloskey, and Edward G. Nawy.....197

LATERAL REINFORCEMENT FOR HIGH-STRENGTH CONCRETE COLUMNS By A. Fafitis and S. P. Shah.....	213
PROPERTIES OF HIGH-STRENGTH FIBER REINFORCED CONCRETE By Antoine E. Naaman and Joseph R. Homrich.....	233
BEHAVIOR OF HIGH-STRENGTH CONCRETE UNDER UNIAXIAL AND BIAXIAL COMPRESSION By Robert C. Chen, Ramon L. Carrasquillo, and David W. Fowler.....	251
SI (Metric) TABLES.....	275
INDEX.....	277

Part 1

Applications and Innovations

SP-87-1

75-Story Texas Commerce Plaza, Houston — the Use of High-Strength Concrete

By Joseph P. Colaco

Synopsis: The article describes the use of high-strength concrete in the 75-story composite frame for the Texas Commerce Plaza in Houston. The advantages of the high-strength concrete were to reduce column sizes and to increase stiffness. All the concrete in this tower (approximately 100,000 cyd.) was successfully pumped, the highest placement being almost a 1,000' above the street level.

Keywords: columns (supports); concrete construction; high-rise buildings; high-strength concretes; pumped concrete; stiffness.

2 Colaco

Joseph P. Colaco received his PhD in Civil Engineering at the University of Illinois in 1965 and joined Skidmore, Owings & Merrill, Chicago office that same year. In 1969, Mr. Colaco joined Ellisor Engineers, Inc., Houston, Texas; in 1975, he and two partners opened their firm of consulting structural engineers, CBM Engineers (previously named Colaco Engineers, Inc.) in Houston, Texas.

I. INTRODUCTION

The Texas Commerce Plaza Tower in United Energy Plaza is a 75-story office building in downtown Houston. The design was started in 1978 and the structure was completed in 1981. The structure is the tallest composite building in the world and the tallest building west of the Mississippi at the present time.

A photograph of the building is shown in Figure 1. Figure 2 shows the general architectural layout of a typical floor plan and the essential elements of the structural system. The exterior structure of the building has columns placed at 10' (3m.) on centers on four sides but on the fifth side, there is an 85' (26m.) clear span. The exterior columns are composite using a steel erection column and cast-in-place concrete. There is also a cast-in-place concrete 5' (1.5m.) deep spandrel beam at each level (see Figure 3). The interior columns and floor framing use structural steel. The foundation of the tower is a 9'-9" (3m.) thick concrete mat, 63' (19.2m.) below grade. A more detailed description of the building is given in an article by Pickard (1). The exterior composite system, called a "ruptured-tube," is the main element that is used to carry the wind loads on the structure. Since the closely spaced tube is ruptured at the 85' (26m.) clear span front face, the healing of the rupture was required. Several alternates were considered viz. providing diagonals across the front face to complete the "tube" or providing stiff truss elements at discrete floors to tie the ends of the front face together. These alternates were discarded for aesthetic reasons. Hence, the only option was to use the building core. A concrete shear wall is placed next to the front row of elevators and the connection between the interior shear wall and the exterior tube is by very stiff steel link beams in the plane of the floor. There is a secondary stiffness element consisting of a steel girder that spans 85' (26m.) and ties the two triangular concrete front piers of the building. There is a dramatic difference in behavior of the structure. If the rupture were not "healed", the vertical movement from one end of the front face to the other would be approximately 6" at the top of the building.

At the time of the design of Texas Commerce Tower, the state-of-the-art in tall building design in Houston was exemplified by the 50-story concrete One Shell Plaza building and the 55-story InterFirst Tower which is a composite "tube" building. At the

start of the Texas Commerce Tower, discussions were held with the Owner on the materials and technology to be used for construction of the building. Due to the poor foundation conditions in Houston, an all-concrete building was ruled out because of its additional weight. The weight would require extremely deep and expensive foundations and would increase the construction time. It was felt, therefore, that the only reasonable choices were the composite building design or an all-steel structure. The architectural features of the building dictated a granite facade which tends to favor the composite building design. Further, experience with the 55-story InterFirst Tower indicated that a composite building could be built fast and economically and minimize foundation premiums. Therefore, it was decided that the building would utilize a composite "tube" system.

At the time of the design of Texas Commerce Tower, the maximum concrete strength utilized in building construction in Houston was 6,000 psi (42MPa) lightweight concrete for One Shell Plaza but most of the building utilized normal weight concrete with 5,000 psi (35MPa) as a maximum strength. This is due to the problems of obtaining higher strengths with river gravel that is a common aggregate in Houston. A 75-story structure utilizing low-strength concrete would result in extremely large columns. The Owners indicated that they would have a maximum exterior column size of 4'x4' at the third floor from a leasing standpoint. Hence, research was conducted on using high-strength concrete to reduce column size. If 5,000 psi were used, the exterior column sizes needed would be 4'x6' which was unacceptable. Discussions with Southwestern Laboratories in Houston indicated that high-strength concrete was being used in limited applications in highway work in Texas. Investigation indicated that if the aggregate were changed from river gravel to a limestone aggregate (which had to be imported into Houston), higher strengths could be achieved. A trial mix was conducted using 7,500 psi (52MPa) as a target design strength. A batch of this concrete was placed on a 24-story tall concrete building in the heat of the summer months to try to simulate a worst case condition. The results were very gratifying. Further discussions with Turner Construction (the Contractor on Texas Commerce Tower) suggested the use of high-strength concrete as a way to minimize costs and column sizes. This analysis confirmed published literature that economical construction is obtained by using high concrete strengths and the lowest reinforcement percentages. Turner retained The Concrete Associates of Dallas as consultants to review mix designs to come up with recommendations for the high-strength concrete. After extensive evaluation and testing, it was concluded that 7,500 psi (52MPa) concrete could be produced in Houston in sufficient quantities and with sufficient quality control to proceed with the design of the project.

4 Colaco

II. HIGH-STRENGTH CONCRETE

The outcome of the design was that 7,500 (52MPa) normal weight concrete was utilized in exterior columns, spandrel beams and shear wall from the mat foundation upto the 7th floor, and 6,000 psi (42MPa) was utilized for these elements from the 8th to the 30th floors. The mat foundation itself utilized 6,000 psi (42MPa) concrete. An extensive discussion of the mix designs, quality control and results of the entire program are described by Cook(2). Due consideration was taken of the Contractor's requirement that the concrete be pumped and also, of hot weather conditions in Houston. Texas Commerce Tower is unique in that all the concrete on the project was pumped with the highest concrete placement being about 1,000' (304m.) above street level. The concrete pump used was a Schwing BPA HDD-15 5000 trailer-mounted, 300 horsepower, 6" material cylinders, capable of 2,800 psi on concrete. The actual pressure ranged from 1,700 - 2,400 psi. No problems were encountered with the pumping operation. The mix designs indicated that 630 lb./cyd. (374 kg/m³) cement with 157 lb./cyd. (93 kg/m³) fly ash Type C was adequate for the 7,500 psi (52MPa) high-strength concrete in the project. The slump was 4.5 inches (10cm) and the water/cement ratio was 0.33. An ASTM C494 Type A water-reducing admixture was used with a dosage rate of 3 oz./100 lb. of cementitious material. The 7,500 psi (52MPa) concrete was placed during the period from November, 1979 to May, 1980 and the average 28-day concrete strengths were 8,146 psi (56MPa) while the 56-day strengths were 9,005 psi (62MPa). A similar set of numbers was achieved for the 6,000 psi (42MPa) nominal strength concrete. The 28-day averages were 7,257 psi (50MPa) while the 56-day average was 8,340 psi (58MPa). A degree of conservatism was exercised in the Specifications for this project as this was the first time that high-strength concrete was being used in Houston. In later work, 56-day strengths as opposed to 28-day strengths were considered to be the norm.

III. ADVANTAGES OF HIGH-STRENGTH CONCRETE

There are several advantages of high-strength concrete and they can be summarized as follows:

A. Additional Stiffness: One of the considerations in tall buildings is the restriction of inter-story drift under lateral loads. This is required in order to keep architectural elements from having any distress. Almost all tall building designs are controlled by stiffness requirements and hence, the use of high-strength concrete with its high modulus of elasticity results in a lower inter-story drift for the same member sizes. On this project, nominal 7,500 (52MPa) concrete had a modulus of elasticity of approximately 5.7×10^6 psi (39,330MPa) which is substantially higher than that which is obtainable with 5,000 psi (35MPa) concrete. The net result of this high E value is that the maximum deflection of the building under hurricane wind loads will not exceed 16" (41cm.) at the top of the building for wind in any direction.

B. Damping: Extensive discussions were held with the University of Western Ontario that conducted the wind tunnel analysis for the project. A detailed description of the structural elements of the design is discussed by Banavalkar (3). Since the basic wind resisting elements are concrete members, the damping ratio was 2% whereas for an all-steel building, a 1% damping factor is generally used. This resulted in a much lower peak acceleration in the wind tunnel test. The building accelerations at the top floor of the building were in the range of 18 milli-gs for a 10-year recurrence interval which is below the generally accepted criteria for the threshold of discomfort due to building motion.

C. Axial Shortening: The differential movements of vertical elements in tall buildings are a critical item in the constructability of level floors. In a composite frame, the problem is exacerbated by the fact that the interior columns of structural steel have only elastic axial shortening while the exterior composite columns are subjected not only to axial shortening due to stress but to shrinkage and creep. The use of high-strength concrete which has a higher modulus of elasticity reduces the axial shortening of the concrete columns. The use of limestone aggregate and the use of fly ash (which lowers cement content) reduce the shrinkage of the concrete columns. Axial shortening compensation tables were developed for the entire structure and interior steel column lengths were adjusted at 10-story increments to try to attain level floors. A detailed description of this is given in the paper by Banavalkar (3). In the worst case, at the roof level the columns at the end of the 85' clear span were placed 2.5 inches (6.3cm.) higher than the columns in the middle of the long faces.

D. Construction Techniques: One of the key ingredients of tall building economics is the ability to build the building as rapidly as possible. The General Contractor suggested two techniques both of which accelerated the construction schedule. These techniques were pumping of the concrete in the exterior frame and the use of a custom-made three-story steel formwork assembly that was self-climbing. Each side of the form assembly not only included the column and spandrel forms but also had lifting column system which enabled the formwork to climb. The column forms and spandrel forms were all hinged. One of the advantages of high-strength concrete in this configuration is that the columns and spandrel beams could be stripped early-on due to the low stress level and the relatively high concrete strengths. The average concrete strengths were 3,827 psi (26MPa) at 1-day and 5,218 psi (36MPa) at 3-days for the 7,500 psi (52MPa) concrete. The construction proceeded at the rate of two floors per week which at least equals that for structural steel construction.

IV. CONCLUSION

The use of high-strength concrete on Texas Commerce Tower has proved to be extremely successful in meeting the Owner's objectives, namely: reducing the column sizes (to increase the

6 Colaco

leasability of the floors); the development of rapid construction techniques; and, the satisfactory performance of the building from a stiffness and motion perception standpoint.

V. ACKNOWLEDGEMENTS:

A project of the magnitude of Texas Commerce Tower involves the hard work of all the team members (Owners, Architect, General Contractor, Concrete Consultant, Concrete Suppliers, etc.). The writer wishes to acknowledge all of them and the CBM Engineers' design team headed by Dr. P. V. Banavalkar and Mr. Tony Abyad.

VI. REFERENCES

1. Pickard, Scott S., "Ruptured Composite Tube Design for Houston's Texas Commerce Tower," Concrete International, July, 1981
2. Cook, James E., "Research and Application of High-Strength Concrete Using Class C Fly Ash," Concrete International, July, 1982.
3. Banavalkar, P. V., "Texas Commerce Tower, Houston (Texas USA)," Proceedings of International Conference on Tall Buildings, Singapore, October 22-26, 1984.



Fig. 1--Texas Commerce Tower in United Energy Plaza, Houston

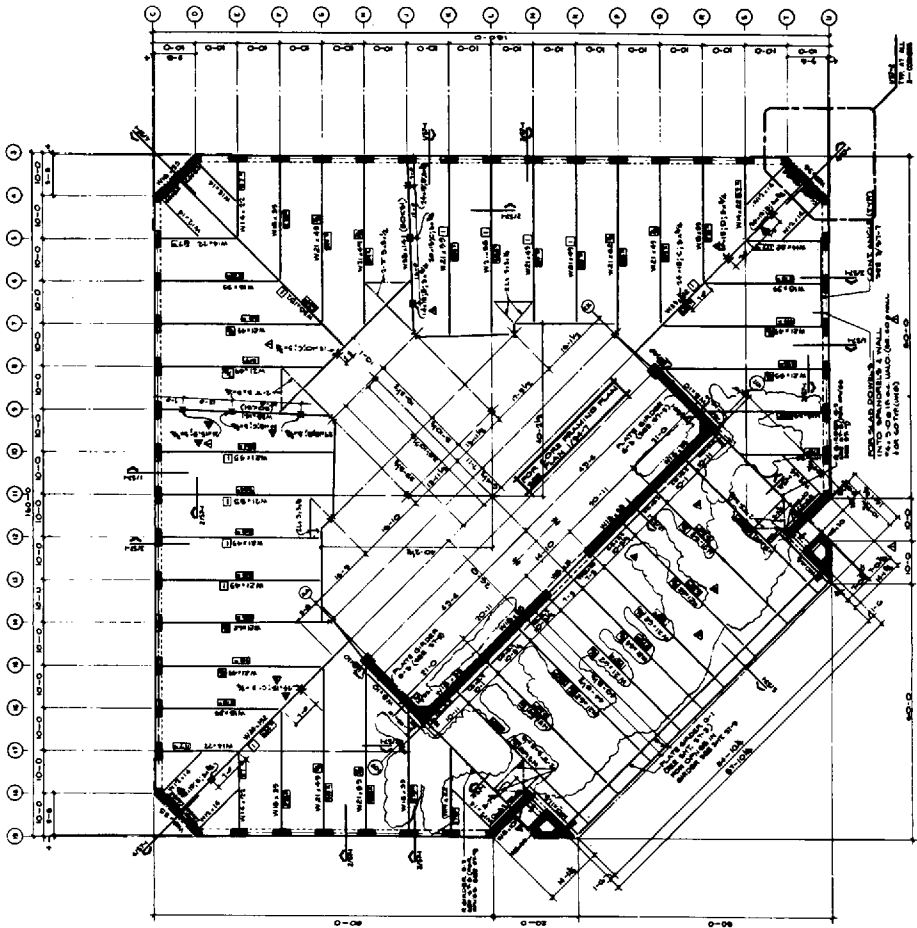
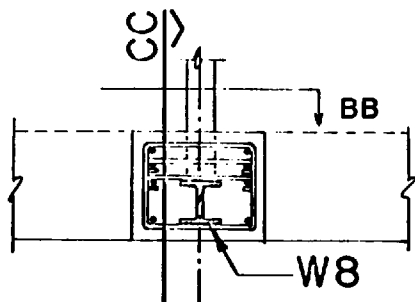
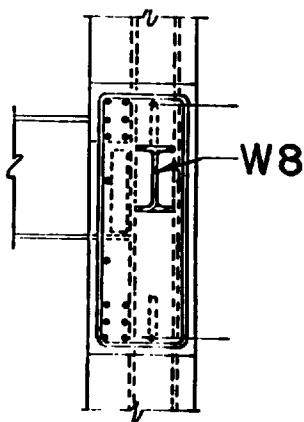


Fig. 2--Typical floor plan

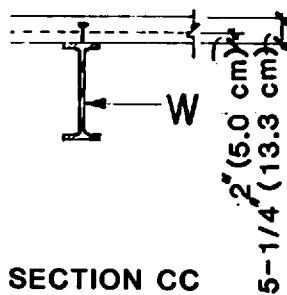
8 Colaco



PLAN OF EXTERIOR COLUMN



SECTION BB



SECTION CC

Fig. 3--Column, spandrel, and floor beam details

SP-87-2

Impact of High-Strength Concrete on Design and Service Behavior of Prestressed Precast Concrete Members

By Alex Aswad and Weston T. Hester

Synopsis: In the prestressed, precast concrete industry high-strength concretes are widely used for axially loaded piles and columns, but also increasingly for flexural members such as double tee beams and girders. Use of high-strength concretes permits fabrication of longer, more slender spans and economic mild and prestressed reinforcing steel patterns. And, for plant-produced members the necessary high strength may be achieved easily and consistently without out-of-the-ordinary quality control procedures, and this is documented here. To facilitate designs using high-strength concretes a number of design aids have been developed, and selected examples are presented. Finally, by its very nature, where high early strengths are required prestressed precast concrete members frequently have a substantial amount of long-term reserve capacity and excellent deflection control.

Keywords: beams (supports); girders; high-strength concretes; precast concrete; prestressed concrete; serviceability; structural design; T-beams.

10 Aswad and Hester

Alex Aswad, Ph.D., P.E., is a Senior Research and Design Engineer with Stanley Structures in Denver, Colorado. Dr. Aswad has published numerous papers on the design and analysis of prestressed concrete structures, and is active in ACI and PCI.

Weston T. Hester is Associate Professor of Civil Engineering, University of California, Berkeley. Dr. Hester is active in ACI Committee work, including ACI 363, High Strength Concrete, and other professional societies. He has extensive research and field experience with the production and control of high strength concretes.

INTRODUCTION

Manufacturers of prestressed, precast concrete products are increasingly using high-strength concretes for flexural and axially loaded members. Use of high-strength concrete permits more slender, higher-capacity and ultimately more economic structural sections. In many cases, use of high-strength concretes is a necessity and not an option if prestressed, precast sections are to be economically feasible for the project, and especially in the case of long span members and/or those carrying unusually heavy loadings. High early and long-term strength concretes are being used in double tees for roofs and floors, inverted tees for buildings, and I-, box- and double-tee-shaped (TT) elements for bridges.

Designers have traditionally defined high-strength concrete as that achieving at least 6,000 psi (42 MPa) at age 28 days or later [1]. But, whereas with traditional cast-in-place (CIP) construction the designer may require 6,000 to 12,000 psi (42 to 96 MPa) compressive strengths at age 28 days, or even 56 to 90 days, prestressed concrete producers may need these strengths much sooner. Producers are primarily interested in strengths at early ages (i.e., 12 to 18 hours), rather than long-term (i.e., at least 28 days), and these producers classify as "high-strength" concretes with early strengths of at least 4,000 psi (28 MPa) at age 12 to 18 hours, and 28-day strengths in excess of about 6,000 psi (42 MPa).

For both prestressed precast and CIP concrete construction, designers and contractors have used high-strength concretes (measured at age 28 days or later) to meet a variety of special needs, including:

1. Ability to use more slender, lighter sections;
2. Longer spans and/or wider member spacings;
3. Greater stiffness and better control of deformations;
4. Reductions in prestressing losses due to a better control over creep and shrinkage;

Prestressed Precast Concrete Members 11

5. Reductions in the amount of shear reinforcement required in some flexural members; and
6. Reduced impact of high early stresses due to prestressing and earlier application of service loadings and the effects it may have on members at early ages.

But, increasingly, producers and designers are realizing that use of high-strength concretes at early ages can also provide additional benefits. By achieving strengths in the range of 4,000 to 8,000 psi (28 to 56 MPa) in one day or less, and still higher strengths at 28 days or later, potential benefits include still greater deflection control, lower prestressing losses, and higher capacities.

The purpose of this paper is to discuss the advantages of high early and long-term strength concretes in prestressed, precast concrete operations, and to document its feasibility and benefits. The advantages of high-strength concrete notwithstanding, some designers and producers are still uncertain about its use. Some are concerned about the applicability of standard ACI and AASHTO design equations, and many are concerned about the precast concrete producer's ability to routinely and consistently produce a high early and long-term strength concrete.

In our experience, the ACI and AASHTO [2, 3] equations may be used to predict the performance of members with early and 28-day strengths in excess of 6,000 psi (42 MPa). This has been corroborated by field tests of structural members, and close examination of structures in service. In this paper we present a number of empirically verified equations established to facilitate the design and analysis of alternate member configurations in this strength range. Data is also presented documenting the fact producers are able to easily and economically produce these high-strength concretes.

HIGH-STRENGTH CONCRETE IN THE PLANT

In the literature a great deal of attention [1] has been given to the use and properties of high-strength concretes for conventional CIP construction. Its proponents cite the obvious advantages (e.g., more slender members), but also acknowledge that exceptional quality control efforts may be required to achieve consistent strength development and that design strengths may not be achieved for 56, 90 days or later. Prestressed, precast concrete producers, however, have an entirely different view of high-strength concretes. With plant cast concrete, early and long-term strengths must and can be achieved much more readily and economically than in cast-in-place (CIP) construction, and considerably more consistently.

For many reasons, designers are increasing the strengths required at early and later ages. Whereas the maximum strength required at 1 day was 5,000 psi (35 MPa) in the mid-1970s, today

12 Aswad and Hester

designers are requiring 12- to 16-hour strengths well in excess of 6,000 psi (42 MPa) and 28-day strengths in excess of 8,000 psi (56 MPa). In Figure 1, the minimum strengths required for one representative project and strengths actually achieved at early ages (12 to 18 hours) and 28 days are compared. The data was compiled for a series of long span, prestressed precast concrete I girders cast in Colorado. Depending upon the specific elements cast, early strengths required were 4,000 or 5,300 psi (28 or 37 MPa) and 28-day strengths were 6,000 psi (42 MPa). Note that even with this mid-range strength mix the requirements for early-strength concretes were easily surpassed.

Now, with the effective use of superplasticizers and other sophisticated applications of concrete technology, many producers can easily achieve strengths in the 6,000- to 7,000-psi range (42 to 49 MPa) in 12 to 18 hours, and above 10,000 psi (70 MPa) at age 28 days.

Central to the successful use of high-strength concrete is the producer's ability to meet the required strengths economically and consistently. And, based on extensive experience, we find each additional 1,000-psi (7 MPa) increase in early or long-term strengths (above a respective minimum of 4,000 psi and 6,000 psi (28 and 42 MPa)) will increase mix material costs approximately 5% to 10%. And, we have found that as a general rule no out-of-the-ordinary quality control efforts are necessary to meet these requirements.

Statistical data compiled for three high-strength concretes regularly cast in a plant confirms the ability to produce high early and long-term-strength concretes consistently, and, referring to Table 1, note the coefficient of variation and standard deviation for the high-strength concretes is comparable to or lower than those achieved with low- and moderate-strength concretes.

APPLICATIONS

The classic and earliest applications of high-strength concretes were for piles, columns, and other axially loaded members. But, in current practice much more attention is given to flexural members. With flexural members, high-strength concretes may provide greater deflection control, reduce prestressing losses, and/or permit use of less mild steel and prestressed reinforcing.

Two prototypical applications are for double-tee (TT) floor and roof members, and for bridge members. Double tee sections are used for long-span roofs (sometimes in excess of 110-foot (33 m)) clear span, and for parking structures and other buildings needing large, column-free areas. Bridge girder sections are designed according to AASHTO or similar design criteria.

Prestressed Precast Concrete Members 13

Double Tee Members

Double tee members represent a classic prestressed precast concrete product, may be used with or without a cast-in-place topping, and are ideal, economic sections for a bridge decks, parking structures, or other applications involving large live loads and long spans.

We assume the design standards presented in ACI 318 [2] apply to members made with high-strength concretes, at least up to strengths in the 8,000-psi (56 MPa) range. Long span members (e.g., with span/depth ratios 35 to 40) are also analyzed for the effects of long-term relaxation in the prestressing strands and creep and shrinkage in the concrete. The applicability of these design and analysis techniques [4] has been confirmed through a number of experimental tests of double tee members with 5,000 to 7,000 psi (35 to 99 MPa) and greater strength concretes [5, 6], and the performance of structures in service has been monitored in detail.

Preliminary Designs--The ACI design procedures notwithstanding, the design of an efficient long span roof or floor member can still be a time-consuming and arduous task, and to help this process a number of design aids have been developed. The preliminary design of floor and roof members is particularly helped by the use of design aids such as SPEED GRAPHS, i.e., graphical, design aids [7], based on the use of moderate- and high-strength concretes. These graphs summarize required early and long-term strengths, number and pattern of 1/2-inch strands, and the estimated camber at erection for a given set of spans and superimposed live loads. But, once a preliminary design has been performed, a final analysis is always required since a flat value for prestress loss is assumed for the graphs but is not allowed by ACI 318-83 [2].

Final Designs (an example)--A specific example illustrates the particular benefits associated with using high-strength concretes. Here, we will consider two designs for a 10DT28 member + 3-inch topping for a parking deck (i.e., a 10 feet wide, 2 feet 4 inches deep double tee with a 3-inch thick 3,000 psi topping). Using a "standard" design with an early, detensioning strength of 3,500 psi (24.5 MPa) and 28-day strength of 5,000 psi (35 MPa), the maximum permissible tensile stress [2] in the member is 848 psi (5.8 MPa) and the minimum shear reinforcement is 0.043 in²/ft. But, if the early strength is increased to 4,000 psi (28 MPa) and 28-day design strength is increased to 6,000 psi (42 MPa), the permissible tensile stress [2] is 929 psi (6.4 MPa) and the minimum shear reinforcement is 0.030 in²/ft.

14 Aswad and Hester

Summarizing, the increase in early and long-term strengths to 4,000 psi and 6,000 psi (28 and 42 MPa), respectively, resulted in the following benefits:

1. Bottom tension is now below the $12 (f'_c)^{1/2}$ limit;
2. Final camber under sustained dead loads is slightly larger (0.36 in (9 mm) versus 0.29 in (7 mm)) while deflection under live loads is less (0.94 in (24 mm) versus 1.21 in (30.7 mm));
3. The required total shear mesh dropped to 0.030 in²/ft from 0.043 in²/ft, based on V_{ci} , Equation 11-11 in ACI-318.

Reduced Shear Reinforcement--The reduction in required shear reinforcement is noteworthy. Based on tests of double tee members with and without shear reinforcement [6], it was found that requirements for the minimum area of shear reinforcement may be waived if the following criteria are satisfied:

- a. The members are considered to act as a simple span;
- b. Loads are uniformly distributed along the span;
- c. Shear stresses calculated using ACI Code Equation 11-11 (v_{ci}) required shear reinforcing in the tee of 0.04 in²/ft, or less; and
- d. Shear stresses calculated using ACI Code Equation 11-12 (v_{cw}) do not require shear reinforcing in the tee.

Referring to this work, it is feasible to omit the leg mesh since the minimum amount by calculation is less than 0.040 in²/ft, but, traditionally, a minimum mesh is placed at each end for 10% of the span ($\pm$).

Evaluating Alternate Designs--When designing a series of members all having the same depth but different spans, flange widths or unit loads, Equation 2 [8] is useful in estimating the number of strands, N_2 , once the required number, N_1 , has been found for a different span, L_1 :

$$N_2 = N_1 (L_2/L_1)^{2.60} (W_2/W_1)^X (Y_{b1}/Y_{b2})^2 \quad (1)$$

where

- N = number of strands
- L = member span
- W = total member weight per unit length
- X = 1.30 for untopped sections
- = 1.20 for composite sections (2-inch topping)
- = 1.10 for composite sections (3-inch topping)
- Y_b = distance between section centroid and bottom of stem



Prestressed Precast Concrete Members 15

Based on more precise calculations, we can say the accuracy of Equation (1) is quite good and generally better than 4% when service load stresses control and the maximum bottom tension is about 600 psi (4.2 MPa). The applicability of this equation to high-strength concretes has been independently confirmed by others.

In summary, use of high-strength concretes can permit longer span double tee members with less reinforcing, lower stress levels, and better control over long-term deformations. And, to facilitate preliminary design of these sections, standard design aids and formulas may be developed and used.

Bridge Girders

A second classical application of high-strength concretes is for bridge girders. Most state and local governments have adopted a limited number of bridge girder sections, and these are typically based on a state or AASHTO standard. For a given bridge using different span lengths and/or load patterns, the designer will then typically vary the amount of prestressing and concrete strengths used instead of section sizes in order to maintain a consistent appearance and minimize construction costs. By using higher-strength concretes the designer will then have greater flexibility in span lengths that may be used, and he will have more control of deflection and camber.

The analysis or design of bridge girders is a very tedious operation without the use of a computer. And, as with the conventional double tee construction, standard design aids [5, 7] can be extremely helpful in narrowing down the options and selecting the required prestressing.

Alternate Designs--When designing for several spans and/or girder spacings, the following equation [8] is recommended for predicting the number of prestressing strand required in the case of AASHTO HS-20 loading [3]:

$$N_2 = N_1 (L_2/L_1)^y (G_2/G_1)^z \quad (2)$$

where

- y = 2.29 for I-girders
 - 2.35 for boxes (spread or adjacent)
 - = 2.42 for composite TT
- z = 0.82 for I-girders
- z = 0.87 for spread boxes
- G = girder spacing

L and N designate member span and number of strands, respectively.

16 Aswad and Hester

Evaluating Alternate Strengths--Other useful equations for predicting the required release and final strengths at the harping points of I-girders are:

$$f'_{ci,2} = f'_{ci,1} (L_2/L_1)^{2.3} (G_2/G_1)^{1.15} \quad (3)$$

$$f'_{c,2} = f'_{ci,1} (L_2/L_1)^{1.9} (G_2/G_1)^{0.32} \quad (4)$$

where

$f'_{ci,2}$ = concrete compressive strength (psi) at time of initial prestress of member (2)

$f'_{c,2}$ = specified compressive strength (psi) of concrete for member (2)

The latter equations are based on several hundred cases [5, 8] and similar publications by other precasters. The accuracy of these equations relative to a rigorous analysis is generally better than 4%.

By inspection, using Equations 2 through 4, it is clear the required release strength grows faster than the girder spacing while the final strength increases at a much slower rate (the 0.32 power of G). The combination of Equations 2, 3 and 4 should allow the design engineer to evaluate the impact of span or spacing change on the number of strands and concrete strengths (initial and final). These three are the most important production variables and would determine the feasibility of a job.

Benefits of High-Strength Concretes--A classic, frequently cited example [9] of the benefits of higher long-term strengths is the opportunity to then reduce the number of girders. Raising the long-term design strength from 6,000 to 10,000 psi (42 to 70 MPa) will permit reduction of the number of required I-girders to 9 from 4. Unfortunately, however, largely because of ambiguity in the AASHTO design procedures with respect to strengths in excess of 6,000 psi (42 MPa), some designers will not use higher strengths.

Even if the long-term design strengths are limited to 6,000 psi (42 MPa), the designer and producer can capitalize on high-strength concretes. This fact is particularly important since it is relatively easy to achieve high early strengths.

Referring to Equations 3 and 4, by increasing the early (i.e., detensioning) strength from 4,000 to 6,000 psi (28 and 42 MPa), while maintaining the long-term strength at 6,000 psi (42 MPa), the feasible girder lengths may be increased up to 10 to 15%, or the number of girders required may be reduced as much as 40%. In effect, use of the higher early strengths permits the use of more prestressing force in a given section without a concomitant increase in upward camber, and excess tensile and compressive stresses.

Prestressed Precast Concrete Members 17

Achieving high early strengths can also provide exceptional long-term benefits in terms of reserve capacity and deflection control. If a member were designed for an early strength of, say 5,000 psi (35 MPa), and long-term strength of 6,000 psi (42 MPa), as a matter of practice the producer will provide early strengths of approximately 5,500 to 6,000 psi (38.5 to 42 MPa) or greater and long-term strengths of 8,000 to 10,000 psi (56 to 70 MPa) or greater. This is documented in Table 1, above. The early strengths would control the mix materials and proportions used, and virtually ensure that the required 28-day strengths were exceeded by a substantial margin. This margin would then serve to provide extra capacity in the member, but also significantly reduce prestressing losses due to elastic shortening of the member and creep movement related to high stresses in the section.

In summary, bridge girders provide a number of opportunities for using high-strength concretes to advantage. If the full range of attainable early and long-term strengths can be used, the girder's concrete strength can be selected to permit longer spans, wider spacings or less prestressing without a change in the girder's section. If an upper limit is placed on the long-term design strength, the early strength may be selected to also permit different spans or girder spacings, but also to provide reserve capacity.

CONCLUSIONS

The following are some of the advantages provided by the higher-strength concrete (both at early and later ages):

1. Longer achievable spans; double tees spanning up to 125 feet (38.1 m) for roofs and 70 feet (21.3 m) for bridges have been built. Six-foot-deep (1.8 m) I-girders spanning 158 feet (48.2 m) are currently being designed. In general, span-to-precast depth ratios of 37 to 40 for untopped double tees and 27 to 30 for composite double tees have been achieved. For I-girder bridges a ratio of 22 to 27 is feasible for girder spacings of 8 feet or 5 feet (2.4 or 1.5 m), respectively. With proper detailing these ratios resulted in satisfactory service behavior.
2. Better deformation control. With a higher-strength concrete Young's elastic modulus goes up, resulting in smaller deflections and end rotations.
3. Lower prestress losses due to creep. Because of the higher Young's modulus there are less creep losses which often result in strand savings. High-strength concrete typically has less shrinkage too because of the lower W/C ratio. This will further reduce prestress losses.

18 Aswad and Hester

4. Effect on shear reinforcement. Present ACI-318 ties the amount of shear reinforcement, A_v , to the "concrete contribution" term, V_{ci} . The latter value is larger in high-strength concrete and the required A_v would be lower. It is current practice in the industry [6] to omit the leg mesh in double tees if $A_v \geq 0.040 \text{ in}^2/\text{ft}$ ($\pm$). Therefore, the higher-strength concrete may in some cases result in elimination of such mesh.

REFERENCES

1. "State of the Art Report on High Strength Concrete," ACI Journal, No. 4, Vol. 81, July-August 1984, pp. 364-411.
2. "Building Code Requirements for Reinforced Concrete," ACI 318-83, American Concrete Institute, 1983, Detroit.
3. "Standard Specifications for Highway Bridges, Twelfth Edition 1977," American Association of State Highway and Transportation Officials, 1977, Washington, D.C.
4. Aswad, A., "Rational Deformation Prediction of Prestressed Members," ACI Publication SP-86, Deflections of Concrete Structures, July, 1985.
5. Aswad, A., D. Wathke and R. Hansen, "Simple Span Design Tables and Recommended Details," 1983 Edition, Stanley Structures, Inc., Denver, Colorado.
6. Anon, "Elimination of Shear Reinforcement in Prestressed Double Tees," 2nd printing, 1981, Stanley Structures, Inc., Denver, Colorado, p. 2.
7. Jacques, F.J., "SPEED GRAPHS for Prestressed Concrete," Colorado Prestressers Association, July 1974 and later editions, Denver, Colorado.
8. Aswad, A., "Strand Prediction in Selected Precast Flexural Members," Journal of the Prestressed Concrete Institute, September/October 1979, pp. 114-121.
9. Jobse, H.J., and S.W. Moustafa, "Applications of High Strength Concrete for Highway Bridges," Journal of the Prestressed Concrete Institute, May/June 1984.

Prestressed Precast Concrete Members 19

Table 1

Statistical Data for Selected High-Strength Concretes
Produced in a Prestressed Precast Concrete Plant

<u>Reference Mix No.</u>	<u>Specified Early Strength</u>	<u>Actual Early Strength</u>	<u>Specified 28-Day Strength</u>	<u>Actual 28-Day Strength</u>
1 (80 tests)	5,000 psi (35 MPa)	6,030 psi (42 MPa)	6,000 psi (42 MPa)	9,270 psi (42 MPa)
Standard deviation		630 psi (4.2 MPa)		610 psi (4.2 MPa)
Coefficient of variation		10%		6.6%
2 (35 tests)	6,000 psi (42 MPa)	6,910 psi (48.4 MPa)	6,000 psi (42 MPa)	10,870 psi (76.1 MPa)
Standard deviation		530 psi (3.7 MPa)		700 psi (4.9 MPa)
Coefficient of variation		7.8%		3.7%
3 (13 tests)	6,800 psi (47.6 MPa)	7,570 psi (54.3 MPa)	7,200 psi (50.4 MPa)	11,125 psi (77.8 MPa)
Standard deviation		540 psi (3.8 MPa)		470 psi (3.3 MPa)
Coefficient of variation		7.1%		4.2%

20 Aswad and Hester

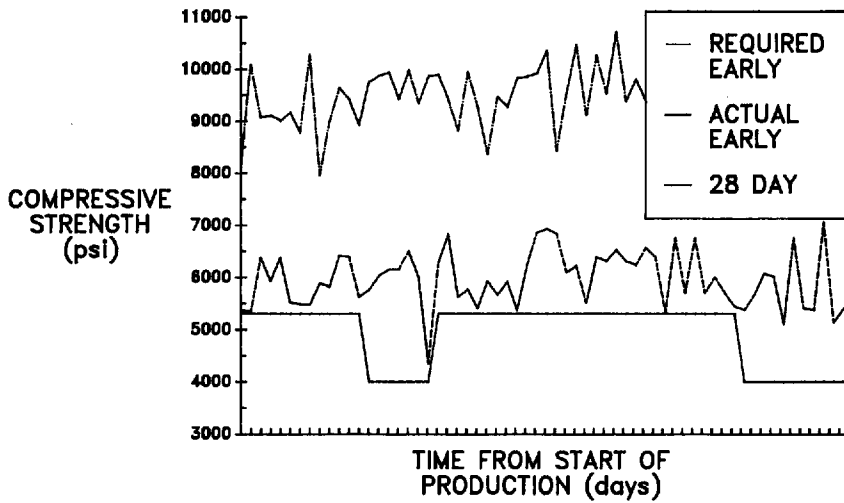


Fig. 1--Quality control chart for early and 28-day compressive strengths

SP-87-3

High-Strength Concrete in Seattle

By Kingsley D. Drake

Synopsis:

During the past 5 years, the utilization of high-strength concrete in Seattle has come a long way. This paper summarizes high-strength concrete use on 7 projects in the Seattle area, with particular emphasis on 9500 psi (670 kg/sq.cm) concrete at the Columbia Center project.

Columbia Center is used as an example of the cooperation and quality control required of the Structural Engineer, Contractor, Concrete Supplier and Testing Laboratory for high-strength concrete.

Columbia Center was the first major project to use a high cement content mix in mass (1200 yd³ - approx. 918 m³) placements. Strength gain, modulus of elasticity and temperature rise characteristics are presented.

Keywords: columns (supports); compressive strength; high-rise buildings; high-strength concretes; mass concrete; mix proportioning; modulus of elasticity; quality control; temperature rise (in concrete).

22 Drake

Kingsley D. Drake, P.E., District Manager - Pittsburgh Testing Laboratory, Seattle, Wash. B.S. - Univ. of Wash. Twenty-four years experience in Mix Proportioning, Testing, Inspection, and Q.C. of Concrete. Member ACI Comm. 363, "High-Strength Concrete", & Comm. 212 - Subcomm. 2, "Properties of Concrete at Early Ages".

INTRODUCTION

Through the late 1960's, 3,000 and 4,000 psi (211 and 281 kg/sq.cm) concrete was commonly used in all the major structures in the Seattle area. Even in the mid-1970's, the concrete in the all concrete King Dome was primarily 3,000 and 4,000 psi (211 and 281 kg/sq.cm) with 5,000 psi (352 kg/sq.cm) used only in the columns. To use in excess of 6,000 psi (422 kg/sq.cm) was unheard of until the late 1970's. This paper outlines the utilization of high-strength concrete in the Seattle area during recent years.

TYPICAL PROJECTS

The first significant use of high-strength concrete in Seattle was in the One Union Square Building (Figure #1). This consisted of 7,500 psi (527 kg/sq.cm) concrete in the 38 story slip formed core. The mix consisted of 3/8" (approx. 1 cm) aggregate and no superplasticizer, which was somewhat of an achievement for 1979.

The next use of high-strength concrete was in the Third and Broad Building. This project utilized 8,000 psi (562 kg/sq.cm) concrete in the interior columns. A 5/8" (approx. 1.6 cm) aggregate mix was designed with 846 lbs. (384 kg) of cement, fly ash, and superplasticizer, with a water cement ratio of 0.24.

In 1982, the Skyline Tower Building was constructed in Bellevue, a suburb of Seattle. This 26 story steel frame building utilized 1,500 yards³ (1,147 m³) of 7,000 psi (492 kg/sq.cm) concrete in the composite columns. The mix consisted of 3/8" (approx. 1 cm) coarse aggregate, 635 lbs. (288 kg) of cement, 20% fly ash, and superplasticizer. The concrete actually averaged 9,500 psi (670 kg/sq.cm) at the design age of 56 days. This project is of special interest because of the use of post-tensioning in the exterior columns, with individual strands stressed a vertical distance of up to 260 feet (approx. 79 m).

Another recent project (1984) was the World Trade Center in Tacoma, which is approximately 35 miles (approx. 56 km) south of

Seattle. This project had 8,000 psi (562 kg/sq.cm) concrete columns on the first floor, 7,000 psi (492 kg/sq.cm) columns on the second floor, and 6,000 psi (422 kg/sq.cm) columns and beams throughout the remainder of the building.

A project that is currently (1984) under construction is the Ballard Hospital addition. This is a relatively small building utilizing the lift slab method of construction. The 8,000 psi (562 kg/sq.cm) concrete used in the shear walls of the elevator shaft consisted of 3/8" (approx. 1 cm) aggregate, 658 lbs. (298 kg) of cement, fly ash, and superplasticizer.

A project started in 1984 is the Century Square Building in downtown Seattle (Figure #2). This is a 29 story steel frame building with composite columns. Figure #3 shows the typical congestion of structural and reinforcing steel found in a composite column utilizing high-strength concrete.

The composite columns will utilize 10,000 psi (703 kg/sq.cm) concrete at an age of 56 days. Based on field and laboratory trials, a mix with 799 lbs (362 kg) of cement and 3/8" (approx. 1 cm) aggregate will be used. The first high-strength concrete for this project was placed in December, 1984.

COLUMBIA CENTER

Probably the most well known of all the Seattle hi-rise buildings is the Columbia Sea-First Center (Figure #4). This 76 story building is the tallest in the Western United States and reported to be the 8th tallest in the world.

The building consists of three slender towers of differing heights which are bundled together, with the towers stepping back at the 43rd and 61st floors. This is a steel frame building except for three large composite columns consisting of structural steel and 9,500 psi (670 kg/sq.cm) concrete.

These columns are located in the middle of each of the building's three straight faces. The columns carry most of the gravity loads, along with the wind and seismic forces. The columns measure 8' x 12' (approx. 2.44 x 3.66 m) at the base of the building and taper to 3' x 3' (approx. 0.9 x 0.9 m) at the top. A refined computer analysis and these columns made it possible to limit the steel in the frame to less than 15 pounds per square foot (73 kg/sq. m) of floor area, compared to in excess of 20 psf (98 kg/sq. m) for a conventionally designed building.

24 Drake

Design Requirements

Columbia Center is an excellent example of the cooperation required in the design and construction of high-strength concrete structures. During the final design stages and approximately 1 year prior to construction, the General Contractor, the Architect, the Owner/Developer, the Structural Engineer, and the Engineering Materials Testing Laboratory met to finalize the design for the high-strength concrete in the composite columns.

The Structural Engineer indicated that a compressive strength of 9,500 psi (670 kg/sq.cm) was required to support the building loads. The main concern was to maximize the modulus of elasticity in order to reduce the amount of sway in the upper stories of the building.

Since the high "E" was for tenant comfort only and the upper floors would not be occupied for at least 6 months following final concrete placements, a design age of 180 days was selected. Because the high-strength concrete was to be used in columns that were very congested with reinforcing and structural steel, a slump of 8-9" (approx. 20-23 cm) was desired.

Mix Design

Following the designation of design parameters, the Testing Laboratory instituted a series of laboratory and field trials. Various sizes of aggregates, ranging from 3/8" (approx. 1 cm) to 3/4" (approx. 2 cm), were tried along with crushed and natural stone. Additionally, experiments were made with Types I, II, and III cement.

The final mix selected consisted of 846 lbs. (384 kg) of Type I/II cement and 20% Class F fly ash. A natural aggregate was decided upon, consisting of a blend of 70% 3/4" (approx. 2 cm) and 30% 3/8" (approx. 1 cm), and a coarse paving sand. The 3/4" (approx. 2 cm) maximum aggregate was selected for its ability to produce concrete with the highest "E" values.

Admixtures used were a normal water reducing agent and a superplasticizer of the naphthalene type, resulting in a final water cement ratio of 0.25. With this mix it was found that the desired 9,500 psi (670 kg/sq.cm) compressive strength could easily be obtained. The Structural Engineer was informed that an "E" of 6.75×10^6 psi (475×10^3 kg/sq.cm) could be used for the final design calculations.

Mass Footing

During excavation, approximately 3 months prior to initial concrete placements, soils problems were encountered that required the Structural Engineer to re-design the footings with 9,500 psi (670 kg/sq.cm) concrete. Since the 9,500 psi (670 kg/sq.cm) mix had been designed for an age of 180 days and the "E" values of the mix had been incorporated into the final design, it was impossible to change the mix at this late date.

Since the main footings were massive in size (48' x 48' x 14') (approx. 14.6 x 14.6 x 4.3 m), there were a number of concerns relative to heat rise (Figure #5). Because there was no published data on heat rise for such a high cement content mix, there was particular concern for the differential temperature between the inside and the outside of the concrete.

In order to determine what sort of heat rise and differential to expect, a series of three test blocks were cast. The test blocks were 6' x 6' x 6' (approx. 1.8 x 1.8 x 1.8 m) and were cast at varying conditions of fresh concrete and air temperature, since it was probable that at least one, or possibly two, of the footings would be cast during the high temperature months of the summer.

In order to determine the heat rise characteristics, thermocouples were placed at the center as well as the exterior faces of the test blocks. As can be seen from Figure #6, we recorded a 35-40°F (approx. 19-22°C) differential between the inside and the outside, and a maximum temperature of 180°F (approx. 82°C). The heat rise characteristics were similar in all three test blocks. The blocks were closely monitored for several months and none showed any visible surface cracking.

Another concern was that of possible strength variation due to the temperature differential. To answer this concern, a series of cores were removed from throughout the block at 28 days. The cores indicated the average strength of the concrete to be within 110 psi (7.8 kg/sq.cm) from outside to inside, and only 500 psi (35.2 kg/sq.cm) below that of the companion laboratory cured cylinders.

Quality Control

Columbia Center is an excellent example of the quality control required for high-strength concrete. The Engineering Materials Testing Laboratory, the Concrete Supplier, and the General Contractor -- all had written quality control procedures.

The Concrete Supplier was required to perform aggregate gradations at the pit as well as at the batch plant. Moisture

26 Drake

content checks were performed daily using the oven dry method as well as continuous monitoring of the sand with a moisture meter. The Supplier was also required to check his trucks on a monthly basis for cleanliness and operational characteristics. But most important, the water content was kept constant per the requirements of the mix design with slump being strictly controlled by the use of superplasticizer.

The General Contractor had a full time Q.A. manager assigned for concrete quality. He monitored not only his personnel, but also the activities of the Concrete Supplier and the Testing Laboratory. One of the Contractor's responsibilities was the controlled dosing of superplasticizer at the jobsite. The slump of concrete in each truck was measured when the truck arrived and then a carefully measured amount of superplasticizer was added. The drum was rotated at mixing speed for 5 minutes, or 75 revolutions, whichever was greater. Then another slump was taken to make sure the concrete was within the desired 8 to 9" (approx. 20-23 cm) range. The Contractor's Q.A. manager was adamant that no water be added at the jobsite in order to increase workability. He made sure that the personnel of the Contractor, Concrete Supplier, and Testing Laboratory strictly kept their hands off the high-strength concrete water content.

The Testing Laboratory had the greatest responsibility for quality control on the high-strength mixes. The Laboratory had a full time batch plant inspector assigned to all high-strength placements. He was required not only to monitor the ingredients going into the mix, and to monitor the moisture content checks being made by the ready mix supplier, but to also check that each truck, as it backed under the central mixer, had it's drum reversing so that all wash water would be expelled.

The Testing Laboratory assigned a concrete engineer to supervise all high-strength placements. He was also required to inspect all concrete trucks, every 3 months, in a detailed manner to insure that the trucks were in conformance with the requirements of NRMCA (1).

On each concrete placement, the Testing Laboratory assigned resident concrete inspectors to monitor the placement as well as the curing. Full time concrete technicians were assigned for casting cylinders and performing slump and unit weight tests. All technicians were registered special inspectors as well as being qualified in accordance with the Laboratory's nuclear certification program.

Cylinders were cast in heavy steel molds and immediately placed in field curing boxes which were thermostatically controlled. In order to segregate the high-strength cylinders for special handling, colored data forms and tags were attached to the fresh cylinders. Needless to say, strict adherence to all ASTM procedures was maintained. All high-strength cylinders were transported to the Laboratory in special carrying racks and

covered with curing blankets to avoid damage.

Immediately upon receipt in the central laboratory, the cylinders were capped and put into a standard moist curing room. In order to be sure there would be no mix-up in the curing room, all high-strength cylinders were marked with a distinctive spot of paint. All cylinders were tested by nuclear qualified laboratory technicians on a 600 kip (272,160 kgf) Universal Testing Machine.

The Structural Engineer was also interested in determining the actual value of "E" throughout the placements, so one set of cylinders per 1,000 yd³ (765 m³) was tested for chord modulus of elasticity at the design age of 180 days.

The General Contractor was concerned relative to the design age of 180 days and felt that even 7 days was not going to be early enough to detect potential strength problems. In order to answer this concern, a field laboratory was set up on the jobsite and additional cylinders were tested at 24 hours, in accordance with the warm water cure method (2), for all column placements. From this data, a correlation was developed to the 7, 28, 56, and ultimately, the 180 day strength.

The Laboratory also set up criteria designated as "trigger points" (Table #1). The "trigger points" enabled all concerned personnel to know the level of strength the concrete should obtain at various ages in order to reach the design strength of 9,500 psi (670 kg/sq.cm) at 180 days.

Construction

The first high-strength concrete was placed in the mass footings, which contained approximately 1,200 yd³/each (approx. 918 m³). The concrete was batched with just enough superplasticizer to get it out of the mixer at a 2-3" (approx. 5-8 cm) slump. This required approx. 2/3 of the total dosage. When the concrete arrived at the jobsite, a haul of approximately 20 minutes, it was re-dosed to bring it to the desired 8-9" (20-23 cm) slump.

In order to determine the heat rise, thermocouples were placed in two of the mass footings. The thermocouples were placed on all 4 sides and throughout the inside at approximately mid-height. The maximum temperature of 190°F (approx. 88°C) occurred in about 7 days, as did the maximum differential of 49°F (approx. 27°C) in footing #1 (Figure 7) and 63°F (approx. 35°C) on the other monitored footing. Even though this was somewhat higher than the 40°F (22°C) differential desired, there was no significant cracking. The amount of cracking observed, over a 2 month period, was similar to what one would expect in a mass footing of this size with a low cement content.

For the column placements, all the trucks discharged into a

28 Drake

high speed hopper which transported the concrete to an intermediate floor for pumping to adjacent floors. The columns were highly congested with structural and reinforcing steel. The reinforcing steel consisted of #8 welded wire fabric; 3" x 3" (approx. 8 x 8 cm) on center with a 2" (approx. 5 cm) vertical spacing.

In addition to the high-strength concrete that was in the column, there was also an area of floor around each column that contained 9,500 psi (670 kg/sq.cm) concrete. Approximately 8,000 cubic yards (approx. 6,100 m³) of 9,500 psi (670 kg/sq.cm) concrete was placed in the columns in this building.

Test Data

The data obtained to date (Figure #8) indicates an average compressive strength of 9,700 psi (682 kg/sq.cm) at 28 days, 11,000 psi (773 kg/sq.cm) at 56 days, and in excess of 12,000 psi (843 kg/sq.cm) at 180 days, with some sets of cylinders as high as 15,000 psi (1,056 kg/sq.cm).

The average chord modulus of elasticity for the 9,500 psi (670 kg/sq.cm) mix was 6.94×10^6 psi (488×10^3 kg/sq.cm). However, as shown by Figure #9, the individual "E" values tend to be somewhat lower, than that predicted by the ACI 318 Building Code Formula, in the range of 14,000 to 16,000 psi (984 to 1,125 kg/sq.cm).

In conclusion, Seattle has come a long way in the past 5 years. Based on current knowledge, high-strength concrete, in the range of 11,000-12,000 psi (773 to 843 kg/sq.cm), is feasible for use in the foreseeable future.

Credits

Owner/Developer - Martin Selig Real Estate
 Architect - Chester Lindsey & Associates
 General Contractor - Howard S. Wright Construction Company
 Structural Engineer - Skilling, Ward, Rogers, Barkshire
 Engineering Materials Testing Laboratory - Pittsburgh Testing Laboratory
 Concrete Supplier - Pioneer Construction Materials

References

- (1) National Ready Mix Concrete Association, "Check List for Ready Mixed Concrete Production Facilities", 5M/3-80.
- (2) ASTM C 684-81, "Standard Method of Making, Accelerated Curing, & Testing of Concrete Compression Test Specimens", Procedure A - Warm Water Method.

Table 1 - Trigger points

TRIGGER POINTS

9500 PSI @ 180 DAYS

AGE	COMP. STRENGTH (PSI)	
	CAUTION	ACTION
1 DAY	3000	2800
7	5900	5500
28	7500	7100
56	8600	8200
90	9300	8800

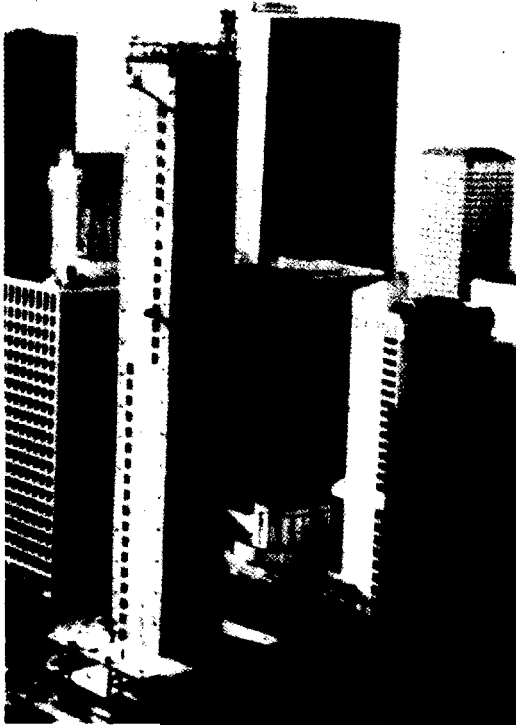


Fig. 1--One Union Square Building



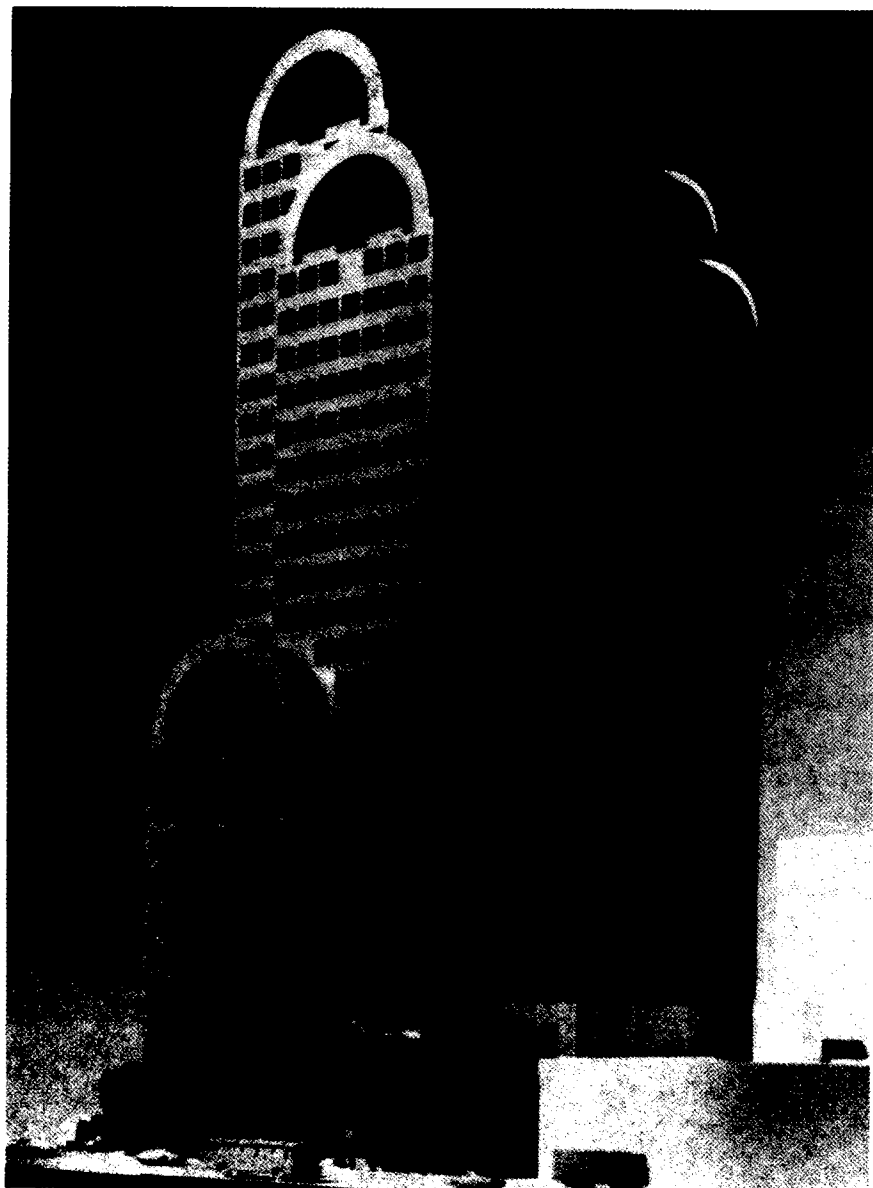


Fig. 2--Century Square Building



Fig. 3--Century square - composite column mockup

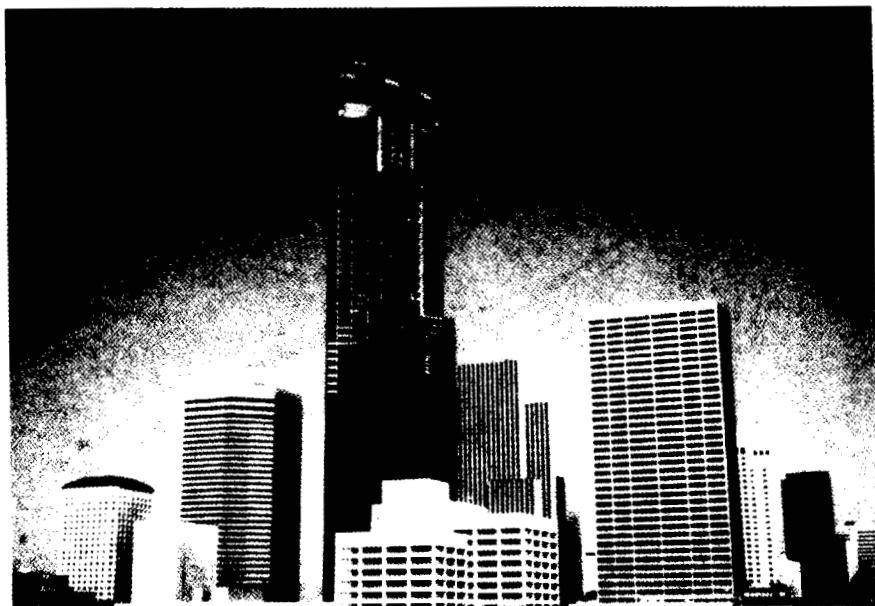


Fig. 4--Columbia Center Building

32 Drake



Fig. 5--Mass footing - Columbia Center Building

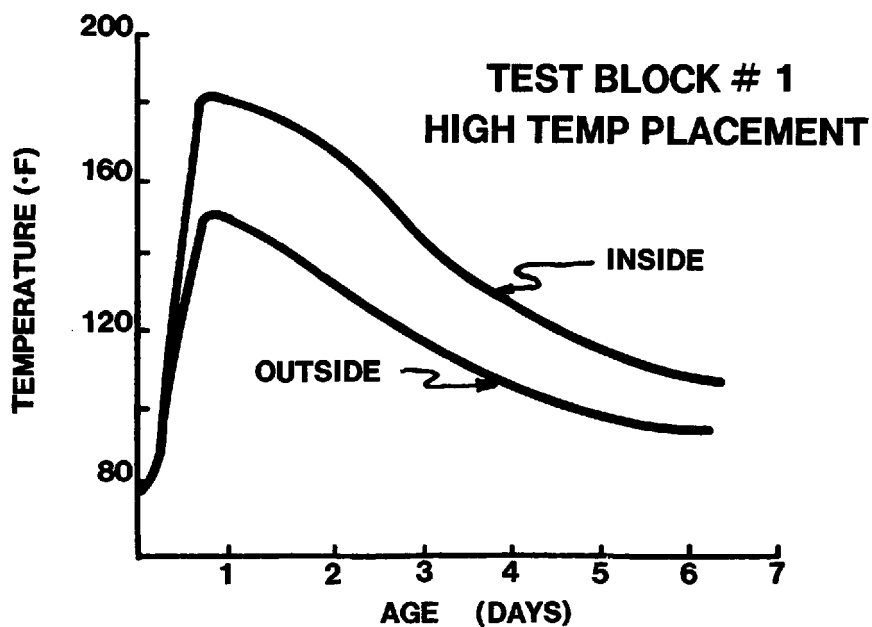


Fig. 6--Heat rise in test block

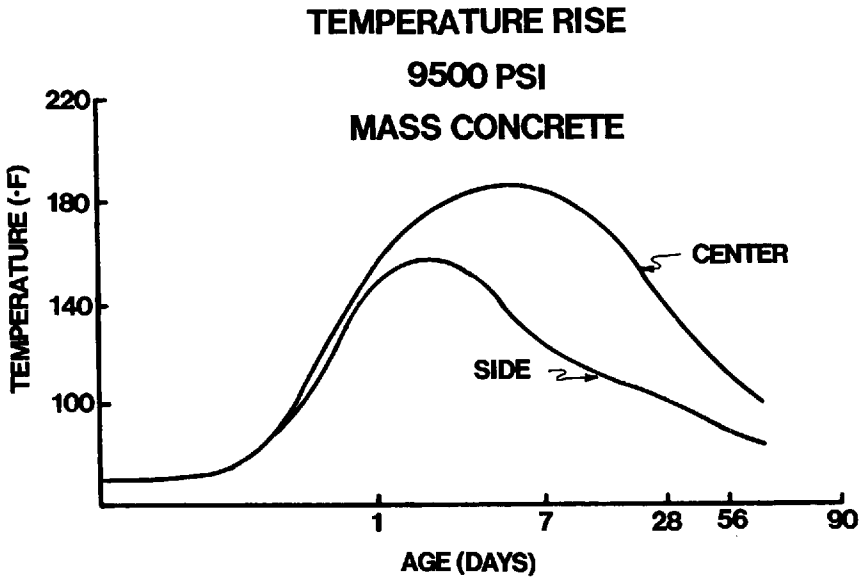


Fig. 7--Heat rise in mass footing - Columbia Center Building

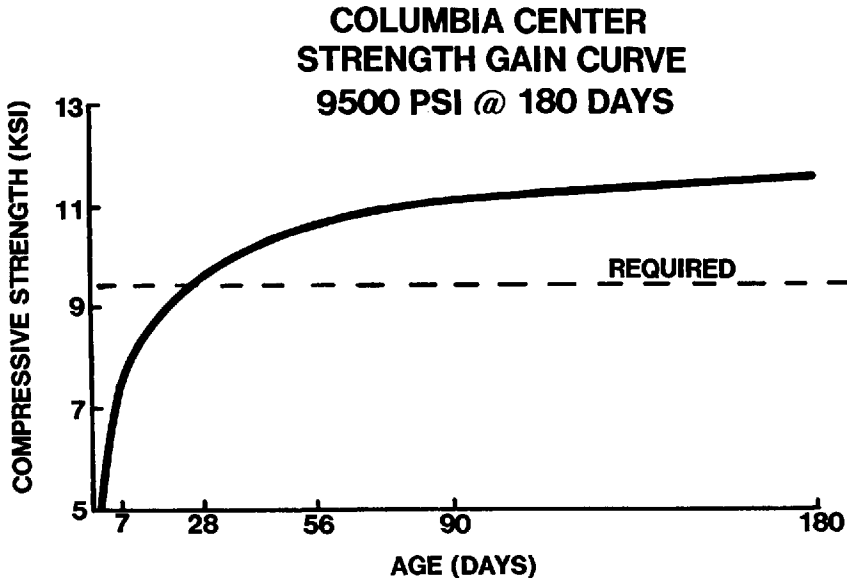


Fig. 8--Strength gain - 9,500 psi (670 kg/sq.cm) mix

34 Drake

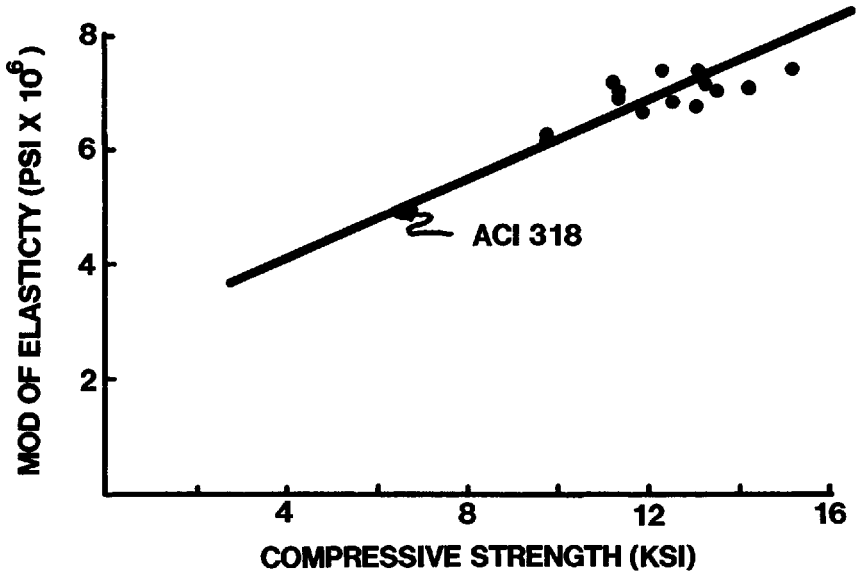


Fig. 9--Modulus of elasticity - 9,500 psi (670 kg/sq.cm) mix

SP-87-4

High-Strength Concrete: How to Use it Every Day

By R. Lacroix and P. Jaugey

Synopsis: A research program has been promoted in order to study the compositions and properties of High Performance Concretes to be produced in France. Concretes with a characteristic compressive strength of 50 to 60 MPa (7 000 to 8 500 psi) on cylinders tests may be fabricated every where in France at a reasonable cost, by selecting aggregates, cement and superplasticizers. The Brittle behaviour of such concretes under loads action needs a reevaluation and a possible adaptation of the codes of design. The other characteristics of this material are significantly improved : reduction of creep and shrinkage, increase of compacity. The gain to be expected from a generalized use of such concretes results from short time characteristic, by decreasing the member sections and improving the rate of utilization of shuttering, and from long time characteristic by increasing durability. However a complementary program is now in progress in order to clear some uncertainties and to design an experimental structure.

Keywords: aggregates; cements; costs; economics; high-strength concretes; plasticizers; research.

36 Lacroix and Jaugey

Roger LACROIX is a professor and head of department at Ecole Nationale des Ponts et Chaussées, PARIS, France. R. LACROIX received diplomas from Ecole Polytechnique and Ecole Nationale des Ponts et Chaussées. He is consulting engineer; he is also honorary president of Fédération Internationale de la Précontrainte and current president of Association Française pour la Construction.

Pierre JAUGEY is head of Quality Control Department of Société Générale d'Entreprises - Sainrapt et Brice. P. JAUGEY received his diplomas from Ecole Nationale Supérieure des Mines de PARIS and Ecole Nationale Supérieure des Mines de SAINT-ETIENNE. His research interests include concrete and materials technology.

INTRODUCTION

Since about one century the mechanical strength of concrete has hardly varied : in France current buildings require a 25 to 30 MPa (3 500 to 4 000 psi) strength, and exceptional structures require more than a 35 MPa (5 000 psi) strength. These figures represent characteristic values achieved from cylinder samples at 28 days of age, often wrongly compared with mean values obtained on cubes at 90 days of age according to obsolete methods. But even taking into account the change in definition, today's values are hardly higher than those obtained by HENNEBIQUE and COIGNET, at the end of the nineteenth century; the concrete strengths of the most advanced present day structures are lower than those prescribed by FREYSSINET and CAQUOT in their designs.

If the look of today's constructions appears more slender than these of our predecessors, this fact results more from an increase in the ratio between service stress and ultimate stress than from a true increase of material strength value. This apparent reduction of the safety factor is entirely justified by recent major advances in many fields :

- better knowledge of actions,
- more precise assessments of stresses,
- better quality standard of materials,
- in some cases, better control on workmanship.

However, modern cement plants deliver high strength cements of excellent quality, chemical industry has developed a complete range of concrete admixtures and equipment builders bring to the sites more and more precise and reliable machines for mixing and transporting the concrete. Therefore, this near regression of concrete performances may surprise many, and has probably its origin in the increasing indifference of engineers in the technical aspect of the job and at a lower extent, in a position of fear in face of responsibilities.

Today, making laboratory concrete of up to 80 (11 500 psi) or even 100 MPa (14 000 psi) compressive strength does not represent any difficulty, provided a proper selection of cement, aggregates and admixture for the mix design is made.

A number of very useful data have been gathered in this field by the ACI Committee 363. [1]

The aim of the research undertaken by SGE (1), CEBTP (2), CERILH (3) and LCPC (4), with the help of DAEI (5), was quite different. The actual targets were :

a) To determine the practical ways to obtain, on site and at a reasonable cost, a concrete with a characteristic strength in the range of 50 to 60 MPa (7 000 to 8 500 psi).

b) To confirm the economical interest in such a concrete, compared with a usual structural concrete.

2. RESEARCH PROGRAM

The research started at the end of 1982. The program included seven steps described below.

2.1. Preliminary Surveys

1a-- Precisions brought on the possible utilizations

1b-- Bibliography on high strength concretes, presently made on site.

1c-- Survey aggregate sources, in order to obtain a first selection of aggregates available in FRANCE to be used in the experimental scheme.

Selection of a few sources of aggregates to be supplied and comparative studies of their characteristics. Synthesis elaboration from all the survey results brought by the research partners.

2.2. Comparative Analysis of Cement & Admixture Combinations

- Experimental survey of french high strength cements.
- Selection supply and tests of six particular cements (chemical composition, granular distribution, mineralogy, hydration heat development, strength).
- Tests on concrete mixed with these cements and standard aggregates, with a cement content of 425 kg/m³ (715 lb/yd³) : workability, strength, swelling/shrinkage.
- Tests on the six concretes with four selected admixtures.

2.3. Concrete Mix Study and tests

3a-- Comparative tests on selected aggregates : systematic

- (1)SGE is one of the most important civ.eng. contracting firms in FRANCE.
- (2)Centre Experimental du Bâtiment et des Travaux Publics
- (3)Centre d'Etudes et de Recherches de l'Industrie des Liants Hydrauliques
- (4)Laboratoire Central des Ponts et Chaussées
- (5)Direction des Affaires Economiques et Internationales (Ministère de l'Urbanisme et des transports).



38 Lacroix and Jaugey

studies of several concrete mixes with two of the cement-admixture combinations selected during the previous step. Rheology, workability, shrinkage, creep, static modulus, tensile strength.

3b-- Strain-stress diagrams at different ages : strength under sustained compression.

2.4. Evaluation of the Conditions to be met for using the previously designed Concretes

4a-- Design conditions.

4b-- Site conditions

- Comparison of crushed and natural sand mixing
- Type of mixing and time available before hardening
- Influence of percentage of fines and out granular classes (eventuality of rescreening operation)
- Curing
- Convenience and control tests
- Procedure of compression test.

2.5. Complementary tests to be undertaken during the previous studies and report of evaluation finalizing the five first steps.

2.6. Rebars-Concrete Bond and Structural Tests

6a-- Rebars-concrete bond (ultimate bond stress) : the aim is to compare the behaviour of the previously designed concretes to that of standard structural concrete.

6b-- Flexural beams behaviour : the rupture of rectangular section beams made of usual structural concrete is compared with that obtained with the selected concrete. Some tests are performed with flexural ruptures and other for shear rupture. Phase 6 points to the particular conditions which have to be taken into account in every design using such concretes.

2.7. Preliminary Design of a Structure in Comparison with Design using standard Concrete

The previous studies must be completed by an experimentation on site : the aim of this design phase is to turn the use of high strength material to the best account for building a given structure.

This is a direct preliminary to a complete execution : it will be necessary to specify the measuring equipment to be placed on and into the structure, and the level of quality control required for a good achievement.

3. BIBLIOGRAPHY SURVEY AND FIELD OF UTILIZATION

The bibliography related to high strength concrete is very wide and the papers reflect more and more the purpose of concrete quality improvement, a purpose which becomes common throughout the world.

The ways for achieving this improvement are as follows :

3.1. Selection and Content of Cement in Concrete

Various authors studies either pure Portland cement or cements combined with fly ashes, or High Alumina cement. Their main conclusions are :

- Portland cement, the most common, is sufficient to bring the required strength.
- The strength increases with the cement content but it seems there is an economical optimum around 400 kg/m³ (675 lb/yd³).
- It is possible to substitute mineral fillers for about 15% of the cement, without appreciable change in the strength.

3.2. Aggregates

Three main ideas are prevalent :

- The high strength aggregates are not the best for concrete, because of their very high elastic modulus, compared with the modulus of a cement paste; from this contrast stress concentrations occur, which are detrimental to the concrete mechanical behaviour.^[2]
- For high strength concrete, the traditional criteria of size distribution, which have been set and taught by the good authors, become less important and sometimes no longer usable.
- The bond between aggregates and cement paste is an important factor, which is explained by the fact that concrete rupture in compression comes through the loss cohesion on planes running parallel to the stress axis.

3.3. Mixing Water and Admixtures

The water content, sufficient for a complete hydration of Portland cement is near 26 % on the cement weight. Usual concretes have a water/cement ratio around 0.45 ± 0.03 , the excess of water being necessary for obtaining the required workability.

Now, high strength concrete implies a much lower W/C ratio, in the range of 0.30 to 0.35. Such a low water content can only be reached by using a superplasticizer which has a drastic fluidifying effect, although for a short time.

Entrained air, as micro-bubbles, brings durability to the concrete, but a too high content of entrained air reduces concrete strength.

3.4. Production and Workmanship

Publications are scarce on this sensitive matter. Actually, most papers are written by university professors or laboratory researchers, few of whom have had the opportunity of achieving site testing on a large scale. However, there is very much to be said on tolerances control, application, methods, etc... They represent important cost factors which are to be considered in cost comparisons between standard and high strength concrete.^[5]

We also should admit that the field of application is rather

40 Lacroix and Jaugey

restricted judging by the use of the cement : statistics issued by the National Syndicate of Cement Producers show that concrete is mostly used in standard building constructions, roads, sewages, urban water and street networks, etc... which do not necessitate high strength.

On the other hand, it may be interesting to prescribe high strength in the following cases :

- Medium and large span bridges; overseas applications, in particular in JAPAN prove it [6]
- In high rise building an improved concrete quality would reduce the height of the beams and the cross section of columns; [4]
- For offshore construction, platform columns are submitted to high stresses and should weigh as little as possible for reasons of floating stability;
- For certain precast beams, the weight reduction is noticeably influential on the economy of handling operations and early concrete strength allows shorter production cycles; [7]
- The early strength is also an important factor of cost reduction of in situ building construction, for it allows a dramatic shortening of the total construction time, enabling an earlier unloading of the successive floors;
- Furthermore, thanks to its low water content, the high strength concrete has other advantages : lower creep and shrinkage, and above all greater compactness, which is an essential factor of durability.

4. CEMENT AND AGGREGATES SELECTION

Six cement types, CPA 55 (1) or CPA HP (1) were selected. In all regions the selection was made first in relation with the mechanical properties and handling distances between cement factories. In any case the delivery distance was rarely over 150 kms (100 miles).

All selected cements have a low C₃A content. This particularity allows their use in maritime or offshore works.

The aggregates problem is more complex with such a large variety of potential or existing supply sources spread all over French territory.

For tests purpose : hard limestone from Boulonnais and an alluvial aggregate from the Rhône-Alpes region were selected as examples.

From indications given by regional laboratories, it is possible to draw a map of FRANCE defining the approximate borders between three zones corresponding to the possible production concrete, with strength characteristic

(1) French designation (Ciment Portland Artificiel) corresponding to American Type I Portland Cement with strength classification in four classes. The two upper classes CPA 55 and CPA HP correspond respectively to guaranteed minimum AFNOR mortar strength after 28 days of 45 MPa (guaranteed mean value 55 MPa) and 55 MPa. The strength requirement for CPA 55 is near the optional physical requirement after 28 days for ASTM C 150 type I Portland Cement (with conversion factor of 1.5 between ASTM and AFNOR mortar strength).