

# LIGHTWEIGHT CONCRETE FOR TOTAL DESIGN OF ONE SHELL PLAZA

By FAZLUR R. KHAN

The author discusses the design and construction aspects of the 714-ft tall, 52-story One Shell Plaza Building in Houston. Inasmuch as the economic success of this building depended on the use of high strength, cast-in-place lightweight concrete, the various special considerations in relation to the behavior of lightweight concrete and the development of the structural system as well as the structural details are discussed in this paper. It is concluded that the use of cast-in-place high strength lightweight concrete is not only practical, but also a very important economic factor in the successful design and construction of ultra high-rise concrete buildings.

Keywords: columns (supports); concrete slabs; creep properties; field tests; fire resistance; floors; foundations; frames; high-rise buildings; lightweight aggregate concretes; office buildings; reinforced concrete; shrinkage; skyscrapers; structural design; temperature; tube-in-tube; wind pressure.

## INTRODUCTION

The 714-ft tall, 52-story One Shell Plaza building in Houston, Texas, (Fig. 1-1) is an investment type office building of high quality. Inasmuch as it was conceived as an investment building by the developer, Gerald D. Hines Interests of Houston, economy was one of the most important factors in choosing the structural system for the building. Because the Houston foundation soil is primarily clay extending down to more than 2000 ft below ground level, the most efficient foundation construction has traditionally been floating type foundations on thick concrete mats. Consequently the taller the building, the deeper the foundation. On the basis of previous experience in the Houston area, the maximum practical depth of excavation for foundations is about 60 ft. Hence, within this maximum depth of foundation, the lighter the construction system for superstructure, the taller the building can be.

This project was originally thought of as a 35-story building due to practical foundation limitations, even though a taller building up

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to 50 stories was desirable from an investment point of view if it was economically feasible. The author's preliminary analysis indicated that if high strength, lightweight concrete weighing 115 lb per cu ft could be used in combination with an optimum overall dimension of 192 x 132 ft, and an efficient mat configuration of 232 x 172 ft, then a 52-story building could be built for the original estimated unit price of the 35-story building!

The decision to build the world's tallest concrete building, therefore, was made on only one basic structural condition—that the structural system must produce the most economical total building. This led to the final conclusion that lightweight concrete must be used for the construction of the entire building, including the heavy mat foundation.

The effect of lightweight concrete on the choice of the structural system, the analysis and design of all structural elements and the resulting architectural details are discussed in this paper.

### TUBE-IN-TUBE SUPERSTRUCTURE

The structural system developed for this building consists of closely spaced exterior columns forming a framed tube, shear walls enclosing the core area forming an interior wall tube, and light structural concrete joists spanning column-free between the exterior framed tube and the interior shear wall tube. This is schematically shown in Fig. 1-2. This particular system is a refinement of the shear wall and frame interaction type of structure. Proper proportioning of the two tubes made the entire system so efficient that the effect of the high design wind load became insignificant on the design of columns and shear walls. Although from the strength point of view, high wind load did not control the design of the vertical elements, the control of lateral sway nevertheless required that the minimum modulus of elasticity  $E_c$  of the concrete be  $3 \times 10^6$  psi. Using normally available data, the expected modulus of elasticity of the specified 6000 psi lightweight concrete would have been approximately

$2.3 \times 10^6$  psi. Therefore, to expect a  $3 \times 10^6$  psi modulus of elasticity for lightweight concrete could be considered unrealistic. However, there was sufficient evidence to believe that, given the right combination of coarse lightweight aggregate and fine materials, a much higher modulus of elasticity than usually expected can be achieved.

The spacing of the exterior columns was divided on the basis of their efficiency in terms of resisting wind load without any "premium for height," and at the same time providing support for an efficient floor structure. The spacing chosen was 6 ft and the structural floor joists were also spaced at 6 ft, thereby each joist was supported at a perimeter column thus avoiding spandrels from supporting the joist reaction and consequent torsion. Fig. 1-3 shows a typical floor plan.

### FLOOR STRUCTURE

The floor slab system is required to have a 2-hr fire rating. The initial design thickness of the slab between joists was 3-1/2 in. This was slightly modified later to provide the 2-hr fire rating. Because dead weight is the most significant factor affecting the economy of this building, the slab was designed first for optimum structure and the required fire rating was to be achieved, if necessary on the basis of a fire test, by lightly spraying the 3-1/2 in. slab with vermiculite which hardly affects the dead weight of the slab. For a 2-hr fire rating, a 3-1/2 to 3-3/4 in. all-lightweight concrete slab is generally adequate without further treatment whereas for normal weight concrete the thickness would have to be at least 1 in. more as shown in some of the recent fire tests.

Fig. 1-4 shows a section through the typical joist-and-slab floor construction. The 24 in. deep joist slabs span 36 ft between the interior shear wall and the exterior columns, which provides considerable end fixity. Except for the few upper stories, the effect of the end moments from the joist is generally insignificant on the design of the perimeter columns. However, the end fixity made the structural floor system considerably more rigid, resulting in smaller deflections and eliminating unacceptable floor vibrations. The combination of the lightweight concrete with high strength (4500 psi) resulted in considerable savings in quantity of reinforcement as well as of concrete.

No additional floor fill was considered because electrification of the floor was provided by a so-called "punch-through" method. The punch-through method simply means that the electrical conduits are hung below the floor system and are brought up as necessary by drilling small holes through the 3-1/2 in. slab. This method of electrification eliminates the traditional 3 in. floor fill.

The one-way joist system was transformed in a natural way into a two-way waffle system in the four corner areas of the floor as shown in Fig. 1-5. The use of the two-way waffle also provided loading on all columns around the corners. The resulting total floor slab system weighs an average of only 71 lb per sq ft.

### MAT FOUNDATION

Houston soils dictate floating type foundations for heavy structures. The foundation design concept required that the entire superstructure be supported on an 8 ft 3 in. concrete mat 60 ft below grade and extending about 20 ft beyond the edge of the tower on all four sides (Fig. 1-6). For optimum performance, 6000 psi concrete was required, and it would have been much easier to use normal weight concrete for that mat in terms of availability. But using normal weight concrete in the mat, instead of lightweight concrete, would have increased the superimposed load under the mat by about 280 lb per sq ft, which would then require the foundation to be another 2-1/2 ft deeper, thus displacing another 280 lb per sq ft of soil. In order to limit the excavation within the realistic maximum of 60 ft, the mat, therefore, was also designed for lightweight concrete, with a specified strength of 6000 psi. The mat reinforcement consists mainly of No. 18 bars, both top and bottom, with varied spacing. The bars were spliced in a staggered manner using mechanical splices consisting of a sleeve into which molten iron is poured. Placement of the mat concrete was made in segments of about 1100 cu yd and adequate measures were taken to control the heat of hydration as discussed in another paper.\* A typical section through the mat is shown in Fig. 1-7.

### DESIGN FOR WIND LOAD

The structures in the Houston area are designed for hurricane-type wind loads. Generally, this is equivalent to 40 lb per sq ft of lateral load applied to the entire height of the building. In order to limit the sway under wind load to an acceptable value (1/600) it was necessary to achieve a modulus of elasticity of concrete of  $3 \times 10^6$  psi. The entire structural system was analyzed on the assumption that the modulus of elasticity will be a minimum of  $3 \times 10^6$  psi and the result of the analysis indicated that the lateral sway of the building would be within the specified limit and that the sway motion would not be perceptible under normal storms.

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\*"Quality Control of High Strength Lightweight Concrete for One Shell Plaza," Paper SP 29-2.

### DESIGN FOR TEMPERATURE

The choice of lightweight concrete for the structure has three distinct advantages in terms of temperature effects. First, lightweight concrete has lower conductivity of heat and therefore takes longer to reach any steady state of temperature.<sup>1</sup> This is a great advantage in overcoming severe temperature changes over a short period of time. The second advantage is a result of the first leading to a lower steady state temperature. The third advantage is that lightweight concrete has a lower coefficient of expansion as compared to normal weight concrete. While the average coefficient of expansion for normal weight concrete is about  $6.5 \times 10^{-6}$  in./in./deg F, the average value for lightweight concrete is generally about  $4.5 \times 10^{-6}$  in./in./deg F. This means that the expansion and contraction due to temperature changes would be about 70 percent of that for normal weight concrete. For the One Shell Plaza building, which is designed for a maximum floor racking of  $3/4$  in. due to temperature changes in the exterior columns, use of lightweight concrete in the columns allowed a larger portion of the exterior columns to be exposed to outside temperature changes.

The location of the glass line was established on the basis of the average temperature of the exterior columns under extreme winter conditions. Fig. 1-8 shows a typical column section with isothermal lines indicating lowest temperature at the outside face of the column and room temperature on the inside face of the column. In the design of the building, the vertical relative movement of the exterior columns at the top of the building was limited to  $3/4$  in. Although this maximum movement did not affect the structural design of the floor slab system by any significant amount, the racking of the floors due to temperature movement was considered in detailing the partitions in the building. It is obvious that if normal weight concrete were used, the temperature movement problem would be much more severe for the same column exposure.

### DESIGN FOR CREEP AND SHRINKAGE

Previously available test results generally indicated that lightweight concrete has considerably higher shrinkage strain as well as increased creep compared to normal weight concrete. Typically, the shrinkage strain in normal weight concrete for a period of 1 year is generally on the order of  $400 \times 10^{-6}$  in. per in., whereas the shrinkage strain for comparable lightweight concrete in the past has been as much as  $600 \times 10^{-6}$  in. per in. In recognition of this large difference, a comprehensive testing program was started, the results of which indicated that the proper mix design and selection of materials for lightweight concrete can effectively reduce the shrinkage strain very close to that for normal weight concrete. The One Shell

Plaza building, therefore, dictated proper choice of aggregates so that excessive shrinkage could be avoided. A series of testing and quality control programs were undertaken, which is reported.\*

It has been generally accepted that lightweight concrete has a higher creep coefficient. However, most of the data available on creep tests have been obtained from tests on flexural members, which is not applicable to compression creep. The author further questioned the validity of data available on compression creep because most of the tests run on compression creep were aimed at understanding the behavior of flexural members. Tests reported in the past almost invariably loaded the compression specimens at full design load as soon as the concrete reached design strength. In an actual building, such as One Shell Plaza, the rate of application of load to the compression members (columns and walls) is dependent on the rate of construction of the floors. Therefore, typically in a 50-story building, the first story column is never loaded more than 2 percent of the design load every week. If we assume that the construction schedule is one floor a week, it seems apparent that this rate of loading would substantially change the realistic creep strain in a column.

At the request of the author, the Portland Cement Association, under the direction of Dr. Eivind Hognestad and Mr. Donald Pfiefer, undertook a series of incremental creep tests on 6 by 6 ft columns reinforced approximately 3-1/2 percent to simulate the same properties in the One Shell Plaza building columns. The results to date are surprisingly different from what would have been normally considered to be the creep strain based on earlier tests.<sup>2</sup> The typical creep strain curve is plotted against the test results of incremental time creep tests in Fig. 1-9.

The differential creep effect between exterior columns and interior shear wall core may cause high stresses in the floor slab, as well as large redistribution of loads, if the span is relatively small. For One Shell Plaza, the clear span between exterior columns and the shear wall core being 36 ft, the relative movement of the exterior column with respect to the interior core does not cause high secondary moments in the slab structure.

This is not true, however, for three interior columns located close to the end of the shear walls. The purpose of providing the interior columns was to capture more open space in the floor area for rentable purposes. However, the closeness of these columns to the shear wall raised some serious questions relative to differential movements due to creep and shrinkage. The large columns are reinforced with an average of about 3.5 percent of the gross area, whereas the adjacent shear walls are reinforced on an average with about 3/4 percent. This large difference of percentage of reinforcement in the two

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\*"Quality Control of High Strength Lightweight Concrete for One Shell Plaza," Paper SP 29-2.

adjacent vertical members will normally cause large differential movements due to creep. If one takes earlier tests run on columns, without considering the incremental load effect, as reported by Pfiefer<sup>3</sup> and Seed,<sup>2</sup> it will be obvious that the average creep strain for 3-1/2 percent reinforcement would be on the order of  $300 \times 10^{-6}$  in. per in., whereas the average creep strain for 1/2 percent reinforcement would be on the order of  $600 \times 10^{-6}$  in. per in. If this relationship were true also for incremental loadings, the relative creep movement between the interior columns and the shear walls would have been approximately 3.2 in.

In practice relative displacement within 10 ft of the floor area should not be more than 1/4 in. Therefore, theoretically, the resulting construction would invariably lead into serious architectural and structural problems. As discussed earlier, however, the author's conclusion that creep strain under incremental loading is substantially less than creep strain under sudden full load applied at 28 days was proven to be correct as shown by the PCA test results. Based on the PCA test results and further theoretical evaluation of incremental creep,<sup>4</sup> a system of connecting links was designed to hold the relative creep movement between columns and shear walls to an acceptable minimum. The purpose of the link beams is to transfer some load from the large interior columns into the shear wall so that the creep of the interior columns will be further reduced while the creep strain of the shear wall will increase, thereby helping to equalize the creep strains.

The design method of such redistribution is discussed in Reference 4.\*

#### INSTRUMENTATION AND MEASUREMENTS ON THE PROJECT

A project of this nature and size, where special design methods have been used, should be the basis for developing future design criteria. To do so, such projects should be instrumented for verifying actual behavior characteristics. The One Shell Plaza building has been instrumented to obtain the following categories of information:

1. Pressure distribution under lightweight concrete mat which will give a better understanding of the soil structure interaction of such deep foundations using lightweight concrete.
2. Heat of hydration of mass concrete using lightweight aggregates. The mat was instrumented with thermocouples which validated the initial test results using  $8 \times 8 \times 8$  ft cubes to simulate the mass concrete.

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\*Designing for the Effects of Creep, Shrinkage and Temperature, SP 28-9, to be published, ACI, Detroit, 1971.

3. Strain measurements of the exterior columns, interior columns, and the shear wall core.
4. Floor deflections.

Only the program for measurement of creep and shrinkage in the vertical elements will be discussed in detail. The creep measurement program was jointly undertaken with Portland Cement Association. Vertical creep and shrinkage strains are being measured on a number of exterior columns, interior columns and at some locations on the interior shear wall core. A total of four floor levels were chosen—the basement, eighth floor, twenty-fifth floor, and fortieth floor. As construction continues, strain readings have been taken using a 20 in. Whittemore gage. To date, these readings indicate that the laboratory test readings were realistic. A final report on the actual measurements will be available in about 2 years.

#### QUANTITIES OF MATERIALS AND RELATIVE COST OF STRUCTURE

As a major reinforced concrete building project, it would be of interest to discuss the total quantities of various materials used and the relative cost. Approximately 90,000 cubic yards of concrete was used for the entire project for a total covered area of 1.4 million sq ft. About 50 percent of this concrete was 6000 psi and the remainder 4500 psi. The total amount of reinforcing steel used in the project was approximately 7500 tons, of which about 60 percent is ASTM A615 - Grade 75. The remaining steel, mostly in the slab structure, is ASTM 615 - Grade 60. The total quantity of excavation for the foundation was approximately 90,000 cubic yards. The total cost of construction of the superstructure, including the mat foundation but excluding the cost of excavation, was estimated (1965) to be approximately \$4.60 per sq ft. It should be pointed out that this cost of construction of the superstructure, combined with the reduction of cost of the window wall system because of the closely spaced exterior columns, may mean as much as a 25 percent reduction over some of the more traditional methods of building a similar building.

A point, however, must be made regarding the rate of construction and its effect on the total real cost of construction. Inasmuch as concrete technology from purely the engineering point of view has been refined to a point that ultra high-rise buildings in the future can be economically built in reinforced concrete, the technology of construction methods for reinforced concrete buildings must not lag behind the advancement made in the engineering aspects. Time being equated to money, total real economy can be achieved only if the construction technology matches materials and engineering technology. It is only with this combination that the continuing socio-economic need for tall concrete buildings can be met successfully.

## CONCLUSION

The main items of design and analysis of the world's tallest (as of 1970) reinforced concrete building has been discussed. It is the use of lightweight concrete that made this project economically competitive and architecturally compatible.

It was shown that proper choice of lightweight aggregate for the concrete is imperative for successful design and construction of an ultra high-rise building using lightweight concrete.

A number of areas of special studies and possible problems have been pointed out and it is hoped that engineers will further refine some of the methods discussed in this paper for future application to ultra high-rise buildings.

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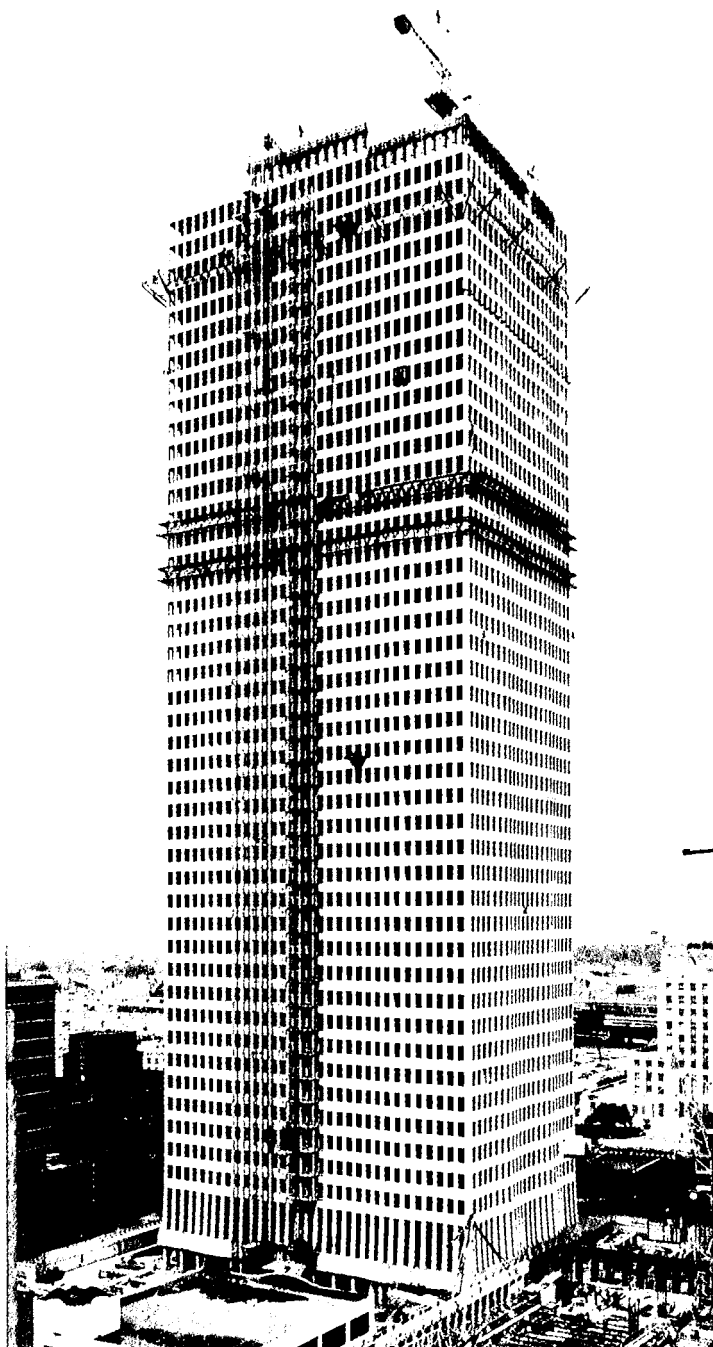


Fig. 1-1—General view of One Shell Plaza, Houston under construction

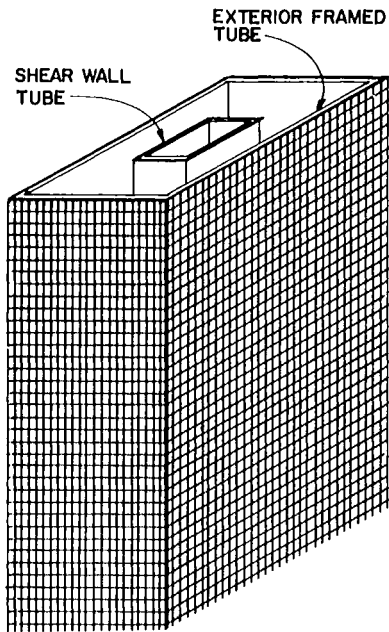


Fig. 1-2—Schematic concept of tube-in-tube system

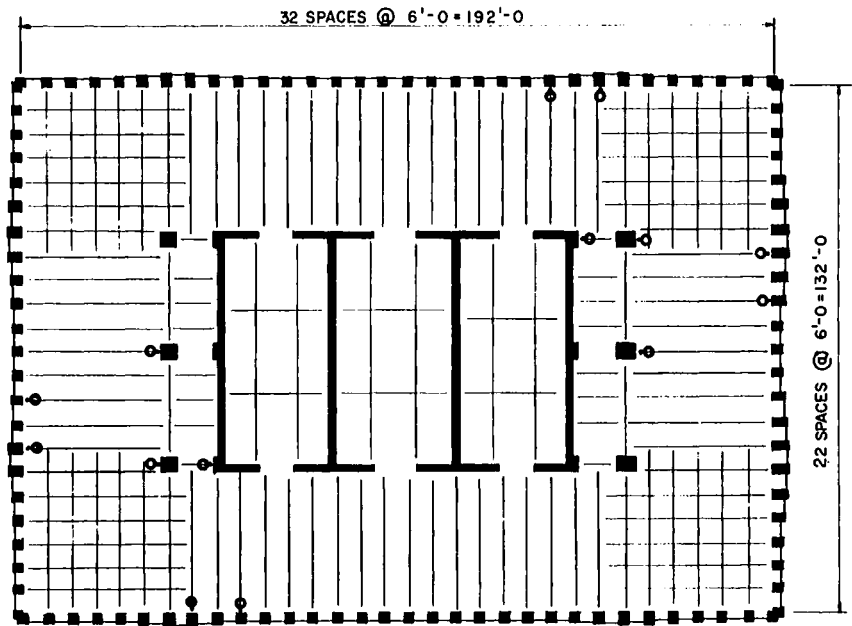


Fig. 1-3—Typical structural floor plan

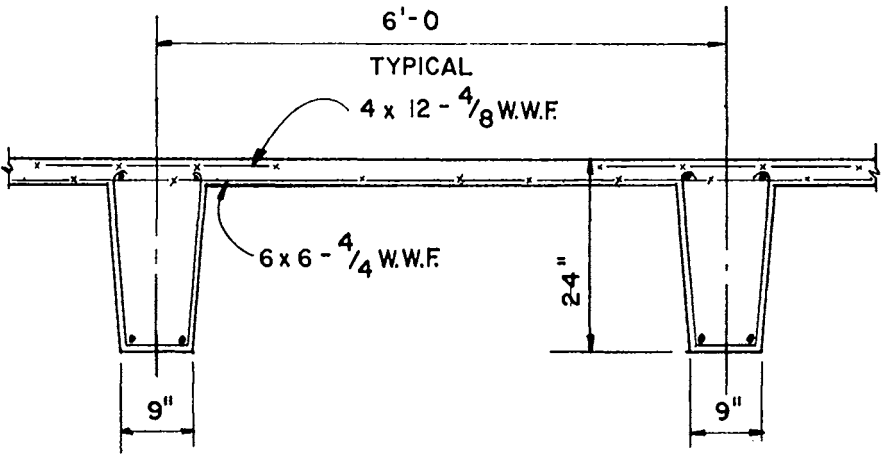


Fig. 1-4-Section through joist-and-slab floor construction

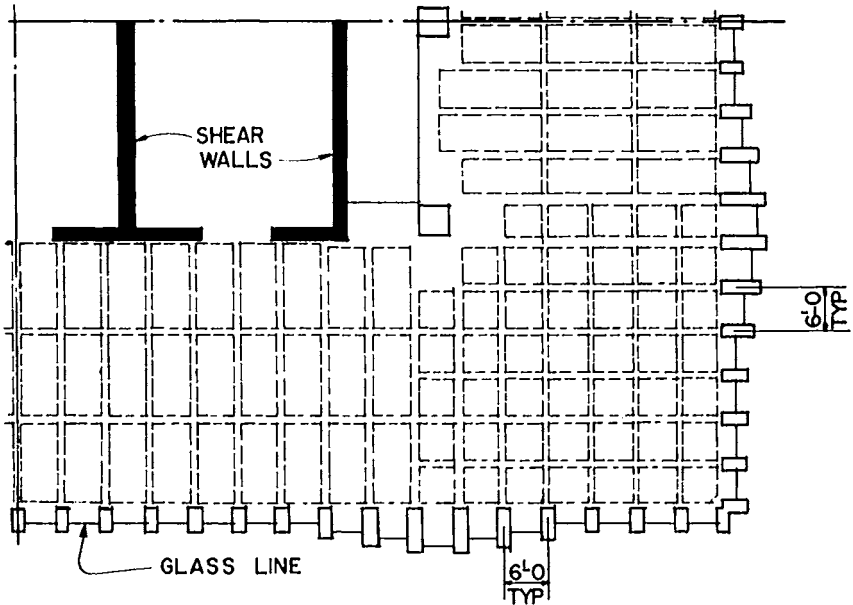


Fig. 1-5-Typical corner area floor plan

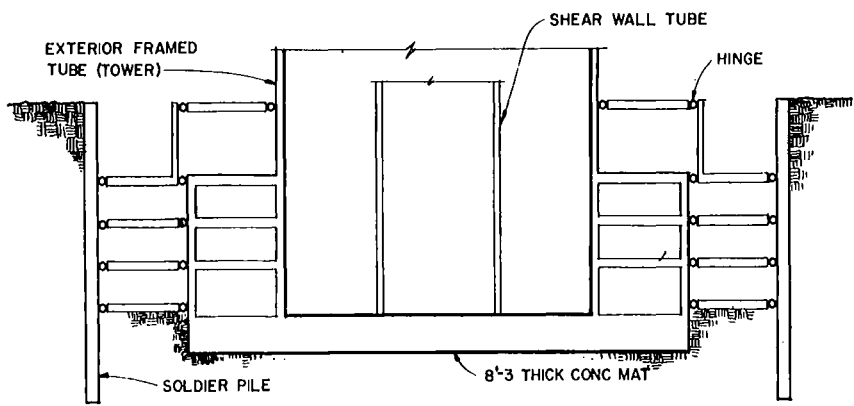


Fig. 1-6-Section through floating mat foundation

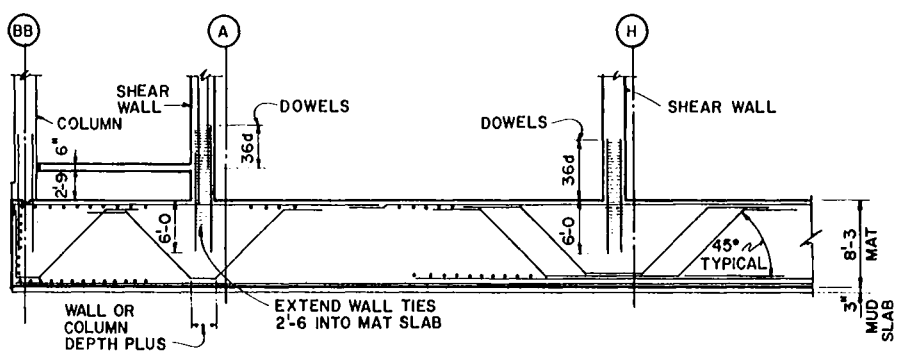


Fig. 1-7-Mat foundation details

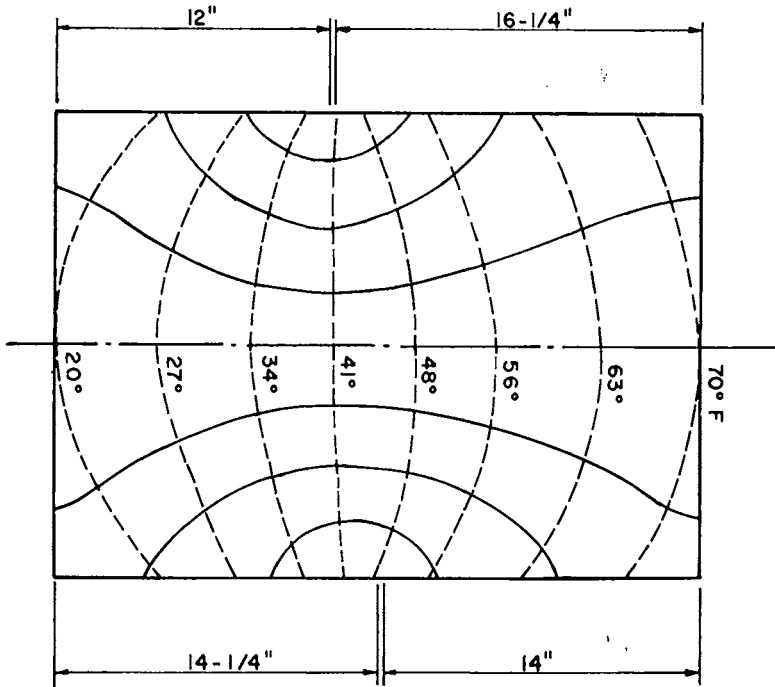


Fig. 1-8—Isotherms through a typical column section

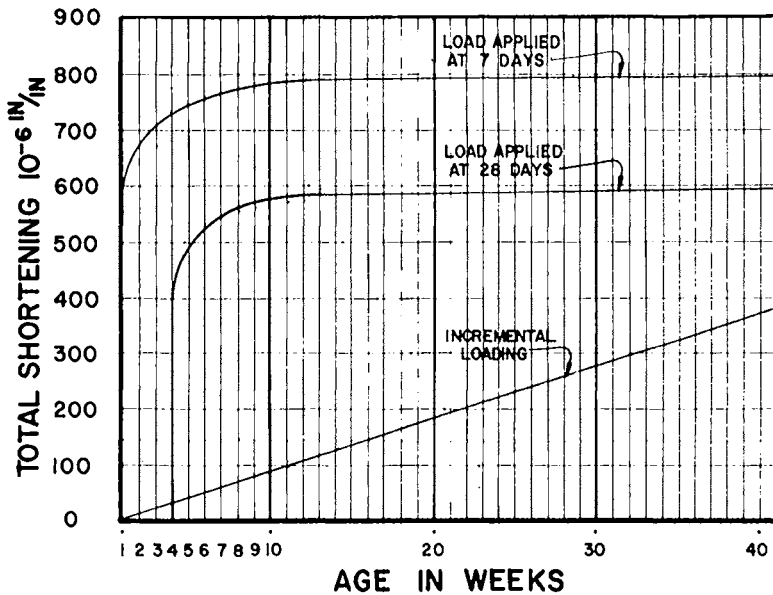


Fig. 1-9—Creep strain for incremental loading versus full loading

# QUALITY CONTROL OF HIGH STRENGTH LIGHTWEIGHT CONCRETE FOR ONE SHELL PLAZA

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The success of the 52-story One Shell Plaza building in Houston depended on the use of high strength lightweight concrete. To achieve 6000 psi lightweight concrete at the site for the mat foundation, columns, walls, and spandrels a rigorous quality control program was developed and followed through the entire project. The authors discuss the development of the design mix, tests made for establishing design criteria, and procedures for plant and field inspection to maintain quality control throughout the project. A checklist for quality control for future high-rise lightweight concrete buildings is proposed.

**Keywords:** coefficient of variation; cold weather construction; compressive strength; concrete construction; concrete cores; foundations; high-rise buildings; high-strength concretes; hot weather construction; inspection; lightweight aggregate concretes; mass concrete; mix proportioning; mixing plants; office buildings; quality control; reinforced concrete; skyscrapers; temperature rise (in concrete).

## INTRODUCTION

The 52-story One Shell Plaza building in Houston, Texas (Fig. 2-1) which was designed with a tube-in-tube structural system, could be economically feasible only if high strength, lightweight concrete was used. From a structural design point of view, the height of the building (714 ft) combined with the requirements of high wind load, necessitated selection of the best possible lightweight concrete aggregate in the Houston area to produce concrete with structural properties comparable to normal-weight concrete. The structural, architectural proportions of the building required 6000 psi lightweight concrete for the 8 ft 3 in. thick mat, interior and exterior columns, and shear walls. For the 36 ft span floor slab construction, 4500 psi strength was required. In addition to these strength requirements, it was also necessary to achieve a minimum modulus of elasticity of  $3 \times 10^6$  psi for the high-strength concrete. And as if to make

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matters more challenging, it was specified that the maximum creep under compression and the maximum drying shrinkage of the lightweight concrete be comparable to good quality, normal-weight concrete.

To most practicing engineers, it is probably obvious that the properties of the lightweight concrete for One Shell Plaza were difficult to achieve. After all, it is normally taken for granted that, while lightweight concrete reduces the weight of structures, it generally results in considerably more creep and shrinkage than normal-weight concrete.

Inasmuch as the physical properties of lightweight concrete are primarily affected by the type of lightweight aggregate used for such concrete, it was imperative that the most suitable lightweight aggregate be used in this project. While the general definition of lightweight aggregate includes expanded clay or shale produced by a rotary kiln or by sintering, the physical properties of aggregate seem to vary with the bloating characteristics of the raw material and the processing equipment used in manufacturing the aggregate. A comprehensive testing program was, therefore, initiated by the architect-engineer, Skidmore, Owings and Merrill, to make an objective and comparative evaluation of a number of lightweight aggregates available in the Houston area. A large number of tests were made on the aggregates separately and in trial batches of concrete. The final

choice of aggregate was based on these tests, the results of which are discussed in this paper.

The choice of aggregate was only the first step in arriving at the optimum concrete design mix. After the aggregate was chosen, a large number of concrete mixes were made with it and tested for strength, modulus of elasticity, shrinkage, creep, and heat of hydration. Based on these tests, the final design mixes were selected.

It was established in the preliminary phase of the project that a comprehensive quality control program would be required at both the concrete plant and the site to guarantee the consistent production and placement of a high-quality concrete. Along with the Southern Inspection Service, Inc., a detailed system of controls at the plant and at the job site was developed.

This paper will discuss all testing performed and the evaluation of the test results from the planning of the project to its present point of completion (45 of 52 floors).

#### DEVELOPMENT OF THE DESIGN MIX

In the development of the concrete for One Shell Plaza, the following characteristics were set as essential properties which the concrete mix had to pass to satisfy structural design criteria:

1. The building was designed to support a self-weight of 115 lb per cu ft for all cured concrete incorporated in it. Early tests indicated that the difference in the weight of the cured concrete and the fresh unit weight of the mix was on the order of 5 to 6 lb per cu ft. To allow for possible variations in the constituents at the batching plant, an optimum fresh unit weight of 116 lb per cu ft was established.
2. The splitting cylinder strength factor had to be at least 5.5 because this value was used to determine allowable shear stresses in the structural members.
3. A minimum modulus of elasticity of  $3.0 \times 10^6$  psi had to be maintained since it formed the basis for all deflection calculations and affected the distribution of forces and moments between columns and slabs.
4. The allowable shrinkage at 28 days was set at  $600 \times 10^{-6}$  in./in., based on available literature and previous experience. It was felt that a shrinkage in excess of this would introduce undue internal stresses into the structural members, very possibly resulting in unacceptable differential movement and cracking.
5. The basic and most obvious requirements were the specified compressive strengths of the selected mixes at 28 days.

Other criteria were taken into consideration, such as air content, slump, water-cement ratio, concrete temperature, and the temperature rise of the concrete mass, but only insofar as each affected the critical design parameters.

In the Houston area, several different sources of lightweight aggregate and cement were available. To develop the optimum design mix from the constituents on the basis of cost, performance, and a control standpoint, the following detailed comparisons were carried out by Southern Inspection Service.

Available data were reviewed on four aggregates which were readily available in sufficient quantity in the area and a fifth aggregate which was not readily available in the area, but had been used successfully by the architects on earlier projects.

A summary of the five aggregates is shown in Table 2-1. From the table, it appears that a correlation exists between unit weight, specific gravity, absorption, porosity, and degree of saturation. If these samples are representative of lightweight aggregate generally, it appears that a decrease in unit weight will generally increase absorption, porosity, and saturation, while decreasing the specific gravity.

Of the 5 aggregates reviewed, only A and B were selected for further testing and study. Aggregate A was selected because it possessed properties very similar to those of Aggregate E which had proved to be a satisfactory aggregate on earlier projects. Aggregate B was selected because it was the preference from an economic standpoint.

Concrete test specimens were made with each aggregate, holding all other constituents constant, with a cement factor of 7-1/4 sacks per cu yd (680 lb per cu yd) and tests were conducted for modulus of elasticity, shrinkage, splitting cylinder strength factor, and compressive strength (see Table 2-2). Based on these tests, Aggregate A was finally selected because B did not satisfy the modulus of elasticity requirement and it was felt that the lower absorption of A would result in a concrete mix easier to control.

Two normal-weight sands, economically available locally, were selected as the fine aggregates. One was too coarse to meet ASTM C-33, and the other too fine. By blending them, about 80 percent coarse to 20 percent fine, a satisfactory gradation was achieved.

With the aggregates selected, a study was made of the four major brands of Type I cements available locally. Concretes were made as nearly alike as practicable using Aggregate A with the cement being the only variable (see Table 2-3). From this study, the mix using Brand A cement was selected because of its high 28-day strength and high modulus of elasticity. Brand C was selected as an alternate

and was used for a period during the job when Brand A was not available.

Small scale, reinforced lightweight concrete column specimens (6 x 6 x 36 in. high) were cast and test-loaded at the Portland Cement Association Research Laboratory to simulate the conditions to be expected in the first floor columns on One Shell Plaza during construction. The columns were loaded with 50 equal weekly increments, each 2 percent of the full load.

These tests indicated that the shrinkage and creep characteristics of the lightweight concrete could be expected to differ very little from columns containing normal weight concrete.

Selected columns in the One Shell Plaza tower have been instrumented and readings are presently being taken to verify these findings. To date, the selected mixes have produced consistently high-quality concrete and no conditions of unpredicted behavior have been encountered.

#### DESIGN CRITERIA FOR MASS CONCRETING OF THE MAT

Because of the mass of the 8 ft 3 in. foundation mat, it was felt that a study to determine the temperature rise of the concrete was essential. Based on previous experience and available data, it was felt that to maintain the design strength of 6000 psi at 28 days, the job conditions for mass placement would require that a maximum temperature of 165 F. for the concrete in place should not be exceeded.

A total of 30 trial mixes were made utilizing various kinds of Type II (low heat of hydration) cement in amounts of 6-1/2 to 7-1/2 sacks per cu yd (610 to 705 lb per cu yd). Of the 30 trial batches, 26 had a pozzolanic fly ash material, utilized as part of the cement.

To simulate the conditions which would be encountered in the mat, an 8 x 8 x 8 ft cube was cast using the most promising of the trial mixes. The cube was instrumented with thermocouples to read rise of temperature due to heat by hydration. The selected design used 6-3/4 sacks (635 lb) of Type II cement, 80 lb of fly ash, and 81 percent of the mixing water was replaced with ice. This reduced the temperature of the concrete mix at the plant to 75 F. From Fig. 2-2, it can be seen that the temperature of the mix on arrival at the job site was 83 F. For the 5-hr period during the effective retardation, no significant hydration or heat gain was noted. After this period, the concrete temperature increased rapidly until the 80th hr when it began to subside. This resulted in a total temperature differential of 95 F. The maximum temperature recorded (178.5 F.) was higher than the allowable required by the specification. It was, therefore,

decided to perform additional tests. Because 8 x 8 x 8 ft concrete cubes are rather hard to dispose of, it was decided to perform this second series of tests on 1 x 1 x 1 ft cubes. These cubes were encased in 3/4 in. plywood forms insulated with 4 in. of polyurethane to simulate mass concrete (see Fig. 2-3).

From a study of these and previous test results, it was evident that the temperature rise in a 8 ft mat would be in the order of 95 to 100 F. regardless of placing temperature. Therefore, to satisfy the design requirement, the placing temperature of this concrete would have to be held to 60 F. maximum. Calculations indicated that to maintain this temperature, 125 to 175 lb of ice per cu yd would be required. The exact quantity had to be computed on the basis of the ambient temperature, water temperature, and cement and aggregate temperature at the time of mixing.

A 60 F maximum temperature was maintained during the placement of all concrete in the mat foundation.

#### PLANT AND SITE INSPECTION

To ensure that the supplier had facilities capable of providing the required quality concrete consistently and in sufficient quantity (1200 cu yd per week), the initial step was to perform a complete inspection of the two batch plants which were to be used. Both plants had an electronic, automatic punch-card system to weigh and record all materials, thereby providing a permanent weight record of each load of concrete delivered. The scales were all checked and found to have been certified by an state inspection agency. All trucks were inspected and found to be in good operating condition.

A check of stockpiles, however, revealed possible contamination of aggregates and some segregation problems. A detailed list of work to be done at each plant before it would be accepted was given to the concrete supplier. The list consisted of such items as raising the concrete pad for lightweight aggregate storage, building shades over the lightweight aggregate stockpiles for summer, installing a sprinkler system for prewetting lightweight aggregate, and installing sump pumps for removal of excess water from storage areas. All of these items were corrected prior to any concrete being produced for the project.

From experience, it was evident that the lightweight concrete mix design could not satisfy the workability and strength requirements during construction without adjustment at the plant for weather and material conditions. Changes in temperature, water content, and unit weight of the aggregate resulted in concrete variations of an unacceptable magnitude which had to be compensated for at the plant by necessary changes in the mix proportions.

A full time plant inspector was given complete authority to adjust mixes as necessary, as well as to reject any unsatisfactory mixes.

A 10 x 20 ft portable laboratory was placed at each batch plant for the inspector and, at the start of each day, the following tests were performed:

1. A sieve analysis was made of a sample of the lightweight aggregate to be used that day.
2. The unit weight of the lightweight aggregate was checked.
3. The absorption and amount of free water in the lightweight aggregate was determined.
4. A sieve analysis was made of a sample of the two types of sand.
5. The blended sand was checked for ASTM C-33 conformity.
6. The moisture content of the sand was checked.
7. Unloaded and stocking of materials were observed to assist in preventing segregation.
8. Sprinkling of lightweight aggregate was checked to insure uniformity of water absorption.
9. Mix adjustments were made and weighing and mixing each load of concrete were observed.
10. During hot weather, the ice was inspected for weight and consistency (shavings).
11. The slump and temperature of the concrete were recorded.

The distance from the batch plant to the site was approximately 5 miles. The trip normally took from 15 to 25 min.

Upon arrival of the truck at the site, the concrete was checked again. The unit weight, slump, air content, and temperature of the concrete were checked twice per 100 cu yd. One set of five standard test cylinders was also made from each 100 cu yd for testing in the laboratory at 3, 7, 28, and 56 days to verify design strength and modulus of elasticity.

Based on the various test results and workability evaluation during the initial phase of the job, it was felt that if the following criteria were met, an acceptable grade of concrete would result:

1. Wet weight per cubic foot not more than 121 lb per cu ft.
2. Slump from 3-1/2 to 4 in.
3. Entrained air of 3 to 5 percent of the volume of the concrete.

4. The addition of no more than 1-1/2 gal. of water per yd at the site to develop the above mentioned slumps.
5. No additional water permitted after 1 hr from time of batching.
6. Time elapsed from time of batching to time of placing not to exceed 1-1/2 hr.
7. Temperature of the concrete not greater than 90 F. (except for mat).

To provide the plant inspector with valid and timely information for making adjustments in the mix, direct telephone communication was established between the job site and the batch plant.

Upon arrival of the first load of concrete at the job site each day, the job site inspector would perform the tests mentioned earlier, accept or reject the batch of concrete, and telephone all concrete data back to the inspector at the plant.

Of the data provided, the adjustment was predominately based on unit weight, amount of water added at the site, air content, and temperature. If the unit weight was not  $116 \pm 2$  lb per cu ft, the inspector at the plant would either increase or decrease the volume of lightweight aggregate, which would also increase or decrease the sand content. This would bring the weight per cubic foot within the specification. If the entrained air was not within the 3 to 5 percent range, he would increase or decrease the quantity of air-entraining agent. This would not only change the air content, but would also affect the unit weight. Weight and air content, therefore, had to be considered jointly.

Based on the concrete temperature and amount of water added at the job site, the inspector could determine the loss of slump through absorption and evaporation and rise in temperature between plant and job site. He could then make adjustments by adding or deleting ice and increasing or decreasing water. Once these adjustments were properly made, the concrete normally arrived at the job site very close to an ideal mix.

Of the variables involved in adjustments, the weight per cubic foot of the aggregate was the most critical. If the voids, porosity, or gradation of the aggregate changed, so did its weight and absorption. This also affected the water required and the water addition at the job site. The mix design was based on a set volume of lightweight aggregate (20 cu ft per cu yd) irrespective of its weight; compensating adjustments for differences in the determined fresh unit weight were made by varying the other concrete constituents of the lightweight concrete.

## ANALYSIS OF FIELD TEST RESULTS

Copies of all test results and recorder data related to quality control of the lightweight concrete were sent to Skidmore, Owings, and Merrill. These data were then punched on computer cards for easy access, if required, and to allow statistical analyses to be performed easily.

Because of time and economic considerations, analyses were confined to a monthly statistical analysis of the recorded concrete cylinder strengths and all other data were simply checked for conformity to specifications. The statistical analysis selected and recorded the minimum and maximum cylinder strengths for each month and calculated the interval, mean and median strengths, standard deviation, coefficient of variation, and the percentage of tests below design strength.

A graphical illustration of the monthly mean strength and coefficient of variation over time is shown in Fig. 2-4 through 2-6.

Fig. 2-4 indicates no consistent pattern to the coefficient of variation for either the 4500 or 6000 psi concrete over the period of the job thus far. There is, however, a very clear difference between the coefficient of variation for the 6000 psi and 4500 psi concrete. Over the period of almost the entire job, there has been greater variation in the 4500 psi concrete than in the 6000 psi concrete.

Probably the main factor contributing to this variation is the fact that the 4500 psi concrete was for slabs and had to be finished, while the 6000 psi concrete was placed in wall and column forms. Modifications for workability of the 4500 psi concrete were regularly requested by the finishing crews and modifications within set limits were often made.

Readily apparent from Fig. 2-5 and 2-6 is a fairly good relationship between cylinder strengths at all ages of specimens. The 3-day strength ranged from 25 to 60 percent of the 28-day strengths the 7-day values ranged from 75 to 98 percent, and the 56-day strength ranged from 98 to 113 percent.

From Fig. 2-5 and 2-6, it can be seen that on all but 5 occasions, the required design strength at 28 days was satisfied. Two of the periods of low strength occurred during start up, and the remaining three were very likely a direct result of an unauthorized brand of cement being substituted. The authors cannot stress too strongly the importance of not deviating from the design mix in any way. Of the five times when the concrete strength was deficient, only two did not reach design strength at 56 days and required certain local repair.

## CORRELATION OF CORE TEST RESULTS

Some of the lower floor columns visually indicated weaker concrete at the top of the concrete lifts. These columns were cored for evaluation. Core samples, 2-1/4 in. in diameter, were taken horizontally, through the visually weak zones of two selected columns. These cores broke at strengths as low as 4000 psi.

In view of the preliminary findings, the following steps were taken:

1. The upper portions of all columns were visually inspected.
2. Where the quality of the concrete was visually poor, Swiss impact hammer tests were performed.
3. All columns were cored in which the impact hammer tests were found to be less than 43 (equivalent to 6000 psi).
4. A number of cores were taken in what was felt to be an area of high-quality concrete, based on impact hammer readings and visual inspection.

By comparing core test results from the suspected areas with those from good areas, the contractor was required to replace all concrete in areas where the core strengths were less than 5000 psi. In areas where the core strength was between 5000 and 5400 psi, only partial repairs were made and in areas where the cores broke above 5400 (10 percent below the specified 6000 psi), no repairs were required.

## HOT WEATHER CONCRETING

Summer temperatures normally ranged between 90 and 110 F. and unshaded stockpiles often reached temperatures of 100 to 125 F.

To help reduce initial concrete temperatures, the stockpiles of lightweight aggregate were put under a roof and were kept constantly moist under a fog spray of water. Trucks were painted white to absorb less sun radiation, and they were cooled between loads by spraying the outside of the drum with cold water.

The site inspector had orders to reject any truck which arrived at the site with a concrete temperature in excess of 90 F (60 F. for the mat).

Experience showed that the average rise in temperature which could be expected from the plant to the site was about 10 F. Therefore, enough ice was introduced into the mix so that it left the plant at 80 F. or less. Normally, approximately 8 lb of ice per cu yd brought about a reduction of 1 F in temperature of the mix. During hot spells, normally between 50 and 125 lb of ice per truck had to

be added. The amount of air-entraining agent used also was affected by high temperatures and had to be increased. Even under these circumstances, a uniform and well-controlled mix was produced. The difficulty was to predict the quantity of ice which would be required 24 hr in advance. There were no facilities available for storing ice at the plant. Therefore, a hot-weather mix was reviewed each day for the following day's work, based on available weather reports.

Fig. 2-4 indicates that no increase in the coefficient of variation was encountered on the job during the summer months in spite of the additional control problems introduced by hot weather.

The importance of maintaining good quality control in hot weather can best be seen from Fig. 2-7. In spite of all precautions taken, and the uniform consistency of the mix throughout the year (indicated by the relatively constant coefficient of variation), there was still a reduction in strength of about 10 percent during the hot weather months. It should be recognized, however, that even with these reduced strengths, the design criteria were satisfied.

#### COLD WEATHER CONCRETING

The coldest temperature encountered in the Houston area was in the low 20's. Normal temperature range for the winter months is 35 F to 50 F.

There are only about 10 days in a winter season when the temperature drops below 32 F. Because of this, no provisions were made for heating and protecting the concrete. The criteria for cold weather was not to mix or place concrete when the temperature was expected to go below 40 F within the following 48 hr.

Acceptable lower temperatures, however, did have an effect on form stripping time. Columns and spandrels were gang formed so the stripping time was of great importance to the progress of construction. A strength requirement of 1500 psi was set as the criterion for removing the forms.

A study was carried out to evaluate possible accelerated mixes which could be used during the colder periods. Mixes using Type III cement (see Table 2-4) and Types I and III with 1 percent calcium chloride (see Table 2-5) were studied.

In this study, as in the Type I cement study earlier, Brand A cement was selected because of higher 28-day strength and higher modulus of elasticity and Brand C cement was selected as an alternate.

The contractor chose to use Type III cement with a normal set admixture allowing him to strip forms at approximately 20 hr when

the temperature dropped to the low 40's. These times are comparable to those of the normal mix in warm weather.

#### QUALITY CONTROL CRITERIA FOR FUTURE HIGH STRENGTH LIGHTWEIGHT CONCRETE PROJECTS

A few years back, it would not have been considered practical to use 6000 psi lightweight concrete cast-in-place construction for a high-rise building with 90,000 cubic yards of concrete. The authors' experience to date has proved conclusively that it is not only possible to build ultra high-rise buildings in high strength, lightweight concrete but also that such buildings can be built without any serious quality control problems if a well thought-out quality control program is established at the time of the design of the structures. On the basis of the experience on this project, the authors propose the following items as a check list for future high-strength, lightweight concrete cast-in-place construction:

1. **Lightweight Aggregate.** The choice of lightweight aggregate should be based on its weight, its absorption of water, and its resistance to freeze-thaw deterioration. The specification should, therefore, clearly define the maximum 24-hr absorption of water, as well as the minimum number of cycles for freeze-thaw deterioration. In general, 24-hr water absorption should not exceed 15 percent.
2. **Mix Design.** The specifications should include the requirement of at least four alternate design mixes, using different brands of cement and sand. The mix producing the best combination of material properties should be accepted.
3. **Split Cylinder Strength.** If shear properties are important, the split cylinder strength should be specified. Normally, a split cylinder strength factor of 5.5 should be considered adequate.
4. **Modulus of Elasticity.** If deflection of floor slabs is critical in the design, the required modulus of elasticity should be specified. The same may be true for controlling lateral sway of the building.
5. **Unit Weight of Concrete.** One of the best methods of consistently controlling the properties of the accepted mix design is to control the fresh unit weight of concrete at the job site. This should be specified if such test results are available.
6. **Placement.** The specifications should clearly require proper placement practices to be followed. These commonly known practices should be repeated in the specifications if necessary. Vibration of concrete in columns, particularly at the top, must

be planned in designing the formwork and followed meticulously throughout the job.

7. Plant Inspection. Full time plant inspection by an approved, qualified inspection agency should be required. It is generally much more difficult to control concrete once it has arrived at the job site. The plant inspector should make a continuous check of the concrete produced at the plant and alter the proportions as required by weather or other material conditions.
8. Site Inspection. Site inspection should include control of water added at the job site. Normally, not more than one gallon per cubic yard of water should be allowed to be added at the job site. Site inspection should also include, among other normal tests, checking of the fresh unit weight of lightweight concrete.

### CONCLUSION

The authors have presented significant aspects of a quality control program for high-strength, lightweight concrete for high-rise construction, beginning with the design phase of the building and continuing through to the end of construction. The world's tallest reinforced concrete building (714 ft) was economically feasible only through the use of high strength, lightweight concrete. The success of the project depended on the comprehensive quality control program.

The authors hope that the results and conclusions presented in the paper will lead to further use of high-strength, lightweight concrete for ultra high-rise buildings.

### ACKNOWLEDGMENTS

The success of a major project depends on many individuals. Among many who were personally involved and helpful, the authors wish to acknowledge Dr. Joseph P. Colaco for his initial involvement in design of this building; Mr. William Goldfield and Mr. John Carson, field representatives of Skidmore, Owings and Merrill; Mr. E. M. Scarce, of Southern Inspection Service, Inc., who did many of the initial tests of the program; Mr. Warren Bellows, of W. S. Bellows Construction Company, who continually supported the quality control program; and Mr. John Harris, project manager of Gerald D. Hines Interests, owners of the project, whose continual support made the implementation of all the phases of quality control possible.

TABLE 2-1—PHYSICAL CHARACTERISTICS OF LIGHTWEIGHT AGGREGATES AVAILABLE IN THE HOUSTON AREA.

| Aggregate | Unit weight, lb/cu ft | Specific gravity | Absorption, % 40 days | Porosity | Degree of saturation @ 40 days |
|-----------|-----------------------|------------------|-----------------------|----------|--------------------------------|
| A         | 48.1                  | 1.43             | 11.0                  | 0.43     | 0.37                           |
| B         | 42.0                  | 1.41             | 23.5                  | 0.44     | 0.76                           |
| C         | 38.1                  | 1.27             | 22.0                  | 0.47     | 0.62                           |
| D         | 35.8                  | 1.18             | 32.3                  | 0.48     | 0.80                           |
| E*        | 45.3                  | 1.36             | 11.4                  | 0.40     | 0.39                           |

\*Not readily available.

TABLE 2-2—STRUCTURAL PROPERTIES OF LIGHTWEIGHT AGGREGATE CONCRETE

| Lightweight aggregate | Modulus of elasticity, x 10 <sup>6</sup> psi | Shrinkage, x 10 <sup>6</sup> in./in. | Splitting cyl. strength ratio | Compressive strength, psi |
|-----------------------|----------------------------------------------|--------------------------------------|-------------------------------|---------------------------|
| A                     | 3.5                                          | 304                                  | 5.53                          | 6740                      |
| B                     | 2.7                                          | 450                                  | 5.63                          | 6190                      |

TABLE 2-3-CONCRETE TEST DATA FOR BRANDS OF CEMENT  
AVAILABLE IN THE HOUSTON AREA.

| Cement brand | Sacks per cu yd | Slump, in. | Weight, lb/cu ft | Air content, % | Compressive strength, psi | Modulus of elasticity, $\times 10^6$ psi |
|--------------|-----------------|------------|------------------|----------------|---------------------------|------------------------------------------|
| A1           | 7-1/2           | 4-1/2      | 118.2            | 3.4            | 7261                      | 3.5                                      |
| B1           | 7-1/2           | 4-1/2      | 117.0            | 3.5            | 6779                      | 3.1                                      |
| C1           | 7-1/2           | 4          | 119.6            | 5.0            | 7015                      | 3.4                                      |
| D1           | 7-1/2           | 4-1/2      | 118.4            | 5.2            | 6470                      | 3.3                                      |

TABLE 2-4-CONCRETE TEST DATA FOR BRANDS OF HIGH-EARLY  
STRENGTH CEMENT AVAILABLE IN THE HOUSTON AREA.

| Cement brand | Sacks per cu yd | Slump, in. | Weight, lb/cu ft | Air content, % | Compressive strength, psi | Modulus of elasticity, $\times 10^6$ psi |
|--------------|-----------------|------------|------------------|----------------|---------------------------|------------------------------------------|
| A3           | 7-1/2           | 4-3/4      | 117.0            | 5.0            | 7680                      | 3.3                                      |
| B3           | 7-1/2           | 4-3/4      | 116.8            | 5.0            | 6453                      | 3.0                                      |
| C3           | 7-1/2           | 4-1/2      | 117.9            | 5.2            | 7340                      | 3.3                                      |
| D3           | 7-1/2           | 4-3/4      | 118.7            | 5.5            | 7140                      | 3.2                                      |

TABLE 2-5-HOURLY CONCRETE STRENGTH FOR FORM STRIPPING.

| Type<br>Cement | Sacks<br>per cu yd | Admixtures | Air<br>Temp.,<br>deg F | Age of specimen in hr |      |      |      |      |      |      |      |      |
|----------------|--------------------|------------|------------------------|-----------------------|------|------|------|------|------|------|------|------|
|                |                    |            |                        | 16                    | 17   | 18   | 19   | 20   | 21   | 22   | 23   | 24   |
| III            | 7-1/2              | None       | 59                     | 1010                  | 1060 | 1180 | 1480 | 1570 | 1640 |      |      |      |
| III            | 7-1/2              | None       | 41                     | 820                   |      | 870  |      | 1170 |      | 1400 | 1510 | 1690 |
| I              | 7-1/2              | 1% CaCl    | 64                     |                       |      | 780  |      |      | 1170 |      |      | 1500 |
| I              | 7-1/2              | "          | 46                     |                       |      | 955  |      | 1060 |      |      |      | 1460 |
| I              | 8                  | "          | 66                     | 1120                  | 1400 | 1640 | 1860 | 1910 | 2080 |      |      |      |
| III            | 7-1/2              | "          | 60                     | 1520                  |      | 2060 |      | 2300 |      | 2970 |      | 3145 |
| III            | 7-1/2              | 1% CaCl    | 72                     | 1590                  |      | 1965 |      | 2345 |      | 2600 |      | 2855 |

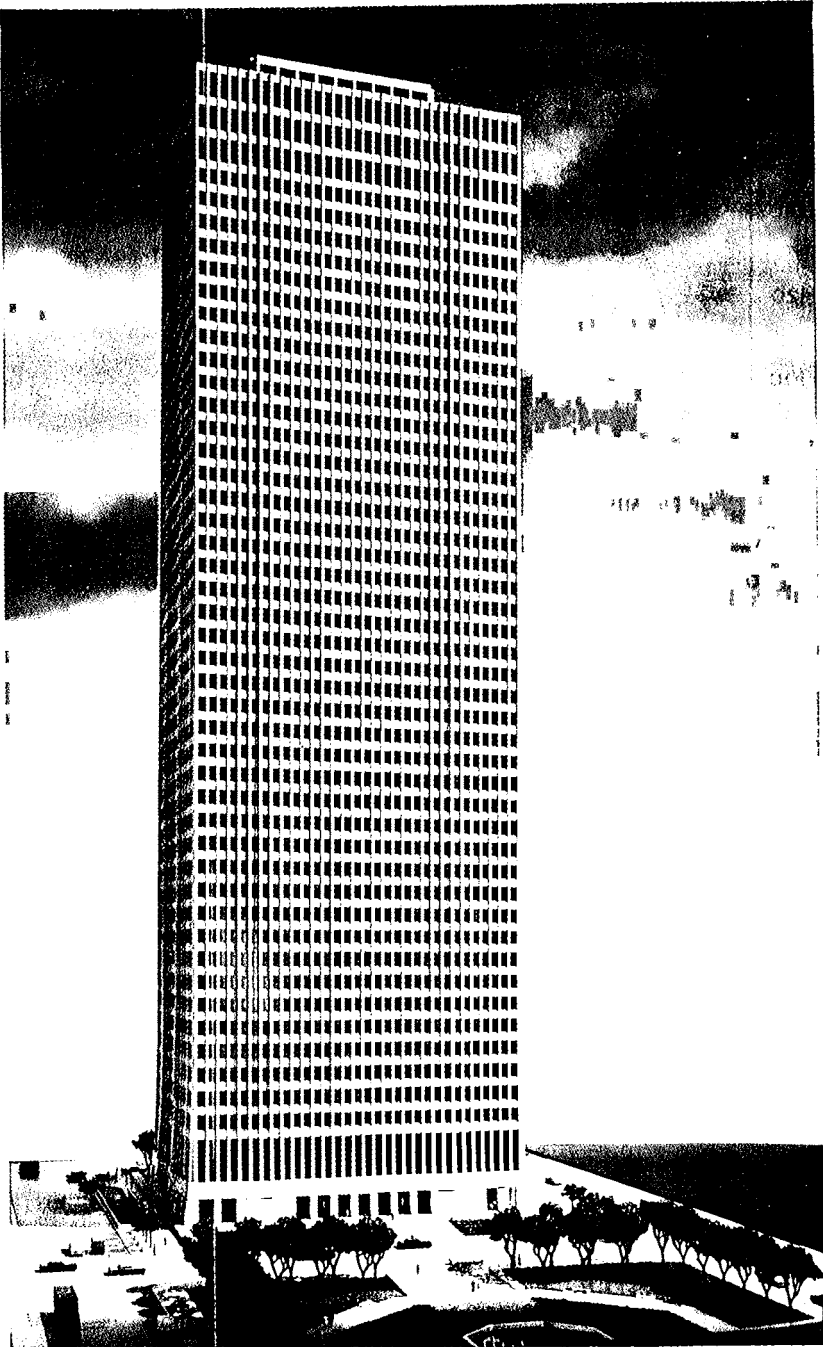


Fig. 2-1—One Shell Plaza, Houston

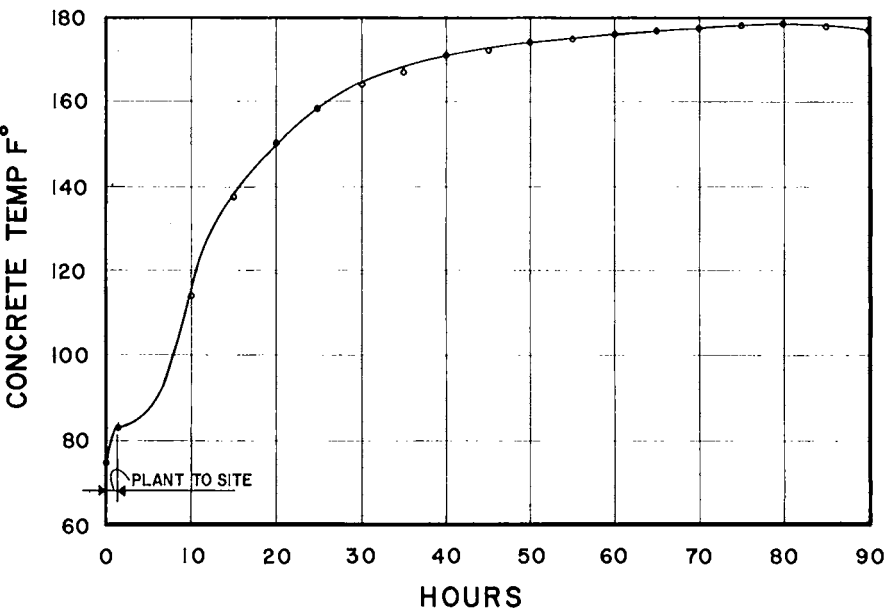


Fig. 2-2—Temperature rise for 8 ft cube mass concrete test specimen

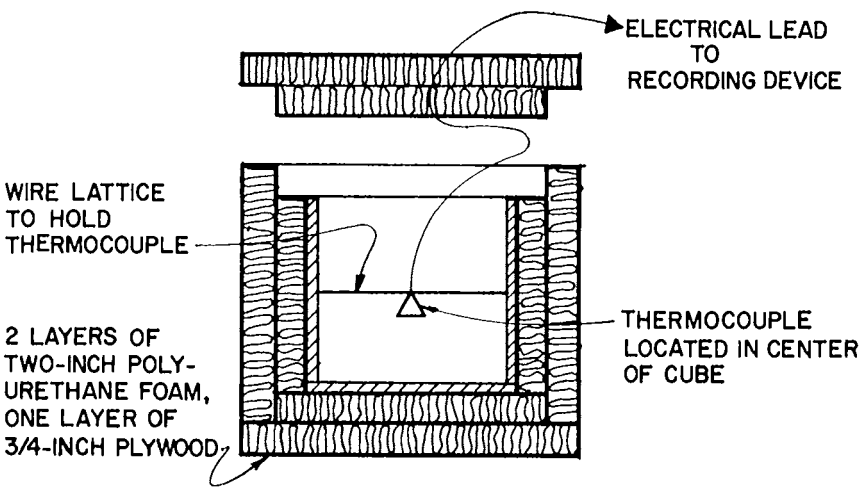


Fig. 2-3—1 x 1 x 1 ft test cube for heat of hydration study

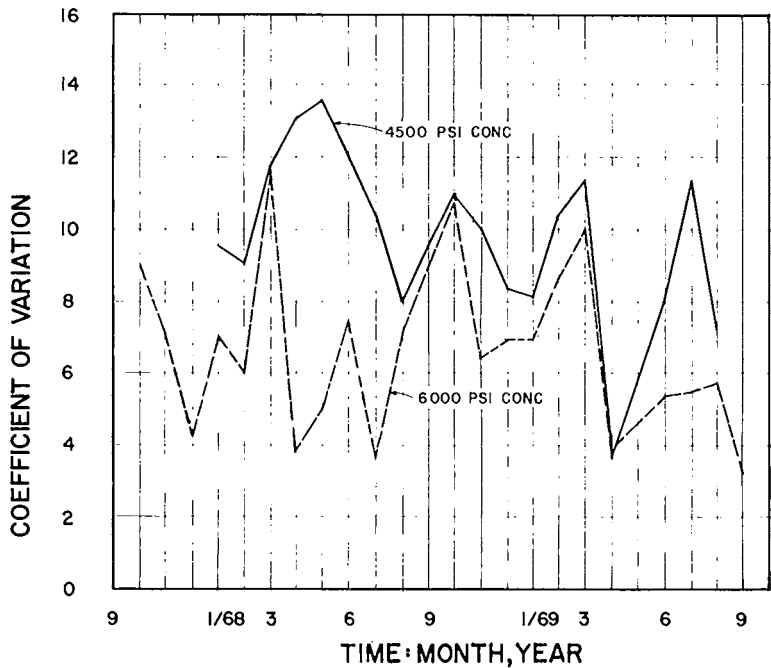


Fig. 2-4—Changes in coefficient of variation of the concrete over the period of the job

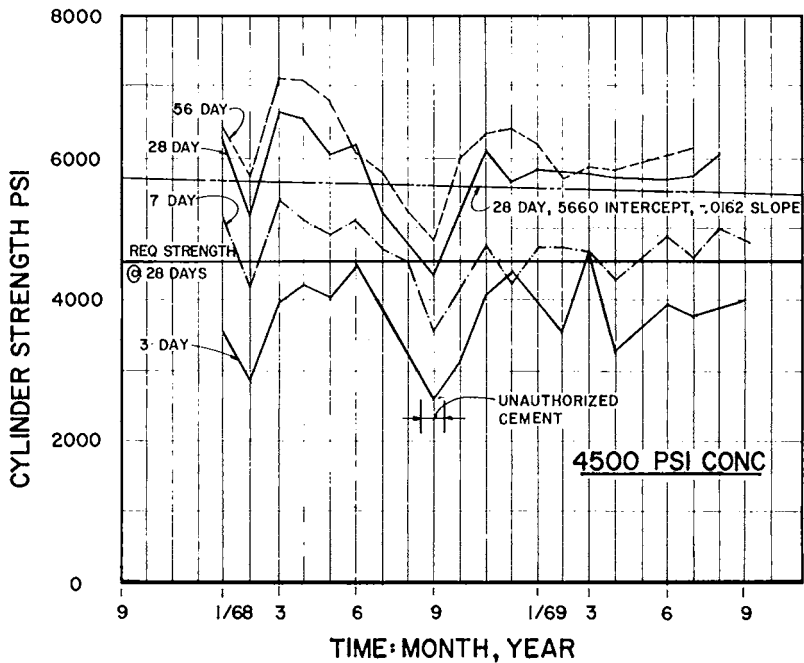


Fig. 2-5—Strength variation of the 4500 psi concrete over the period of the job

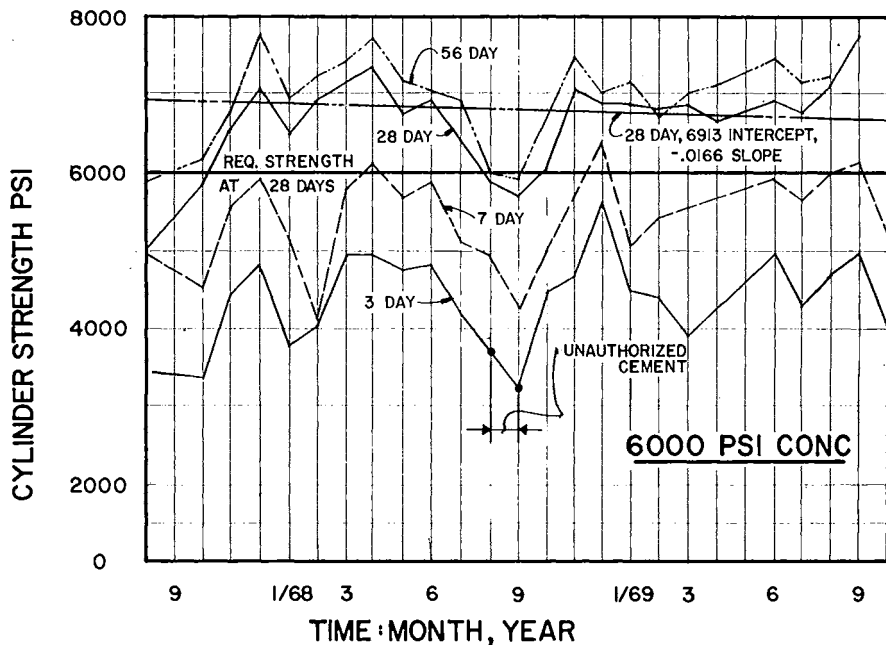


Fig. 2-6—Strength variation of the 6000 psi concrete over the period of the job

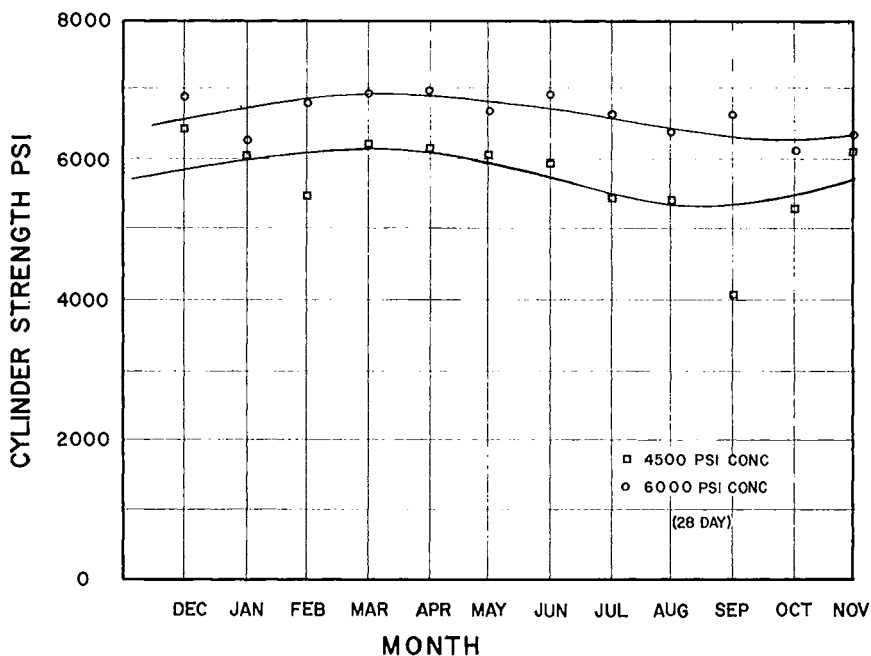


Fig. 2-7—Variation in average strength of the concrete by months

# INCREMENTAL LOADING OF REINFORCED LIGHTWEIGHT CONCRETE COLUMNS

By DONALD W. PFEIFER and EIVIND HOGNESTAD

Tests were made at the Portland Cement Association Laboratories regarding the elastic and time-dependent shortening of reinforced lightweight concrete columns which were fabricated and loaded to simulate construction conditions encountered in a 50-story concrete building 714 ft (218 m) tall. The measured data from these incrementally loaded columns show low creep when the load is applied at a realistic rate. A 3-year field investigation of the actual structure will be undertaken and comparison between laboratory and field data will be made. Such comparisons will provide data toward developing improved design concepts for ultra-high-rise concrete buildings.

Keywords: age; columns (supports); compressive strength; creep properties; drying shrinkage; expanded shale aggregates; high-rise buildings; lightweight aggregate concretes; loading rate; loads (forces); modulus of elasticity; plain concrete; reinforced concrete; research; volume change.

## INTRODUCTION

Recent experimental investigations<sup>1,2</sup> have shown that high quality structural lightweight aggregate concrete can safely be used in reinforced concrete columns, including ultra-highrise buildings. Their creep and shrinkage characteristics differ little from those of columns containing normal weight concrete. In both cases, elastic and time-dependent column shortenings were found to be governed primarily by reinforcing steel percentage, the influence of concrete type was relatively minor.

The previous study involved the typical laboratory procedure of moist curing the specimen for 28 days and then applying the full design load at that age. This loading technique bears little semblance to the actual incremental loading of concrete columns as construction of a tall structure proceeds. At the suggestion of Dr. Fazlur Khan of Skidmore, Owings and Merrill, laboratory tests dealing with incremental loadings were therefore undertaken. The main objective was

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Over the past 20 years, he has been actively involved in the engineering development of structural concrete, notably in the areas of ultimate strength design, provisions for shear, and utilization of high strength deformed bar reinforcement.

comparison with the shortening characteristics of columns loaded in the typical laboratory manner.

#### DESCRIPTION OF LABORATORY INVESTIGATION

##### General

These tests were made to determine the elastic and time-dependent shortenings of reinforced lightweight concrete columns which were fabricated and loaded to simulate conditions encountered in a 50-story concrete building 714 ft (218 m) tall. The investigation primarily concerns columns loaded in weekly increments to simulate the actual construction schedule. These columns required 50 equal weekly increments, each 2 percent of full load. Companion reinforced columns were also instantaneously loaded to full load when the columns were 1, 4, 35, and 50 weeks old.

Tests were also undertaken on non-reinforced concretes to determine compressive strength, elastic deformation, creep, and drying shrinkage characteristics as functions of time.

##### Materials

The coarse lightweight aggregate, No. 14 in the PCA numbering

series, is an expanded shale produced in a rotary kiln that has been studied extensively in previous investigations.<sup>1,2</sup> Normal weight Elgin, Illinois sand was used as all the fine aggregate. The concrete was proportioned to produce a nominal compressive strength of 6000 psi (422 kg/cm<sup>2</sup>) at 28 days. Lightweight concrete of this strength and containing 100 percent normal weight sand fines has a nominal modulus of elasticity of  $3.0 \times 10^6$  psi (211,000 kg/cm<sup>2</sup>) at 28 days. The slump and air contents were maintained at approximately 3 in. (7.5 cm) and 4 percent, respectively. The laboratory mix data are presented in Table 3-1; accompanying measured physical properties of this concrete are presented in Table 3-2.

The reinforced columns contained 5/8-in. (16-mm) diameter high strength deformed bars, which conform to the ASTM A431 specification and have a nominal yield point of 75,000 psi (53 kg/mm<sup>2</sup>).

#### Fabrication, Curing, and Instrumentation of Specimens

The study involved fabrication and long-time testing of eight 6-in. (15-cm) square by 36-in. (91-cm) long reinforced columns and 32 non-reinforced 6 by 12-in. (15 by 30-cm) cylinders.

The reinforcing was fabricated into tied column assemblies each containing four deformed bars. Lateral tie reinforcement consisted of 1/4-in. (6-mm) bars spaced at 6 in. (15 cm). The symmetrical longitudinal reinforcement was positioned to provide 1-in. (2.5-cm) concrete cover over the lateral tie reinforcement. The longitudinal bars were welded to 1-in. (2.5-cm) thick steel bearing plates.<sup>1,2</sup>

The columns were cast in a horizontal position and consolidation of concrete was by table vibration. The columns were moist cured for three days; they were then sealed in 0.003-in. (0.08-mm) thick copper foil to simulate idealized moisture conditions in the large prototype columns of the structure.

The elastic and time-dependent deformations were observed by a mechanical strain gage. At midheight of the columns, brass plugs were glued directly to the concrete on 10-in. (25-cm) centers on three of the four sides of the column. The deformation of the reinforcing steel and concrete have previously been measured to be equal,<sup>1,2</sup> and measurements directly on steel were therefore not made during this study. More detailed description of fabrication and instrumentation procedures reported previously<sup>1,2</sup> are similar to those used in this study.

The 6 x 12-in. (15 x 30-cm) cylinders were cast in the vertical position and consolidation of concrete was by internal vibration. All cylinders including those wrapped in copper foil were continuously moist cured until time of test.

## TEST PROGRAM

All eight reinforced columns were eventually loaded to 70,000 lb (31,780 kg). Two were loaded in 50 increments of 1400 lb (636 kg) over a 50-week period, starting when concrete was one week old. The remaining six were loaded instantaneously to 70,000 lb (31,780 kg), two at a concrete age of 1 week, two at 4 weeks, one at 35 weeks, and one at 50 weeks.

The 6 x 12-in. (15 x 30 cm) plain concrete cylinders were subjected to strength and modulus of elasticity testing at ages 7, 28, 90, and 365 days. Creep and drying shrinkage tests on unsealed cylinders began at ages 1 and 4 weeks and those specimens were then stored at 73 F (23 C) and 50 percent relative humidity. Creep tests on copper-foil wrapped cylinders began at ages 1, 4, 35, and 50 weeks. All creep specimens were loaded to 1500 psi (105 kg/cm<sup>2</sup>). The later-age sealed creep specimens (plain and reinforced) were initially observed to verify the lack of drying shrinkage in the copper-wrapped specimens.

## TEST RESULTS

## Strength and Elastic Properties

The properties of the plain concrete presented in Table 3-2 indicate that substantial increases in strength and stiffness of the concrete occurred as a function of curing time.

## Creep Properties of Plain Concrete

The measured creep of these sealed and unsealed concretes are shown in Fig. 3-1.

Fig. 3-1 (a) shows the creep of concretes loaded at an age of 1 week. After 150 weeks of loading, the measured creep of the sealed and unsealed cylinders was 425 and 900 millionths, respectively. The drying shrinkage of the companion unsealed concrete was 600 millionths at that same time. It is noted that the presence of drying shrinkage has the significant effect of approximately doubling the measured creep.

Fig. 3-1 (b) shows the creep of concretes loaded at 4 weeks. After 146 weeks of loading, the measured creep of the sealed and unsealed cylinders was 250 and 725 millionths, respectively. The drying shrinkage of the companion unsealed concrete was 550 millionths at that same time. With this loading age the creep is almost tripled when drying shrinkage is allowed.

Fig. 3-1 (c) shows the measured creep of the sealed cylinders which were loaded at ages 1, 4, 35, and 50 weeks. These data show the well-known influence of age of concrete at loading, and illustrate low creep characteristics of these sealed lightweight concretes.



The measured creep coefficients (creep strain per unit load) at age 50 weeks range from 0.08 to 0.25 millionths/psi (1 to 4 millionths/kg/cm<sup>2</sup>) for the sealed concretes. The data shown in Fig. 3-2 relate this measured creep coefficient occurring from the time of loading to the 50 and 150-week age versus the concrete age at loading. These data will be used later in the application of the theoretical prediction equation to the incrementally loaded sealed reinforced columns.

#### Elastic and Creep Properties of Instantaneously Loaded Columns

The measured data from the instantaneously loaded columns are shown in Fig. 3-3. The elastic response to load is in accord with elastic theory, and the significant increase in modulus of elasticity of concrete as a function of curing time is quite evident in the measured elastic response of these reinforced columns. The measured time-dependent creep characteristics also reflect the influence of age of concrete at loading on the time-dependent behavior of reinforced columns. At age 50 weeks the ratio of creep strain to elastic strain ranges from 0.57 with the 1-week loading to 0.25 with the 35-week loading. At age 150 weeks this ratio ranges from 0.61 with the 1-week loading to 0.41 with the 50-week loading.

#### Elastic and Creep Properties of Incrementally Loaded Columns

The measured data from the incrementally loaded columns are shown in Fig. 6. It is quite evident that the non-linear creep behavior was small as observed by the measured linear response during the incremental loading period. Creep is being further observed after the 50th and last load was applied. Since this last load application, the columns have shortened 65 millionths during the 100-week period following the last loading.

The computed elastic shortening, taking into account the increased modulus of elasticity of the concrete, is also shown in Fig. 3-4. It is seen that the influence of creep is small when sealed reinforced columns are incrementally loaded during a long-time period. At an age of 50 weeks the ratio of measured creep strain to computed cumulative elastic strain was 0.25. At an age of 150 weeks this ratio was 0.39.

## ANALYSES OF DATA

### Elastic Analysis

The elastic response of these reinforced lightweight concrete columns was found to be in accord with elastic theory. The measured elastic shortenings for the 5 conditions of loading were as follows:

|                       |          |                  |                    |   |   |
|-----------------------|----------|------------------|--------------------|---|---|
| Instantaneous Load at | 1 week   | = 540 millionths | elastic shortening |   |   |
| “                     | “        | 4 weeks          | = 434              | “ | “ |
| “                     | “        | 35 weeks         | = 400              | “ | “ |
| “                     | “        | 50 weeks         | = 400              | “ | “ |
| Incrementally         |          |                  |                    |   |   |
| Loaded during         | 50 weeks | = 433            | “                  | “ | “ |

#### Creep Analysis

Theoretical analyses as discussed by Leonhardt<sup>3,4</sup> have been shown<sup>1,2</sup> to adequately predict the time-dependent strain in reinforced columns caused by creep and drying shrinkage. The following equation<sup>1,2</sup> was used to predict the time-dependent steel stresses in the reinforced columns of this study caused by creep:

$$\Delta f_s = \frac{f_o}{p_g} [1 - e^{-\alpha C E_c}] \quad (1)$$

Eq. (1) can be converted to time-dependent reinforced column strain by applying the measured creep values  $C$  obtained from the unreinforced copper-wrapped cylinders and then by calculating the resulting change of steel strain which also equals change of column strain.

Application of Eq. (1) to the sealed reinforced columns which were instantaneously loaded results in single-step solutions which underestimate the measured time-dependent shortenings at 50 weeks of age by 13 to 27 percent. This underestimate may result because the use of the singular creep coefficient determined at a particular loading age does not take into account the change in creep characteristics as a function of time.

However, when Eq. (1) is applied to the incrementally-loaded columns in a 50-step solution, using the data in Fig. 3-2 to account for changing creep coefficients and the assumptions of superposition, much better results are obtained. The cumulative creep of the incrementally loaded columns at age 50 weeks was predicted to be 113 millionths and the measured creep was 106 millionths. The multistep solution, which takes account of changing concrete properties results in a ratio of  $\Delta f_s(\text{meas.})/\Delta f_s(\text{calc.})$  of 0.94.

#### CONCLUDING REMARKS

Data obtained from the incrementally loaded reinforced columns show that little creep takes place when the load is applied at a realistic rate and when the drying shrinkage influence on creep is eliminated. The creep that was measured during this 50-week loading period was only 25 percent of the elastic response, and the time-dependent creep phenomena essentially stopped after the 50th and final load was applied.

Theoretical time-dependent strains<sup>3</sup> compared well with the test data, so that theoretical analyses may be used to estimate such time-dependent movements.

### ACKNOWLEDGMENTS

The investigation described in this report was suggested in December, 1966 by Dr. Fazlur Khan of Skidmore, Owings, and Merrill. The experimental study concerned One Shell Plaza, Houston, Texas and was carried out in the former Concrete Products Research Section of the Portland Cement Association. Credit is due to D. F. Meinheit, E. Greinke and E. Valko.

### NOTATION

- $A_S$  = cross-sectional area of longitudinal reinforcement  
 $A_g$  = gross cross-sectional area of concrete column  
 $C$  = unit creep coefficient of plain concrete  
 $e$  = base of natural logarithms  
 $E_C$  = modulus of elasticity of concrete  
 $E_S$  = modulus of elasticity of reinforcement  
 $f_O$  = initial elastic stress in concrete  
 $n$  = modular ratio  $E_S/E_C$   
 $p_g$  = percentage of reinforcement  $A_S/A_g$   
 $\alpha = \frac{p_g n}{1 + p_g n}$   
 $\Delta f_S$  = change in steel stress due to creep

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TABLE 3-1—LABORATORY CONCRETE MIX PROPORTIONS

| Fine aggregate,<br>percent<br>by vol. | Quantities per cu yd (m <sup>3</sup> ) of concrete |                       |                        |                     | Percent<br>air,<br>roll-a-<br>meter | Plastic<br>unit<br>weight,<br>lb/ft <sup>3</sup><br>(kg/m <sup>3</sup> ) | Slump,<br>in.<br><br>(cm) |
|---------------------------------------|----------------------------------------------------|-----------------------|------------------------|---------------------|-------------------------------------|--------------------------------------------------------------------------|---------------------------|
|                                       | Water,<br>lb<br>(kg)                               | Cement,<br>lb<br>(kg) | Air-Dry aggregates     |                     |                                     |                                                                          |                           |
|                                       |                                                    |                       | Coarse,*<br>lb<br>(kg) | Sand†<br>lb<br>(kg) |                                     |                                                                          |                           |
| 44                                    | 315<br>(187)                                       | 623<br>(370)          | 900<br>(534)           | 1245<br>(739)       | 3.8                                 | 114.2<br>(1830)                                                          | 2.8<br>(7)                |

\*3/4 in. maximum size lightweight aggregate

†Normal weight Elgin, Illinois sand

TABLE 3-2—PHYSICAL PROPERTIES OF CONCRETE\*

| Compressive Strength,†<br>psi<br>(kg/cm <sup>2</sup> ) |               |               |               | Modulus of Elasticity,†<br>10 <sup>6</sup> psi<br>(10 <sup>5</sup> kg/cm <sup>2</sup> ) |                |                |                |
|--------------------------------------------------------|---------------|---------------|---------------|-----------------------------------------------------------------------------------------|----------------|----------------|----------------|
| 7 days                                                 | 28 days       | 90 days       | 1 yr.         | 7 days                                                                                  | 28 days        | 90 days        | 1 yr.          |
| 5220<br>(367)                                          | 6360<br>(447) | 6990<br>(491) | 7230<br>(508) | 3.07<br>(2.16)                                                                          | 3.34<br>(2.35) | 3.60<br>(2.53) | 3.82<br>(2.69) |

\*All cylinders were continuously moist cured

†An average of four specimens.

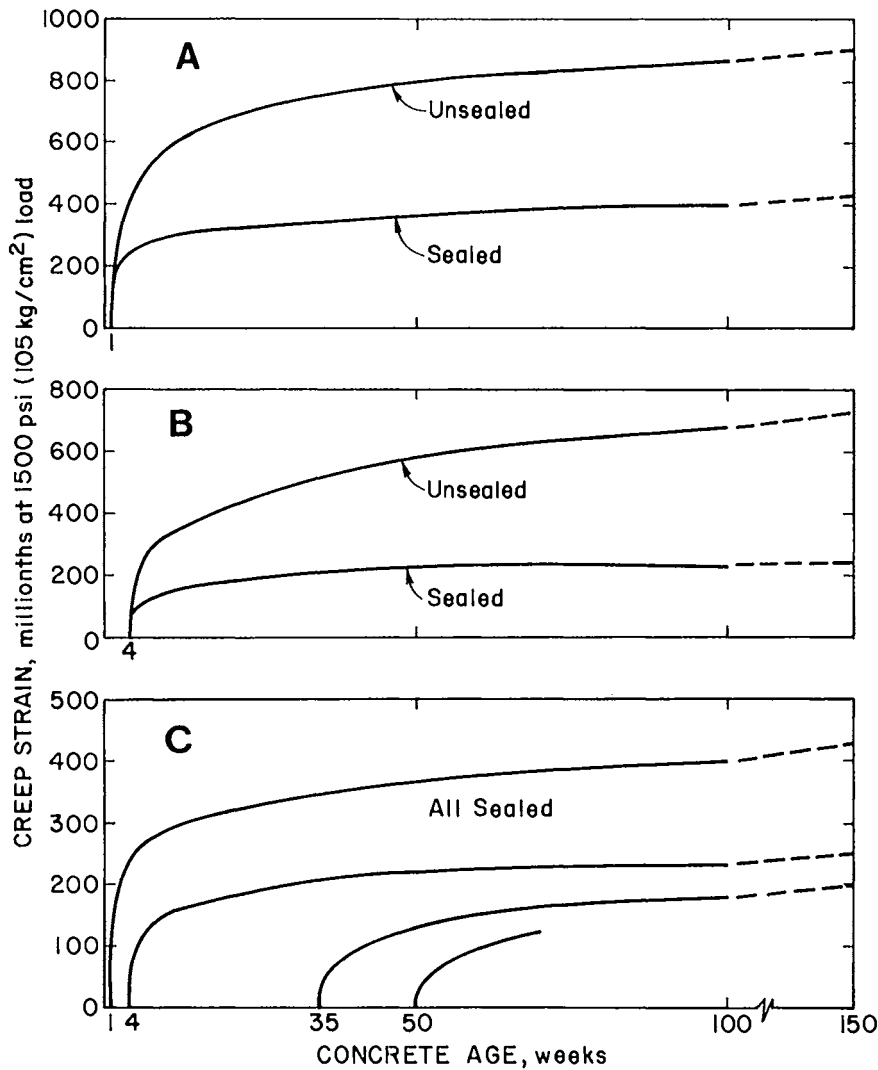


Fig. 3-1-Creep of plain concrete—effect of age of concrete at loading and drying shrinkage

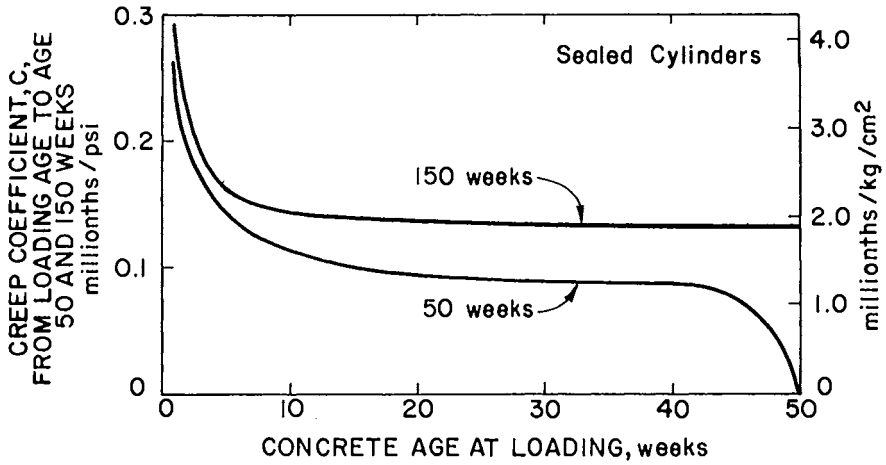


Fig. 3-2-Measured creep coefficient of plain concrete versus concrete age at loading

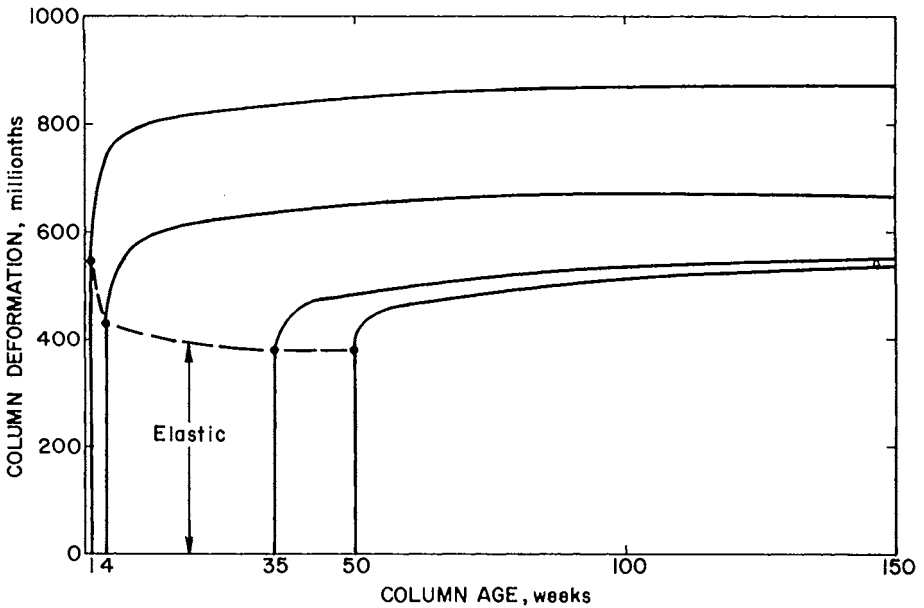


Fig. 3-3-Elastic and time-dependent creep deformation of instantaneously loaded reinforced columns

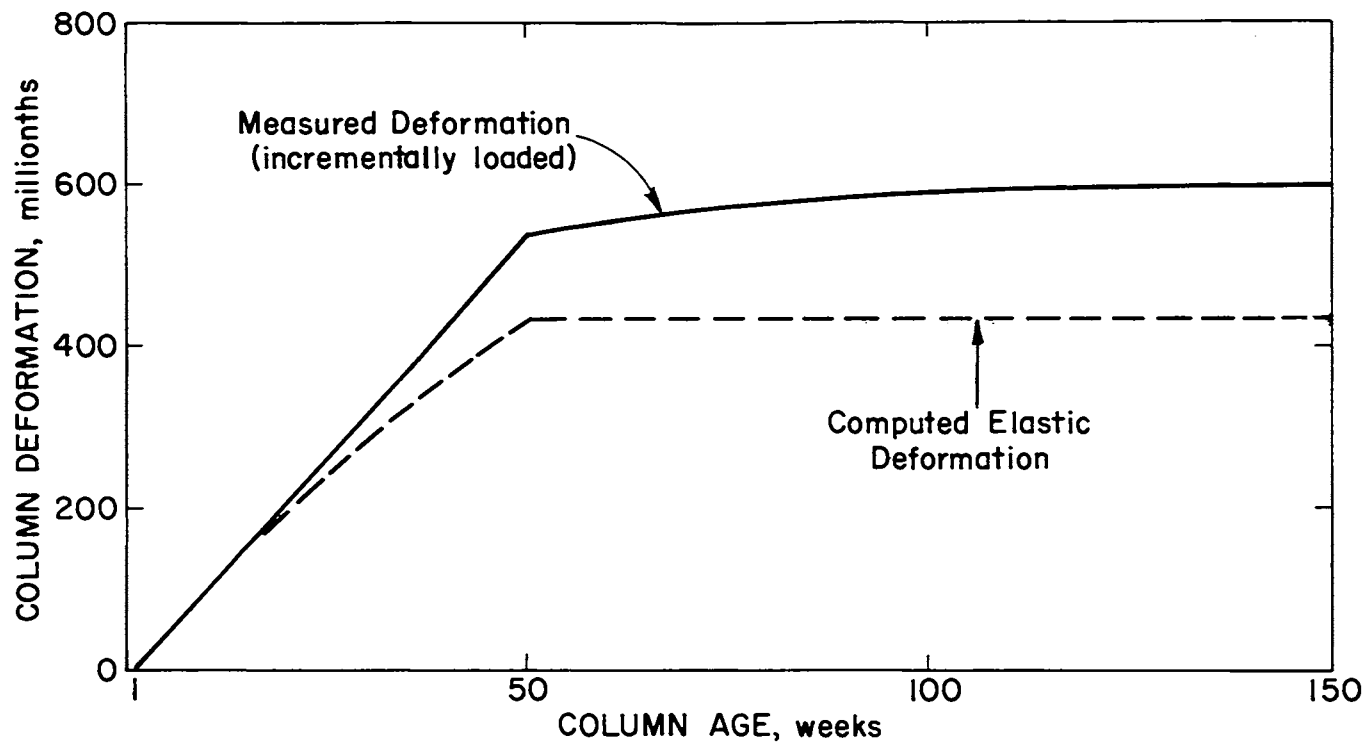


Fig. 3-4-Elastic and time-dependent creep deformation of incrementally loaded reinforced columns

# LEBANON SENIOR HIGH SCHOOL - UNIQUE USE OF LIGHTWEIGHT CONCRETE FOR LONG CIRCULAR SPANS

By VINCENT J. DeSIMONE

This article presents highlights of the structural analysis and design, as well as the quality and control of materials, in what is considered to be the largest circular concrete plate in the United States. A review of long term deflection control is presented with a view towards resolution of laboratory test data in lightweight concrete with design and field practice.

Keywords: camber; coefficient of variation; compressive strength; concrete construction; deflection; domes (structural forms); flat concrete plates; lightweight aggregate concretes; plates (structural members); quality control; reinforced concrete; roofs; school buildings; shells (structural forms); structural analysis; structural design.

## PROJECT

Lebanon Senior High School, Lebanon, Pa., consists of three circular buildings (see Fig. 4-1), two of which represent unusual design and quality control for lightweight concrete. The two buildings under discussion are the library and the gymnasium.

The library is a two story building with a domed roof spanning a central two story high clear space (see Fig. 4-2 and 4-3). The dome has a 66 ft diameter and a rise of 7 ft 11 in. An opening of 24 ft was provided at the center of the dome forming a skylight for this central space. The spring line of the dome is supported by a 20 x 24 in. circular beam for vertical forces, and by the flat horizontal roof system around the dome for the horizontal forces.

The flat roof structure includes an 8-1/2 in. flat plate spanning 27 ft and 17 ft, and a 13 in. one-way slab with hollow voids spanning 33 ft.

The gymnasium is also a two story structure, whose central clear story is spanned with a flat circular plate (see Fig. 4-4 and 4-5). The diameter of the plate is 130 ft, and the thickness of the slab is 20 in. also cast with hollow voids. Adjacent to the plate is a 15 in. thick slab which spans 45 ft to the outside perimeter of the building.

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The plate is supported at it's perimeter by a 40 x 30 in. circular beam. This beam in turn is supported by 24 x 40 in. columns spaced at 18 ft. The size of the beam and support columns was determined as a result of analysis to develop edge restraints in the plate.

#### ANALYSIS

The library and gymnasium roofs were analyzed according to the theory of plates and shells.\* The dome was analyzed as a spherical surface of revolution which has primarily membrane stresses.

The vertical component of the force  $N_\phi = 1.73$  k and is supported by a vertical beam; the horizontal component of  $N_\phi = 3.34$  k is developed by the large circular beam formed by the perimeter slab (see Table 4-1). Four in. of reinforced concrete with 2 layers of mesh as reinforcement was sufficient to handle these thrusts and any local bending as a result of unsymmetrical loading and edge bending. Due to the relatively shallow angle of the shell, and the high stiffness of the supports, edge bending of the supports was virtually eliminated.

The 130 ft diameter plate forming the center of the gymnasium roof was analyzed as a circular plate with symmetrical loading. The analysis for two conditions of edge support are determined as in Table 4-1 where  $\nu$  represents Poisson's ratio. For concrete, the factor was taken from the German code as

$$\nu = \sqrt{\frac{f'_c}{350}}$$

Thus for  $f'_c = 5000$  psi,  $\nu = 0.2$

The function D is the flexural rigidity of the plate and thus

$$D = \frac{E h^3}{12(1-\nu^2)}$$

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\*Timoshenko, S., Theory of Plates and Shells, McGraw Hill, New York, 2nd Ed., 1959.

In determining the rigidity of the plate for deflection, it was decided that the more conservative value based on a cracked section should be used for the  $h^3$  term. An analysis of this equation shows that the normally accepted values for Poisson's ratio, that is 0.15 to 0.30, will affect the value of  $D$  minimally. Knowing  $M_t$ ,  $M_r$ , and the deflection of the two boundary conditions, that is, simply supported and clamped edges, it was then necessary to determine the actual degree of fixity of the plate to be designed. To achieve this a moment distribution was performed whereby the plate was analyzed as the center span in a three-span frame.

In determining the stiffness of the plate, the moment of inertia was based on the uncracked section. Since all the equations for deflection substitute radius for the length of span, the radius of the plate was used in the frame analysis as contrasted against the diameter (Fig. 4-6).

A comparison of the fixed end moment of -110.0 kip-ft with the final distributed moment of -105.9 kip-ft shows that the edge restraint condition of the plate very closely approximates that of a plate with clamped edges. Therefore the final moments are as follows:

|       | Plate Edge    | Middle of Plate |
|-------|---------------|-----------------|
| $M_r$ | -106.0 kip-ft | -70.4 kip-ft    |
| $M_t$ | -21.2 kip-ft  | -70.4 kip-ft    |

The above values assumed a dead load of 154 lb per sq ft and a live load of 46 lb per sq ft.

While it has been the practice of some designers to design circular plates using a more conservative approach of assuming a strip in bending with the span equal to the diameter, the more precise method as outlined results in moments and deflections of a circular plate  $3/8$  of the values using the diameter as the span.

Deflection calculations (see Table 4-3) indicate an instantaneous total deflection of 5.75 in. and a total long range deflection, considering creep, of 11.72 in.

## DESIGN

The design of the plate was based upon  $f'_c = 5000$  psi, and  $f_y = 60,000$  psi. The concrete was specified as lightweight. Hollow voids were cast in the plate to reduce dead load by approximately 38 lb per sq ft (Fig. 4-7). All reinforcement was designed based upon ultimate strength requirements of ACI 318-63. Reinforcement for the plate was placed in both the longitudinal and transverse directions. Since  $M_r$  and  $M_t$  are equal in the center of the plate, a

two-way grid of reinforcement was placed at the bottom of the plate at its center (Fig. 4-4).

Table 4-4 indicates desired performance criteria which resulted in choices of material and control requirements.

#### CONCRETE QUALITY

The following mix proportions were determined by preliminary trial mixes. To insure concrete consistency as much as possible, specifications required that cement shall be of one brand, and that the lightweight aggregate be from a single source. The concrete mixes shown below are the result of mix designs based upon performance criteria rather than minimum cement factors.

|             |                   |               |
|-------------|-------------------|---------------|
| 4000 psi    |                   |               |
| Cement      | 5.91 sacks        | 556 lb        |
| Fine agg.   | Sand              | 1268 lb       |
| Coarse agg. | 3/4 in. to #4     | 928 lb        |
| Admixtures  | Water reducing    | 2 ounces/sack |
| Slump       | 4 $\pm$ 1 in.     |               |
| 5000 psi    |                   |               |
| Cement      | 6.86 sacks        | 645 lb        |
| Fine agg.   | Sand              | 1192 lb       |
| Coarse agg. | 3/4 in. to #4     | 915 lb        |
| Admixtures  | Water reducing    | 2 ounces/sack |
| Slump       | 3 1/2 $\pm$ 1 in. |               |

Requirements for the testing of concrete were specified as follows:

(1) The laboratory is to take one test (two cylinders) for each 50 cu yd placed, with a minimum of one test for any fraction of 50 cu yd, or any day's placement. The laboratory was to break one cylinder at the contractor's request in 7 days to assist in determining stripping times.

(2) The lightweight aggregate supplier was to maintain an up-to-date record of the coefficient of variation for mix control. As a result of strength tests and coefficients of variation, the testing laboratory was to work with the architect and the engineer as the sole agents in changing materials or concrete proportions. Fortunately,

this option was never utilized since concrete tests in compression exceeded the design strength.

### CONCRETE PERFORMANCE

A total of 33 tests were taken for the 4000 psi concrete, representing 67 cylinders, and 16 tests representing 35 cylinders for the 5000 psi concrete specified for the shell and plate.

Of the cylinders taken for the 4000 psi concrete, none broke below design strength. Concrete strengths, however, varied between 4800 psi and 5800 psi with an average strength for the project of 5586 psi. Of the cylinders taken for the 5000 psi concrete, 2 cylinders were below design strength, with an average strength for the project of 5943 psi.

The coefficients of variation, determined in accordance with ACI 214-57, were 9.7 percent for the 4000 psi lightweight concrete and 13.7 percent for the 5000 psi lightweight concrete. A coefficient of variation of 9.7 percent is considered excellent and one of 13.7 percent is rated as very good for field tested concrete. The distribution curves for these two designs are shown in Fig. 4-9 and 4-10.

### CONSTRUCTION

As a result of the exciting shapes that were to be cast, the contractor closely coordinated his placement sequence and schedule with the engineer. The following methods were arrived at as a result of this coordination:

#### Dome

The entire dome represented less than 100 cubic yd of concrete. For this reason, the contractor chose to place the concrete continually from the spring line of the dome to the central compression beam. The concrete was placed on concentric rings starting at the spring line and working up to the skylight opening at the center of the dome. Due to the relatively shallow angle of the arch, concrete placement was not difficult since 3 1/2 to 4 1/2 in. slump concrete held the slope easily.

#### Plate

It was decided that the plate should be placed also in concentric rings working from the perimeter support into the interior. Since the plate was to be placed in three stages, the contractor erected all his formwork for the entire plate with all reinforcement in place prior to first casting. Instructions to the contractor required that the formwork be left in place under the entire plate for a minimum 28 days since deflection calculations were based on the modulus of