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Structures and Materials Research
Division of Civil Engineering

SHEARING STRENGTH OF REINFORCED AND PRESTRESSED CONCRETE LIFT SLABS

A Report of an Investigation

by

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to the

DIVISION OF ARCHITECTURE
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I. INTRODUCTION

1. Object

In recent years the lift slab method of construction has become a popular erection technique for buildings of the flat slab type. These slabs may be made of reinforced concrete or of prestressed concrete. The latter type has the advantage that deflection, which may be excessive in reinforced concrete slabs, can be minimized or even nullified by prestressing. The load on a lift slab is transferred to a column by means of a steel collar. This collar becomes an integral part of the slab at the time of casting and is fastened to the column, by welding or by some other means, after the slab has been lifted to its final position.

Present design of flat slabs with respect to shear is based on only a limited amount of factual knowledge. For reinforced concrete lift slabs most designers follow current specifications similar to those given in ACI 318-56 for shear design of flat slabs. For prestressed slabs, present practice varies considerably but generally consists of keeping the punching shearing stress at the edge of the collar below some arbitrary assigned value. This allowable value may range from 0.04 f'_c to 0.06 f'_c.

Little experimental work has been done to determine the shearing strength of such slabs. It was the purpose of this project to investigate the ultimate shearing strengths of prestressed concrete slabs (also some reinforced concrete slabs) with lift slab collars and to develop, by means of a properly designed experimental program, expressions for their ultimate shearing strengths.

2. Scope

While present design practice bases the allowable shearing stress on only one variable, the cylinder strength, f'_c, it is generally agreed that the ultimate shearing strength is dependent on several other variables as well. The most important of these variables appears to be the distribution of shearing and normal stresses on the critical section. Since in slabs the normal stresses are produced by flexural action, the shearing strength is therefore a function of the shear - moment ratio. In prestressed slabs the amount of prestressing will also have an effect on this variable. Another important factor to be considered is the amount of cracking that has taken place just prior to failure. This determines the net section available to resist shear at failure.

It was felt that the foregoing effects as well as other practical design problems could be best studied by including the following variables in the experimental program:

a. Concrete strength.

b. Amount of prestressing or reinforcing steel.

c. Amount of initial prestress.

d. Size of steel collars.

e. Thickness of slab.

f. Amount of collar recess.

A total of fifteen slabs were tested to ultimate failure. All specimens were 6 ft. square and had thicknesses of 6, 8, or 10 in. Slabs were supported along all four edges and centrally loaded.

Fourteen slabs with lift slab collars were tested. Of this group twelve were prestressed using unbonded cables and two were made of reinforced concrete. In addition one reinforced concrete slab with a concrete column stub and shear reinforcement was tested.

3. Acknowledgements

The program reported herein was conducted in the Structural Engineering Laboratory, Division of Civil Engineering, University of California. The program was sponsored by the Division of Architecture, Department of Public Works, State of California through a research grant administered by the Institute of Engineering Research, University of California. The program was carried on between May 1, 1956 and June 30, 1957.

The Division of Architecture is under the direction of Anson Boyd, State Architect. Special appreciation for their help and suggestions is due to C. M. Herd, Chief Construction Engineer, Charles Peterson, M. W. Sahlberg, Principal Structural Engineers, and A. H. Brownfield, Supervising Structural Engineer, all of the Division of Architecture.

Much credit for the successful conduct of the program should go to S. A. Ravid and S. Y. Chang, Graduate Research Assistants, who participated extensively in the theoretical studies, the conduct of the tests, and the reduction and interpretation of the data.

4. Notation

The letter symbols used in this report are generally defined when they are introduced. The most frequently used symbols are listed below for convenient reference:

a - width of slab

b - perimeter of vertical shear area at critical section

d - average effective depth of tensile reinforcement, distance from compression surface of a slab to the plane common to each layer of tensile reinforcement

E_c - secant modulus of elasticity at 1000 psi obtained from 6 x 12 in. concrete cylinders

E_s - modulus of elasticity of steel

f'_c - compressive strength of 6 x 12 in. concrete cylinders

f_e - effective unit stress in psi in prestressing steel prior to loading

F_e - effective total force in lbs in prestressing steel prior to loading

$f_{p.l.}$ - proportional limit of prestressing steel

f'_t - modulus of rupture of concrete

f_u - ultimate strength of prestressing or reinforcing steel

f_y - yield point of prestressing or reinforcing steel

j - ratio of internal moment arm to effective depth, assumed equal to 7/8

m - bending moment per unit width

P - load

P_{shear} - calculated ultimate shearing load

P_{flex} - calculated ultimate flexural load

P_{test} - measured ultimate load

p - average percentage of tensile reinforcement

q - tensile reinforcement index = $\frac{pf_y}{f'_c}$

r - side dimension of square loaded area

s - cable spacing in inches

t - total thickness of slab

V - shearing force

V_1 - shearing force corresponding to v_1

V_2 - shearing force corresponding to v_2

v - shearing stress

v_1 - shearing stress computed at a distance d from the loaded area

v_2 - shearing stress computed at the edge of the loaded area

ϕ_o - ratio of P_{shear}/P_{flex}

II. EXPERIMENTAL PROGRAM

1. Description of Test Slabs

The test slabs were divided into six groups. A description of each specimen may be found in Table 1 and in Figs. 1, 2, 3, and 5. All slabs were 6 ft. square. Outer dimensions of the square steel collars used were either 13 or 16 in. Nominal concrete strengths used were 3000 or 4500 psi. Average initial prestress for the slabs covered a range between 250 and 500 psi. Slab thicknesses were 6, 8, or 10 in. with collar recesses of 0, 1, 2, or 4 in.

a. Group I The two slabs of this group, S-1 and S-2, were identical reinforced concrete slabs except for concrete strength. They were 6 in. thick with 13 in. steel collars. Fig. 1 shows a sketch of these slabs indicating the reinforcement used. This reinforcement consisted of No. 8 bars placed at $8\frac{1}{4}$ in. c-c at a depth of $4\frac{3}{4}$ in. in one direction and at $6\frac{1}{2}$ in. c-c at a depth of $3\frac{3}{4}$ in. in the other direction. These values were selected so as to give equal moment resistance in either direction. Bars were cut off at the edge of the column opening.

b. Group II This group consisted of five prestressed concrete slabs, S-4 through S-8, which were all 6 in. thick with 13 in. collars. The variables consisted of concrete strength, amount of prestress, and arrangement of prestressing cables. The collars were all flush except that of S-4 which was recessed 1 in. Figs 2 and 3 indicate the prestressing cable arrangements.

For S-4, S-6 and S-8 the cables were spaced at 18 in. c-c in each direction with the distance from the bottom of the slab to the c.g.s. equal to 2 in. and $2\frac{3}{4}$ in. for the two respective directions. The cables in S-5 had the same horizontal spacing of 18 in. c-c in each direction but they were placed with depths of $2\frac{5}{8}$ in. and $3\frac{3}{8}$ in. for the two respective directions. A 12 in. c-c spacing was used for the cables of S-7. These cables had a parabolic sag from a mid-depth position at the slab edges to a depth at centerline of $4\frac{7}{8}$ in. in one direction and $4\frac{1}{8}$ in. in the other direction.

c. <u>Group III</u> This group, consisting of S-9 and S-10, was similar to group II with the exception that the collar size was 16 in. instead of 13 in. Arrangement of prestressing cables was similar to S-4, S-6, and S-8 and is shown in Fig. 2.

d. <u>Group IV</u> Slabs S-11, S-12, and S-13 were all 10 in. thick and had 13 in. collars with recesses of 0, 2, and 4 in. respectively. Arrangement of prestressing cables was similar to group III and is shown in Fig. 2.

e. <u>Group V</u> Slabs S-14 and S-15 were 8 in. thick and had 13 in. collars with recesses of 0 and 2 in. respectively. Arrangement of prestressing cables was similar to group III and is shown in Fig. 2.

f. <u>Group VI</u> This group consisted of a single special slab, S-16, which was a full scale model simulating a reinforced concrete slab and column from a building designed by the Division of Architecture, State of California. Details of this slab are shown in Fig. 4. The slab was 8 in. thick. Tensile reinforcement consisted of No. 6 bars at 5 in. c-c in one direction and No. 6 bars at 4 in. c-c in the other direction. Compressive reinforcement in the top of the slab consisted of No. 6 bars at 18 in. c-c one way and No. 6 bars at 22 in. c-c in the other direction. A "zig-zag" shear head constructed of No. 3 bars was placed concentrically around the column stub. The 18 in. square column was reinforced with four No. 8 bars and No. 3 ties.

2. Fabrication

Slabs were cast on a steel plate with wooden side forms which were bolted to the plate by means of clip angles. The wooden side forms had holes drilled in them to hold the prestressing cables in the desired locations. The steel collars were supported on a small wooden box which bolted to the bottom plate and formed the column opening. Prestressing cables were supported at the ends only. Reinforcing bars were wired securely and supported at three to four points along the bottom bars. The cables in S-7 were wired so that around the collar there was a 14 inch square opening rather than the 12 inch center to center spacing of the cables. Photographs of reinforced and prestressed concrete slab arrangements just prior to casting are shown in Fig. 5.

Eight to thirteen batches of concrete mixed in a 2.5 cubic foot tilting drum mixer were necessary to cast each slab. Concrete was vibrated internally with a 1 $\frac{3}{4}$ in. diameter internal vibrator after at least six batches had been placed in the form.

Forms were stripped 3 to 7 days after casting. The specimens were cured moist for 7 days using damp burlap and left air dry until testing at 13 or 14 days. Prestressing was done the day prior to testing.

3. Materials

Santa Cruz type I Portland Cement was used throughout the tests. The cement was purchased in one lot of 200 sacks and stored in steel drums until used. The fine aggregate used was Elliot S. E. 80 sand. The average fineness modulus was 3.20. The coarse aggregate was Elliot $\frac{1}{4}$ inch to $\frac{3}{4}$ inch gravel having an average fineness modulus of 6.59. Sieve analyses for these aggregates are given in Table 2. Absorption for both the fine and coarse aggregates was about 1.5 percent by weight. The specific gravities were 2.66 for sand and 2.69 for gravel.

Concrete mixtures were designed for 14 day strengths of 3000 and 4500 psi. Two different mixtures were used for the 4500 psi concrete. Water-cement ratios were 0.49 and 0.44 for the 4500 psi concrete mixtures and 0.67 for the 3000 psi concrete. Mix proportions are given in Table 3. Consistency as measured by the Kelly Ball penetration averaged about 2.2 inches for all mixtures. This penetration is equivalent to a slump of about $4\frac{1}{2}$ inches.

Control specimens consisted of four 6 x 12 in. cylinders and three 6 x 6 x 20 in. beams for each slab. Control specimens were cured in the same manner as the slabs. Cylinders and beams were tested on the same day as the corresponding test slab. A typical stress strain curve for a 6 x 12 in. cylinder is shown in Fig. 6. Average values of compressive strength and modulus of elasticity obtained from the cylinders and modulus of rupture obtained from the control beams are presented in Table 4.

The prestressing cables each contained six $\frac{1}{4}$ in. diameter cold drawn steel wires. The wires were coated with a special asphaltic compound and wrapped with sisal-kraft paper to prevent bonding to the concrete. Details of the cable assembly and the anchorage are shown in Fig. 7. The anchorage was modified from that conventionally used by substituting a long (3 in.) stressing washer for the usual $1\frac{1}{4}$ in. stressing washer. A nut was provided for anchorage rather than the typical shims. This permitted an excellent control of the cable force in the short cables used.

A number of samples of the prestressing wire were tested on a 10 in. gage length to determine the proportional limit, $f_{p.l.}$; yield point, f_y, as measured by the 0.2 percent offset method; ultimate strength, f_u; modulus of elasticity, E_s; and percent elongation in a 10 in. gage length. The results of these tests are shown in Table 5. A typical stress-strain curve is shown in Fig. 8.

Reinforcing bars conformed to ASTM A-305 specifications and were of intermediate grade. Bars were delivered in 20 ft. lengths and cut to required lengths in the laboratory. Four samples of No. 8 bars and four samples of No. 6 bars were tested to determine properties similar to those found for the prestressing steel. These results are given in Table 6 and a typical stress-strain curve is shown in Fig. 9.

4. Collars, Loading Column, and Support Frame

The lift slab collars used in the tests had outer dimensions of either 13 in. square or 16 in. square with a column opening of 8 in. square. Short lengths of $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{2}$ in. steel angles with $\frac{1}{2}$ in. stiffening plates were welded together to form the 13 in. collars. $6 \times 4 \times \frac{3}{4}$ in. steel angles with $\frac{1}{2}$ in. plates were used for the 16 in. collars. Details of both types of collars are shown in Fig. 10. The collars were salvaged after each test and reused several times.

To simulate the column used in lift slab construction, a $7\frac{1}{2}$ in. square column was fabricated from an 8 x 8 in. angle section. $\frac{3}{4}$ in. thick plates were welded to each side of the column to transfer the load from the column to the slab. Details of the column are shown in Fig. 11. To provide nearly uniform bearing on each side of the column, shims were placed between the column lug plates and the slab collar. A spherical bearing block was placed between the column and the head of the testing machine.

To approximate the conditions existing in continuous structures a simple support at the perimeter of the test slabs was selected. The position of this support approximates the line of zero moment around an interior column in a typical lift slab structure. To give this support a steel frame was constructed of 8 inch wide flange beams. The open frame was designed to provide a view of the bottom of the slab during testing. Details of the frame are shown in Fig. 12. Wheels permitted the frame, with mounted slab, to be rolled into position under the testing machine after which the wheels could be removed. A 1 x 2 in. strip of oak wood was bolted to the top of the frame to allow some flexibility for the support. Hydrostone, a capping compound, was placed between the slab and support to ensure uniform bearing at the start of each test. No effort was made to hold down the slabs which when loaded tended to deflect upward at the corners.

5. Loading and Instrumentation

Cable prestress was applied by means of a 30-ton capacity hydraulic jack which had been calibrated. Prestressing was done one day prior to testing in all cases. The cables were overstressed 10 percent for a two to three minute period, then released to a 5 percent over-stress which was assumed to be the anchorage takeup loss. Creep and shrinkage for a one day period was considered negligible.

Load was applied with a 4,000,000 pound Southwark-Emery universal testing machine. The least reading of the machine in the range used was 500 pounds. The capacity of the machine was much greater than needed, but it was necessary to use this machine because of the required clearance between screws necessary for the test assembly.

The slabs were loaded in 8 to 25 increments of load to failure. Strains and deflections were measured after each loading increment.

Strains in the concrete were measured by means of twelve SR-4, type A-1 strain gages for each slab. Gage locations are shown in Fig. 13. Locations of the SR-4, type A-5 gages on the reinforcing bars of slabs S-1 and S-2 are shown in Figure 14. Standard procedures were used in mounting all gages. Baldwin type L indicators were used in connection with switch units to read the indicated strains.

Deflections were measured along two lines with fourteen 0.001 in. dial gages attached to a bridge to give the slab profile at each loading increment. 0.001 in. dial gages also measured the deflections of the column and two slab corners. Fig. 13 shows the locations of the dial gages. The corners were free to lift during all tests.

The photographs of Fig. 15 show the test assembly and the instrumentation.

III. ANALYTICAL STUDIES

1. General Remarks

As mentioned earlier it was the purpose of this project to develop, by means of a test program, expressions for the ultimate shearing strengths of prestressed concrete lift slabs.

At present there is no design specification available for computing the allowable shear load on a prestressed concrete flat slab. For reinforced concrete flab slabs, however, the 1956 ACI Building Code [1] prescribes that the shearing unit stress existing in a flat slab be computed by the formula:

$$v_1 = \frac{V}{bjd} \quad \text{- - - - - - - - - - - - - - -} \quad (1)$$

in which V is the total shear acting on vertical sections which follow a periphery, b, at a distance, d, beyond the edges of the column or column capital and parallel or concentric with it. d is specified as the effective depth of the tensile reinforcement and j is normally taken as 7/8. The ACI Code further stipulates that the stress as computed by eq. (1) shall not exceed an allowable shearing stress which is given as a function of the cylinder stress, f'_c, and the amount of bending reinforcement passing over the column or column capital. In applying eq. (1) to lift slabs, designers normally take the outer edge of the steel collar as synonymous with the edge of the column or column capital in the ACI definition.

Recent studies made by Elstner and Hognestad(2) (3) indicated that the above procedure yielded a highly variable factor of safety between allowable and ultimate strengths when applied to slab tests conducted by them and to footing tests reported by Richart (4). These studies indicated that an improved method was needed for determining the allowable and ultimate shearing strength of reinforced concrete slabs.

The ultimate shearing strength of a reinforced or a prestressed concrete slab is a function of the net area available to resist shear at the critical section and the ultimate shearing stress available at that section. Several complications arise, however, in trying to predict the magnitude of these quantities.

a. Where should the critical section be taken?

b. What proportion of the total uncracked depth should be used in computing the net area available to resist shear?

c. What are the variables affecting the ultimate shearing stress and how can the stress distribution be predicted in a particular case?

The location of the critical section recommended by the ACI Code is at a distance d from the edge of the loaded area; Elstner and Hognestad recommend that it be taken at the edge of the loaded area, and others recommend that it be taken at some distance intermediate between these two extremes. At the present stage of knowledge, the actual distance to be taken is indeterminate analytically and must be determined empirically by correlation with test results.

The proportion of the total uncracked depth to be used in computing the net area available to resist shear is also a difficult quantity to ascertain since it is dependent on the amount of flexural cracking that has taken place just prior to ultimate failure. Certainly for slabs with high shear-moment ratios the amount of flexural cracking will be smaller than for slabs with low shear-moment ratios. From a practical standpoint a fixed depth equal to the effective depth at the critical section might be used in computing an existing fictious stress and the ultimate shearing stress could then be determined empirically by including the shear-moment ratio as a variable.

Bresler and Pister[5] have shown that for plain concrete specimens the ultimate shearing stress is a function of the applied compressive stress as well as the concrete strength. Their results show that the ultimate shearing stress gradually increases from 0.08 f'_c to 0.22 f'_c as the applied compressive stress increases from 0 to about 0.55 f'_c, then the ultimate shearing stress decreases to about 0.16 f'_c as the compressive stress increases to 0.90 f'_c. For compressive stresses above 0.90 f'_c the ultimate shearing stress decreases very rapidly with any increase in compressive stress. While not directly applicable to the ultimate shearing strength of slabs, these relationships add much weight to the arguments that to compute the ultimate shearing strength of slabs, the interaction of shear and flexure must be considered.

2. <u>Analytical Study of Test Slabs</u>

From the preceding discussion it is evident that a method for predicting the ultimate shearing strength of reinforced and prestressed concrete slabs can only be developed by applying empirical approaches to the results obtained from tests. The three empirical approaches that were applied to the test data in this study are discussed below:

a. 1956 ACI Code Method

The shearing stress as obtained by the use of eq. (1) $v_1 = \frac{V}{bjd}$ was calculated for all test slabs. The results are given in Table 7. V was taken equal to the ultimate test load and d was taken as the average effective depth of the two layers of steel. Values of v_1 and v_1/f'_c refer to ultimate strength. A factor of safety for each slab was computed by dividing v_1 by an allowable shearing stress of 0.025 f'_c, but not greater than 90 psi, as specified in the 1956 ACI code.

As might be expected the application of this method to prestressed as well as reinforced concrete slabs yields a wide range of factors of safety, varying from about 3.3 to 5.3. Even for slabs S-1 and S-2, which were identical reinforced concrete slabs except for concrete strength, the factors of safety were 4.64 and 3.78 respectively.

b. Elstner - Hognestad Method

In 1953 Hognestad(3), recognizing that the ultimate shearing stress in footings and slab was related to the flexural capacity of the specimen, reported on a re-evaluation and re-analysis of Richart's(4) footing tests. Based on this study Hognestad found the ultimate shearing strength of a variety of slabs could be expressed by the empirical equation:

$$v_2 = \frac{V}{7/8bd} = (0.035 + \frac{0.07}{\phi_o})\ f'_c + 130 \qquad \text{-------(2)}$$

or rewriting:

$$\frac{v_2}{f'_c} = 0.035 + \frac{130}{f'_c} + \frac{0.07}{\phi_o} \qquad \text{--------------(2a)}$$

in which v_2 represents the ultimate shearing stress computed at the perimeter of the loaded area, V is the total shearing force, b is the perimeter of the loaded area, and f'_c is the ultimate compressive strength of the concrete. ϕ_o is defined as the ratio of P_{shear} to P_{flex} where P_{flex} is the ultimate flexural capacity the slab would have if it had not failed in shear.

In 1956 Elstner and Hognestad(2) reported on a continuation of the above study in which they gave the results of new tests performed on thirty-nine 6 ft. square reinforced concrete slabs. These slabs were in general supported only at the edges and centrally loaded through a reinforced concrete column stub. To give better correlation for higher concrete strengths, Elstner and Hognestad revised eq. (2a) for slabs without shear reinforcement to give:

$$\frac{v_2}{f'_c} = \frac{P_{shear}}{7/8\ bdf'_c} = \frac{333}{f'_c} + \frac{0.046}{\phi_o} \qquad \text{---------- (3)}$$

Additional equations were presented for slabs with shear reinforcement. In computing the ultimate flexural strength of slabs, the yield line theory(5) was used. Application of eq. (3) to Richart's column footing tests and the slab tests of

Elstner and Hognestad gives values of $P_{test}/P_{calc.}$ ranging from 0.84 to 1.17 with average values near 1.0.

The application of eq. (3) to the reinforced and prestressed concrete lift slabs tested in the present project gave good correlation with test results. The results are shown in Table 7 which indicates a range of values of P_{test}/P_{shear} from 0.88 to 1.20 with an average value of 1.06 for the fifteen slabs. P_{shear} represents the theoretical value obtained using eq. (3) and P_{test} is the actual ultimate value obtained from the test.

In using eq. (3) the computation of P_{flex}, the ultimate flexural capacity of the slab, presents some difficulties when applied to slabs with lift slab collars and also when considering prestressed concrete slabs with unbonded cables.

In the application of the yield line theory for the ultimate flexural strength of the test slabs the question arises of how much, if any, moment can be developed at the perimeter of the collar. In addition what is the effect on the magnitude of this perimeter moment due to recessing of the collar? Observation during the tests and a study of the test results seem to indicate that for flush collars the full moment can develop at the perimeter of the collar and for slabs with collars recessed an amount equal to about 0.5d, where d is the effective depth for the slab with unrecessed collar, this perimeter moment diminishes to nearly zero. A study of collar strains measured by SR-4 gages attached to a collar flush with the concrete indicates that the collar transfers a large compressive force across the column opening. Visual inspection of salvaged collars showed cracks in the weld at the joint of the vertical angle legs. These joints were rewelded with a heavier weld. With large collar recesses the compression force transmitted was small. From these observations it was concluded that collars flush with the concrete developed the full moment capacity of the section at the perimeter of the collar and for collars recessed 0.5d or greater no moment could be developed at the perimeter of the collar.

For the case of a square simply supported slab loaded through a square column with the corners free to lift, the ultimate flexural strength given by the yield line theory is:

$$P_{flex} = 8m\left[\left(\frac{1}{1 - \frac{r}{a}}\right) - \left(3 - 2\sqrt{2}\right)\right] \text{------} \quad (4)$$

where m is the ultimate moment capacity per unit width of the slab and $\frac{r}{a}$ is the ratio of the size of the loaded area to slab width. Eq. (5) assumes that the moment, m, is developed along the diagonals and at the perimeter of the loading area. Fig. 16 shows the yield line pattern for this case.

If no moment is developed at the perimeter of the loaded area, the ultimate flexural strength for a square slab with the corners free to lift becomes:

$$P_{flex} = 16m\ (\sqrt{2} - 1) = 6.627m \quad \text{---------} \quad (5)$$

To give approximate values of P_{flex} for recesses of less than 0.5d a straight line interpolation between the values given by eq. (4) and (5) can be assumed, e.g. for recess of 0.25d, P_{flex} would be 6.627m plus one half of the difference between eq. (5) and (4) for the particular slab dimensions.

When applying the yield line theory to prestressed concrete slabs it must first be assumed that the prestressing steel has sufficient ductility for the full yield line pattern to form and second, when unbonded cables are used, a method for estimating the steel stress for the needed computation of m, the ultimate or yield moment, must be devised. The yield line theory can only be considered an approximate method when applied to the computation of the ultimate flexural strength of a prestressed concrete slab.

The value of the stress in the prestressing steel developed at ultimate moment was designated f_{yield}. The values of P_{flex} calculated for the slabs tested, by using various expressions for the value of f_{yield} which have been proposed for beams, were greatly below the test loads. This indicated that these expressions for beams, probably developed for much longer cables, were not applicable to the short cables used in the test slabs. A study of the test results, along with approximate computations for the cable elongation due to fiber strain, cracking, and deflection of the slab, showed that f_{yield} for the test slabs could be approximated by the expression:

$$f_{yield} = 157{,}000 + 0.4(f_e) \qquad \text{- - - - - - - - - - - - -} \quad (6)$$

where f_e is the effective prestressing steel stress in psi prior to loading. It must be noted that eq. (6) is purely empirical and is applicable only to the short cables in this test series. For longer cables as would be found in prestressed slab construction other relationships should be used which would depend on the situation considered. Once f_{yield} has been established the yield moment per unit width may be computed by means of the following formula:

$$m = f'_c\, d^2 q\,(1 - 0.59q) \qquad (7)$$

in which $q = pf_y/f'_c$ and average values for p and d are used for the two layers of steel.

Using the method described above to calculate P_{flex}, calculated values for P_{shear} were then obtained using eq. (3). Pertinent data obtained using this method are given in Table 7.

In order to show the effect of concrete strength, f'_c, and ultimate shear-flexure ratio, ϕ_o, the values given in Table 7 were adjusted first to a common value of $f'_c = 3500$ psi and then to a common value of $\phi_o = 1.0$ using the equations

$$\frac{v_2}{f'_c}\ (\text{corr. to } f'_c = 3500) = \frac{v_2}{f'_{c\ obs}} = \frac{333}{f'_c} + \frac{333}{3500}$$

$$\frac{v_2}{f'_c}\ (\text{corr. to } \phi_o = 1.0) = \frac{v_2}{f'_{c\ obs}} - \frac{0.046}{\phi_o} + \frac{0.046}{1.0}$$

These results are shown graphically in Figs. 17 and 18 which indicate a decrease in ultimate shearing strength, v_2/f'_c with increasing values of ϕ_o or f'_c.

c. Other Empirical Approaches

A number of other empirical approaches were tried but none seemed to fit the test data as well as the method described in the preceding paragraph.

An attempt was made to develop an empirical equation which would not necessitate the computation of P_{flex}. It was found that the following expression for the ultimate shear strength gave a good correlation with the test results of the prestressed slabs:

$$\frac{P_{shear}}{bdf'_c} = 0.175 - 0.0000242\ f'_c + 0.000020\ \frac{F_e}{s} \quad \text{- - - - - -} \quad (8)$$

in which F_e is the effective prestress force in lbs. and s is the cable spacing in inches. A comparison between test and calculated values using eq. (8) may be found in Table 8 and is shown graphically in Fig. 19.

For reinforced slabs a similar expression was determined as follows:

$$\frac{P_{shear}}{bdf'_c} = 0.175 - 0.0000242\ f'_c + 0.000100\ pf_y d \quad \text{- - -} \quad (9)$$

in which p is the steel percentage and f_y is the yield point of the reinforcement. This equation gave good correlation for the few reinforced slabs in this test program, as shown in Table 8. However, when eq. (9) was applied to the slabs tested by Elstner and Hognestad, which covered a greater range of variables, the comparison of test and calculated values was not as good as that obtained using eq. (3). The discrepancies were generally on the conservative side as can be seen by a study of Table 9 and Fig. 20.

IV. EXPERIMENTAL RESULTS

1. General Behavior

All slabs were loaded to ultimate failure which in all cases occurred by a final punching of the steel collar through the concrete. The manner of failure was sudden and violent in some cases and often accompanied by the dropping out from the bottom of the slab of a large amount of concrete.

In the case of the prestressed concrete slabs, the sequence of events during the test was generally as follows:

a. The first visible flexural cracks appeared at 40 to 60 percent of the ultimate load for the slabs with flush collars and at succeedingly higher percentages as the amount of collar recess was increased.

b. The corners of the slab lifted and this was generally followed by the appearance of vertical cracks at the edges of the slab a short distance in from the corners.

c. The flexural cracks on the bottom of the slab spread into a crack pattern and in most cases reached the corners of the slab, the amount of flexural cracking visible just prior to failure varied considerably.

d. The collar punched through with concrete dropping from the bottom of slab.

e. The failure cone was very flat and extended in some cases beyond the edge of the slab.

f. Upon release of the load the tension in the prestressing cables tended to put the bottom of the slab back into compression and in cases where a large amount of concrete had fallen from the bottom, leaving no compression area available, the slab buckled upwards.

The behavior of the three reinforced concrete slabs tested differed from the prestressed concrete slabs in that the first visible flexural cracks occurred earlier, at about 15 to 25 percent of the ultimate load, and there was no tendency for the slab to spring back up after the test load had been released. In addition the slabs seem to hold together better at failure, probably due to the closer spacing of the reinforcement.

After each test, the support frame with the slab on it was lifted by means of a crane and a photograph was taken of the bottom of the slab. These photographs are shown in Figs. 21 through 23. Remembering that slabs S-1, S-2, and S-16 were the only reinforced concrete slabs it is apparent that the failure of the prestressed concrete slabs was more violent in nature than the reinforced slabs, generally produced by the squeezing of the cables after failure as described in stage (f) above.

All failures were due to a combination of shear and flexure with one or the other dominating. By studying the load-deflection curves given in Figs. 24 through 29 one can estimate the relative amount of flexural cracking that took place prior to the failure in each case.

A typical set of load-deflection profiles for slab S-6 is shown in Fig. 30. Similar plots were made for each slab in studying test results.

2. Detailed Description for Each Group

a. Group I - The two reinforced concrete lift slabs of Group I differed only in concrete strength. Flexural cracking began at the corners of the column opening at a load of 20 kips for both slabs. At a load of 45 kips a yield line crack pattern was formed along with a multitude of cracks parallel to the reinforcing steel. Corner levers also started to develop. Ultimate loads were nearly the same; 105 kips for S-1 and 109 kips for S-2. The failure surface sloped away from the edge of the collar at an angle of about 20 degrees to the plane of the slab.

The load-deflection curves shown in Fig. 24 indicate shear failures. Strain measurements of the reinforcing steel showed that local yielding started just prior to failure. Concrete strains also indicated a shear failure with yielding of the steel reinforcement just starting.

b. Group II - The five prestressed lift slabs of Group II showed a variation in ultimate strength from 60 kips to 121.5 kips. This variation appears to be related principally to the location and amount of prestress force. The first visible flexural cracks, at the corners of the column opening, were observed at loads between 40 and 60 percent of the ultimate loads. The full yield line crack pattern was developed at load values of 70 to 80 percent of the ultimate load values.

The mode of failure for all slabs of this group was a cone of concrete punching out of the slab. The top of this cone was the lift collar. The cone surface made an angle of about 18 to 22 degrees with the plane of the slab. The load-deflection curves for this group, shown in Fig. 25, indicate that all the slabs, particularly S-5, S-6, and S-7, were approaching an ultimate flexural failure.

c. Group III - The two prestressed lift slabs of Group III had 16 inch collars, and differed from each other only in the amount of prestress force. Flexural cracking was observed at the corners of the column opening at loads of 45 kips for S-9 and 55 kips for S-10, the slab with higher prestress. Full yield line crack patterns were developed at loads of 65 kips and 85 kips for S-9 and S-10, respectively. Failure loads of 105 kips and 118 kips were recorded for the two slabs.

The failure of S-9 was definitely flexural as the concrete at the top surface crushed along the diagonals. The column deflection exceeded the range of the dial gage and was estimated to be about 1.5 inches just prior to failure. The crack pattern on the bottom surface prior to failure showed large cracks along the assumed yield line pattern including cracks outlining the collar perimeter.

The collar of slab S-10 punched out a cone of concrete in a manner similar to those slabs of Group II. The load-deflection curve, Fig. 26, and concrete strains indicated that the failure load approached the ultimate flexural strength.

d. Group IV - The amount of collar recess was the only variable in the 10 in. thick slabs of Group IV. The first cracks at the corners of the column opening were visible at a load of about 95 kips. The yield line crack pattern developed fully in S-11 at a load of 180 kips. The ultimate loads were 225 kips, 171.5 kips, and 109 kips for slabs S-11, S-12, and S-13 respectively with recesses of 0, 2 inches, and 4 inches in the 10 inch slabs.

The mode of failure was that of a shear failure for all three slabs. The cone surface formed an angle of 20 to 25 degrees with the plane of the slab. The load-deflection curves, Fig. 27, show that S-11 was approaching ultimate flexural failure but that S-12 and S-13 were well below the ultimate flexural strength.

e. Group V - The two 8 inch thick slabs of Group V differed from each other only in the depth of collar recess. The first cracks were visible at the corners of the column opening on the bottom of the slab at a load of about 90 kips. The full yield line crack pattern developed at a load of 130 kips in S-14. The ultimate loads were 168 kips and 120 kips for S-14 and S-15 respectively.

A shearing type of failure was observed in both slabs. The load-deflection curves of Fig. 28 show that S-14 was approaching a ultimate flexural failure.

f. Group VI - The special slab of Group VI showed an irregular crack pattern outside the column area at a load of 50 kips. At a load of 80 kips, the yield line crack pattern had developed. Failure was at a load of 250 kips.

A shearing type failure was observed with cone of failure meeting the upper slab surface at the perimeter of the column. The load-deflection curve of Fig. 29 indicates that the condition of general yielding had not been reached.

V. DISCUSSION AND CONCLUSIONS

A total of fifteen 6 ft. square specimens were tested to study the ultimate shearing strength of prestressed and reinforced concrete lift slabs. Twelve of the slabs were prestressed with unbonded cables and three of the slabs were made of reinforced concrete. Major variables were nominal concrete strength (3000 or 4500 psi), average initial prestress (250 to 500 psi), collar size (13 or 16 in.), slab thickness (6, 8, or 10 in.), and amount of collar recess (0, 1, 2, or 4 in.)

Final failure of all slabs occurred when the steel collar punched through the slab. A variable amount of flexural cracking was visible just prior to failure and was generally smaller in the thicker slabs and the slabs with column recesses.

On the basis of the analytical and test results reported herein the following conclusions are advanced:

1. The use of the present 1956 ACI Code method of calculating the allowable shear load when applied to prestressed and reinforced concrete lift slabs yields a wide range of factors of safety on the ultimate strength, from about 3.3 to 5.3.

2. The actual ultimate punching shearing stress at the edge of the collar divided by f'_c, as expressed by $\frac{v_2}{f'_c} = \frac{P_{test}}{7/8\ bdf'_c}$, varied between 0.118 and 0.211 and therefore cannot be considered to be a constant.

3. The ultimate shearing stress at the edge of the collar can be predicted by the following expression:

$$\frac{v_2}{f'_c} = \frac{P_{shear}}{7/8\ bdf'_c} \cong \frac{333}{f'_c} + \frac{0.046}{\phi_o}$$

in which b is the perimeter of the collar edge, d is the effective depth <u>at the collar</u>, and ϕ_o is the ratio of the ultimate shear capacity to the ultimate flexural capacity computed without regard to shear. The ultimate flexural capacity may be computed by means of the yield line theory for slabs. The above expression will yield sufficient accuracy for prestressed concrete slabs as well as reinforced concrete slabs provided a suitable method is used in calculating the ultimate flexural capacity.

4. The ultimate shearing strength of prestressed concrete slabs similar to those included in this program may also be predicted by means of the following empirical expression:

$$\frac{P_{shear}}{bdf'_c} = 0.175 - 0.0000242\ f'_c + 0.000020\ \frac{F_e}{s}$$

in which b, d, f'_c have the same definition as in paragraph 3, F_e

is the effective prestress in lbs., and s is the cable spacing in inches.

5. A steel collar cast flush with the slab will develop the full moment existing at the perimeter of the collar. The amount of this moment developed will decrease as the collar is recessed and will be essentially zero when the collar has been recessed 0.5d, where d is the effective depth for the slab with unrecessed collar.

6. Adequate provisions should be made in design so that ultimate flexural capacity will govern failure rather than ultimate shear capacity since a shear failure may be sudden and without warning.

VI. REFERENCES

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Table 1

Description of Test Specimens

Group	Slab No.	Reinforced or Prestressed	Collar Size in.	Nominal Concrete Strength psi	% Reinforcement or Average prestress	Thickness in.	Collar Recess in.
I	S-1	R	13	3000	2%	6	-
	S-2	R	13	4500	2%	6	-
II	S-4	P	13	4500	450 psi	6	1
	S-5	P	13	3000	250 psi	6	-
	S-6	P	13	4500	250 psi	6	-
	S-7	P	13	3000	500 psi	6	-
	S-8	P	13	4500	500 psi	6	-
III	S-9	P	16	4500	250 psi	6	-
	S-10	P	16	4500	500 psi	6	-
IV	S-11	P	13	4500	300 psi	10	-
	S-12	P	13	4500	300 psi	10	2
	S-13	P	13	4500	300 psi	10	4
V	S-14	P	13	4500	375 psi	8	-
	S-15	P	13	4500	375 psi	8	2
VI	S-16	R	18	3000	1%	8	-

Table 2

Sieve Analysis of Aggregates

Kind of Aggregate	Percentage Retained on Sieve									Fineness Modulus
	3/4 in.	1/2 in.	3/8 in.	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	
Sand	-	-	-	0	14	45	72	91	98	3.20
Gravel	4	39	60	96	-	-	-	-	-	6.59

Table 3

Properties of Concrete Mixtures

Mix. No.	Nominal Strength psi.	Water/Cement Ratio gal/sack	Water/Cement Ratio by weight	Mix Ratio by Weight SSD	% Sand by Weight	Cement Factor Sacks/cu.yd.
1	3000	7.6	0.67	1:3.15:3.55	47.0	5.1
2	4500	5.5	0.49	1:2.17:3.20	40.5	6.3
3	4500	5.0	0.44	1:1.78:2.93	37.8	7.0

Table 4

Average Values of Concrete Properties

Group	Slab No.	Mix No.	f'_c psi	f'_t[a] psi	E_c[b] psi	Kelly Ball Penetration in.	Age at Test Days
I	S-1	1	2813	462	2.37×10^6	2.9	14
	S-2	2	4059	463	3.13	1.9	14
II	S-4	2	3956	571	3.06	2.1	14
	S-5	1	2935	455	2.75	2.0	14
	S-6	3	4700	456	3.52	2.3	13
	S-7	1	2890	488	2.66	1.6	13
	S-8	3	4348	605	3.37	2.3	14
III	S-9	3	4376	623	3.55	2.3	14
	S-10	3	4668	590	3.13	2.3	14
IV	S-11	3	5125	667	2.78	2.1	14
	S-12	3	4919	507	2.69	2.2	14
	S-13	3	5228	519	2.88	2.3	14
V	S-14	3	4800	561	3.46	2.2	14
	S-15	3	5121	472	3.45	2.2	14
VI	S-16	1	3329	517	2.89	1.5	14

a. From 6 x 6 x 20 in. beams loaded at third points of 18 in. span.

b. Secant modulus of elasticity at 1000 psi, from 6 x 12 in. cylinders.

Table 5

Properties of 1/4 Inch Diameter Steel Prestressing Wire

Sample No.	Sample Lot[c]	Proportional Limit f_{pl} ksi	Yield Point[a] f_y ksi	Ultimate Strength f_u ksi	Modulus of Elasticity[b] E_s ksi	% Elongation in 10 in.
1	I	151	227	252	26,800	4.0
2	I	140	230	252	27,100	2.4
3	I	145	223	252	26,400	2.0
4	II	152	207	246	26,100	6.4
5	II	147	204	243	26,200	5.5
6	II	130	212	246	25,300	5.7
7	II	157	217	256	26,700	5.0

a. As measured by 0.2% offset.

b. Up to proportional limit.

c. Cables were purchased in two lots, I and II.

Table 6

Properties of Intermediate Grade ASTM A-305 Bars

Sample No.	Bar No.	Yield Strength f_y ksi	Ultimate Strength f_u ksi	Modulus of Elasticity E_s ksi	% Elongation in 8 in.
1	8	47.0	75.8	29,800	25.8
2	8	48.0	76.4	30,400	26.8
3	8	50.2	76.4	28,200	25.6
4	8	49.7	76.4	27,100	24.5
5	6	52.0	82.4	27,500	20.6
6	6	50.7	81.7	28,100	22.1
7	6	52.7	84.3	--	--
8	6	50.7	83.3	--	--

Table 7a

Test Results

Group	Slab No.	p percent	t in.	Average d at collar in.	r in.	Recess in.	f'_c psi	f_y[a] ksi	P_{test} kips
I	S-1	2.500	6.0	4.25	13	-	2813	48.0	105.0
	S-2	2.500	6.0	4.25	13	-	4059	48.0	109.0
II	S-4	0.452	6.0	2.63	13	1.0	3956	232.5	80.0
	S-5	0.547	6.0	3.00	13	-	2935	220.3	60.0
	S-6	0.452	6.0	3.63	13	-	4700	220.3	78.5
	S-7	0.562	6.0	4.38	13	-	2890	225.9	121.5
	S-8	0.452	6.0	3.63	13	-	4348	235.6	99.5
III	S-9	0.452	6.0	3.63	16	-	4376	220.3	105.0
	S-10	0.452	6.0	3.63	16	-	4668	235.6	118.0
IV	S-11	0.215	10.0	7.63	13	-	5125	235.6	225.0
	S-12	0.215	10.0	5.63	13	2.0	4919	235.6	171.5
	S-13	0.215	10.0	3.63	13	4.0	5228	235.6	109.0
V	S-14	0.293	8.0	5.63	13	-	4800	235.6	168.0
	S-15	0.293	8.0	3.63	13	2.0	5121	235.6	120.0
VI	S-16	1.517	8.0	6.50	18	-	3329	48.0	250.0

a. f_y for prestressed slabs given by 157 + 0.4 fe (in ksi)

Table 7b

Calculated Values by ACI Code or Eq. 3

Slab No.	1956 ACI Method v_1 psi	v_1/f'_c	Factor of safety	Elstner-Hognestad Method Eq. (3) $\frac{v_2}{f'_c}$(test)	P_{flex} kips	$\frac{P_{test}}{P_{flex}}$	P_{shear} kips	$\phi_o = \frac{P_{shear}}{P_{flex}}$	$\frac{P_{test}}{P_{shear}}$
S-1	328	0.116	4.64	0.194	136.3	0.770	98.9	0.725	1.061
S-2	341	0.084	3.78	0.139	149.3	0.739	112.4	0.753	0.971
S-4	476	0.121	5.29	0.169	83.5	0.959	66.7	0.800	1.200
S-5	300	0.104	4.16	0.150	63.0	0.953	63.7	1.011	0.943
S-6	304	0.065	3.38	0.101	84.9	0.925	89.0	1.048	0.884
S-7	366	0.127	5.08	0.211	142.5	0.853	102.9[b]	0.722	1.181
S-8	386	0.089	4.30	0.139	98.7	1.009	90.8	0.919	1.096
S-9	356	0.081	3.96	0.118	90.5	1.160	103.3	1.142	1.017
S-10	400	0.086	4.45	0.125	106.1	1.112	109.6	1.031	1.078
S-11	298	0.058	3.30	0.127	227.8	0.990	205.9	0.905	1.094
S-12	358	0.073	3.98	0.136	202.5	0.846	158.7	0.785	1.080
S-13	422	0.081	4.69	0.126	180.0	0.606	116.2	0.646	0.938
S-14	351	0.073	3.90	0.138	164.3	1.022	147.9	0.899	1.137
S-15	465	0.091	5.15	0.142	140.2	0.855	106.8	0.761	1.075
S-16	355	0.107	4.28	0.183	250.0	1.000	210.0[c]	0.847	1.191

b. Shear taken by parabolic cables neglected.

c. Shear taken by shear reinforcement included. (See Ref. 2 for method of computation.)

Table 8

Comparison of Test and Calculated Values Using Equations 8 and 9

Group	Slab No.	b in.	d at collar in.	f'_c psi	F_e lb.	s in.	pf_yd lb/in	P_{calc} kips	P_{test} kips	$\frac{P_{test}}{P_{calc}}$
I	S-1	52	4.25	2813	--	--	510	98.2	105.0	1.069
	S-2	52	4.25	4059	--	--	510	109.5	109.0	0.995
II	S-4	52	2.63	3956	48,600	18	--	72.0	80.0	1.111
	S-5	52	3.00	2935	27,000	18	--	61.4	60.0	0.977
	S-6	52	3.63	4700	27,000	18	--	80.9	78.5	0.970
	S-7	52	4.38	2890	36,000	12	--	108.5	121.5	1.120
	S-8	52	3.63	4348	54,000	18	--	106.4	99.5	0.935
III	S-9	64	3.63	4376	27,000	18	--	100.6	105.0	1.044
	S-10	64	3.63	4668	54,000	18	--	132.1	118.0	0.893
IV	S-11	52	7.63	5125	54,000	18	--	225.6	225.0	0.997
	S-12	52	5.63	4919	54,000	18	--	166.9	171.5	1.028
	S-13	52	3.63	5228	54,000	18	--	106.9	109.0	1.020
V	S-14	52	5.63	4800	54,000	18	--	166.8	168.5	1.010
	S-15	52	3.63	5121	54,000	18	--	107.2	120.0	1.119

Prestressed slabs

$$\frac{P}{bdf'_c} = 0.175 - 0.0000242\, f'_c + 0.0000200\, F_e/s$$

Reinforced slabs

$$\frac{P}{bdf'_c} = 0.175 - 0.0000242\, f'_c + 0.0001000\, pf_yd$$

Table 9

Elstner and Hognestad Slab Results Compared with Equation 8

Series	Slab No.	b in.	p %	d in.	f_y ksi	f'_c psi	pf_yd lb/in	P_{calc} kips	P_{test} kips	$\frac{P_{test}}{P_{calc}}$
I	A-1a	40	1.15	4.63	48.2	2040	257	57.2	68	1.189
	A-1b	40	1.15	4.63	48.2	3660	257	76.0	82	1.079
	A-1c	40	1.15	4.63	48.2	4210	257	77.0	80	1.039
	A-1d	40	1.15	4.63	48.2	5340	257	70.7	79	1.117
	A-1e	40	1.15	4.63	48.2	2940	257	70.6	80	1.133
	A-2a	40	2.47	4.50	46.6	1980	518	63.8	75	1.176
	A-2b	40	2.47	4.50	46.6	2830	518	80.6	90	1.117
	A-2c	40	2.47	4.50	46.6	5430	518	93.2	105	1.127
	A-7b	40	2.47	4.50	46.6	4050	518	93.9	115	1.225
	A-3a	40	3.70	4.50	46.6	1850	776	69.2	80	1.156
	A-3b	40	3.70	4.50	46.6	3280	776	102.3	100	0.978
	A-3c	40	3.70	4.50	46.6	3850	776	110.5	120	1.044
	A-3d	40	3.70	4.50	46.6	5010	776	118.5	123	1.038
II	A-4	56	1.15	4.63	48.2	3790	257	96.2	90	0.936
	A-5	56	2.47	4.50	46.6	4030	518	131.4	120	0.913
	A-6	56	3.70	4.50	46.6	3630	776	150.8	112	0.743
III	A-7	40	2.47	4.50	46.6	4130	518	94.3	90	0.954
	A-8	40	2.47	4.50	46.6	3180	518	85.7	98	1.144
VI	A-11	40	2.47	4.50	46.6	3760	518	91.9	119	1.295
	A-12	40	2.47	4.50	46.6	4120	518	94.3	119	1.262
VIII	B-9	40	2.00	4.50	49.5	6370	446	75.0	113	1.513
	B-11	40	3.00	4.50	59.3	1960	801	73.3	74	1.010
	B-14	40	3.00	4.50	47.2	7330	637	80.9	130	1.607

$$\frac{P}{bdf'_c} = 0.175 - 0.0000242\, f'_c + 0.0001000\, pf_yd$$

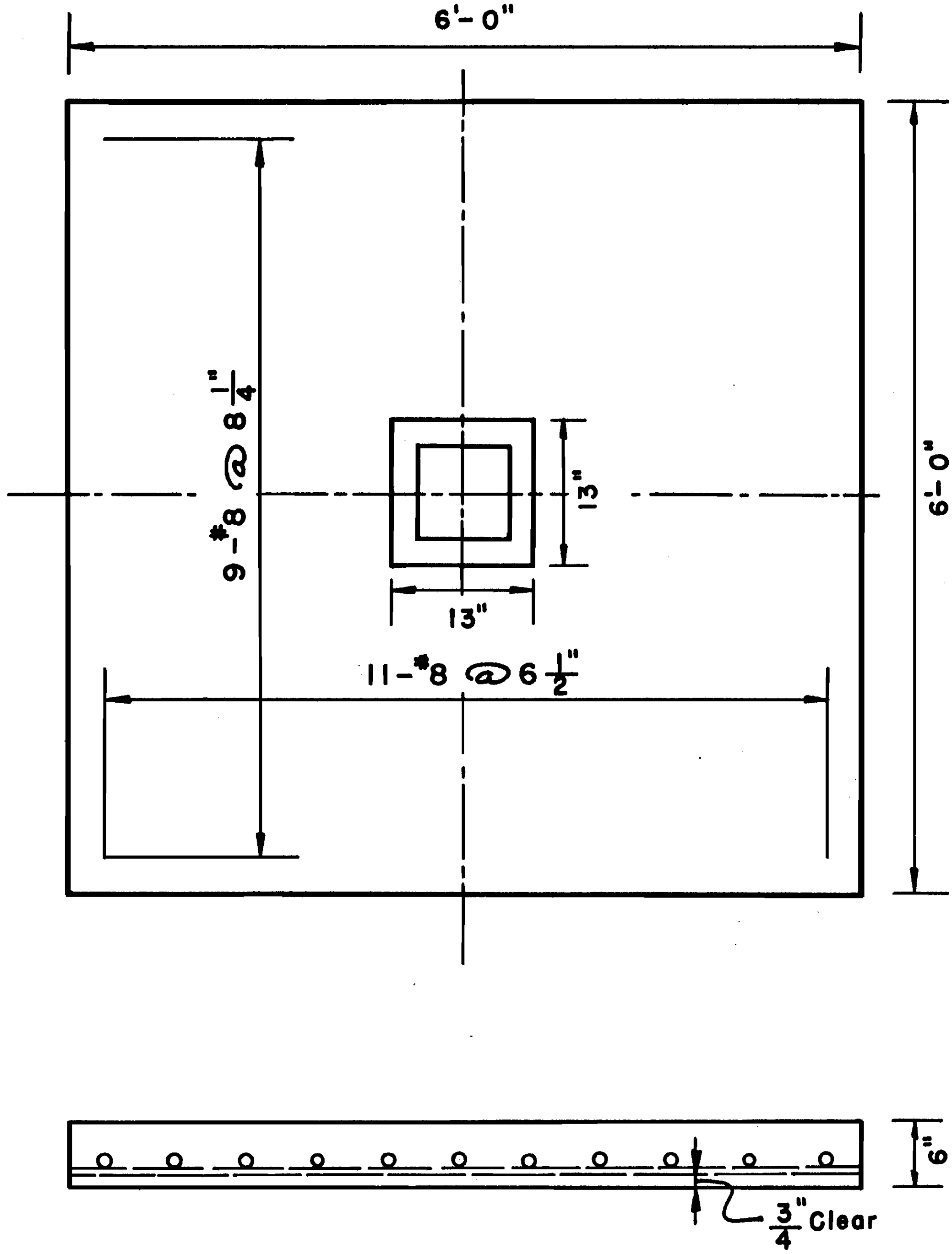


FIG. I LAYOUT OF REINFORCED CONCRETE LIFT SLABS
(S-1 AND S-2)

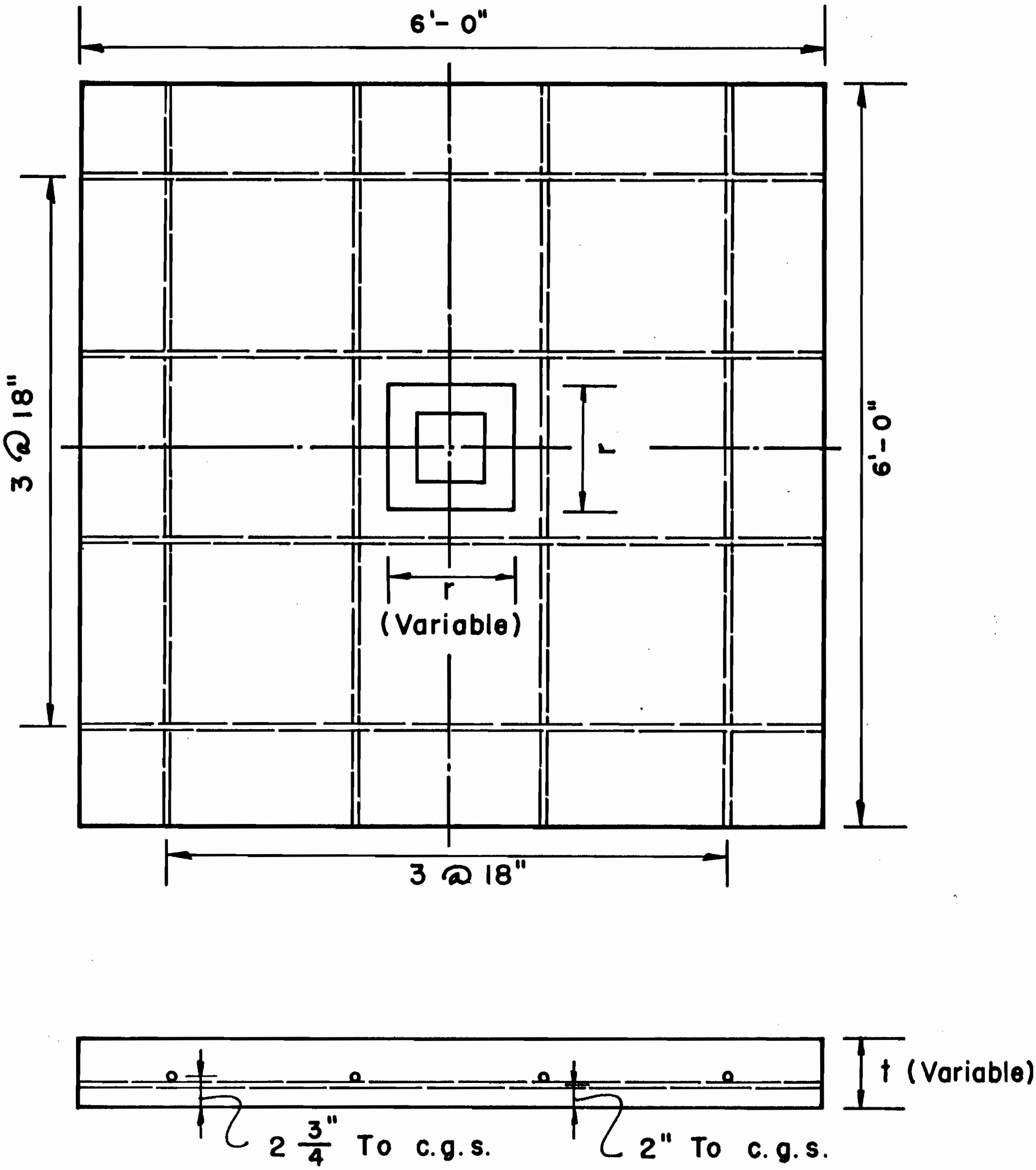


FIG. 2 LAYOUT OF PRESTRESSED CONCRETE LIFT SLABS

(S-4, S-6, S-8, S-9, S-10, S-11, S-12, S-13, S-14, AND S-15)

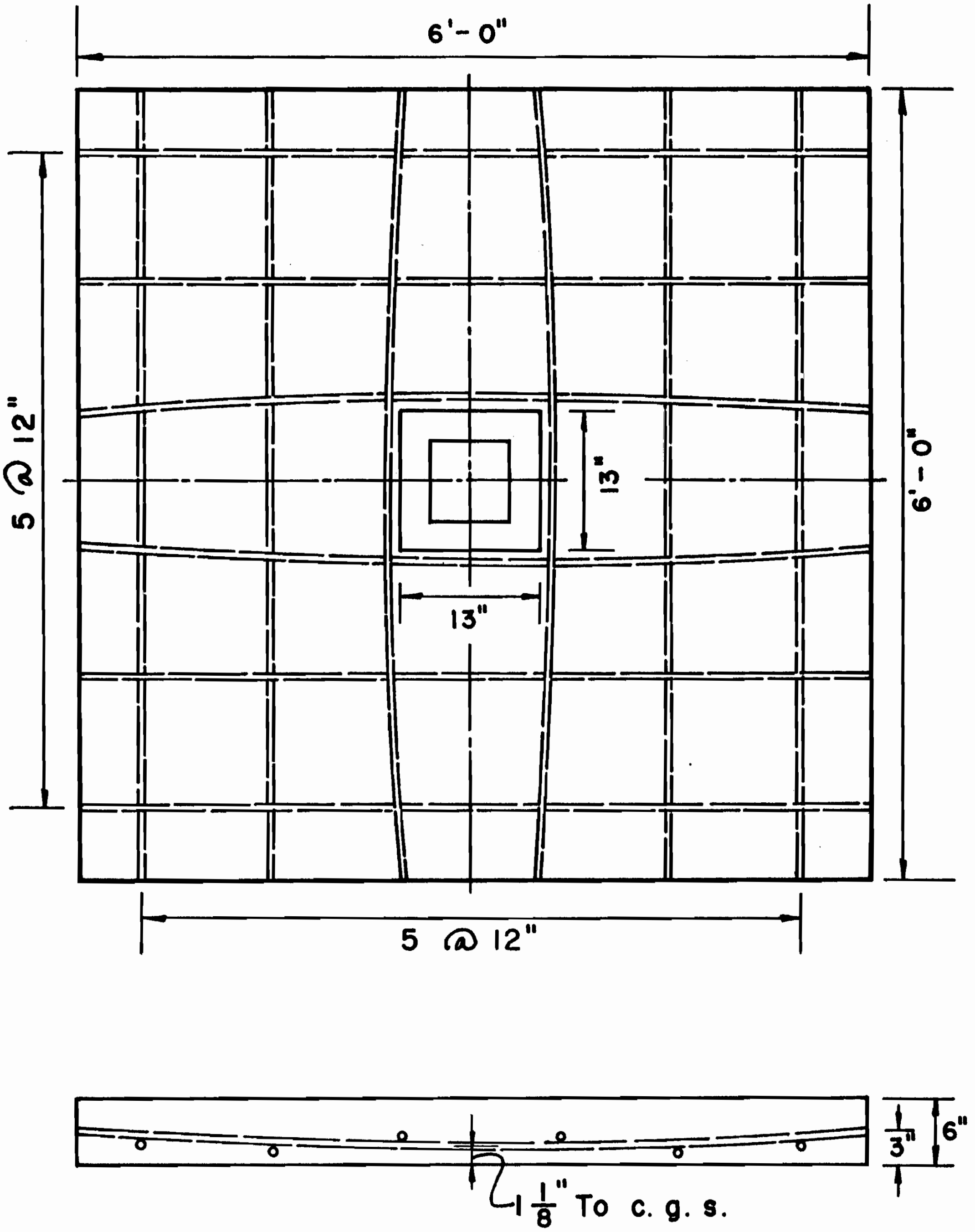


FIG. 3 LAYOUT OF PRESTRESSED CONCRETE LIFT SLAB

(S-7)

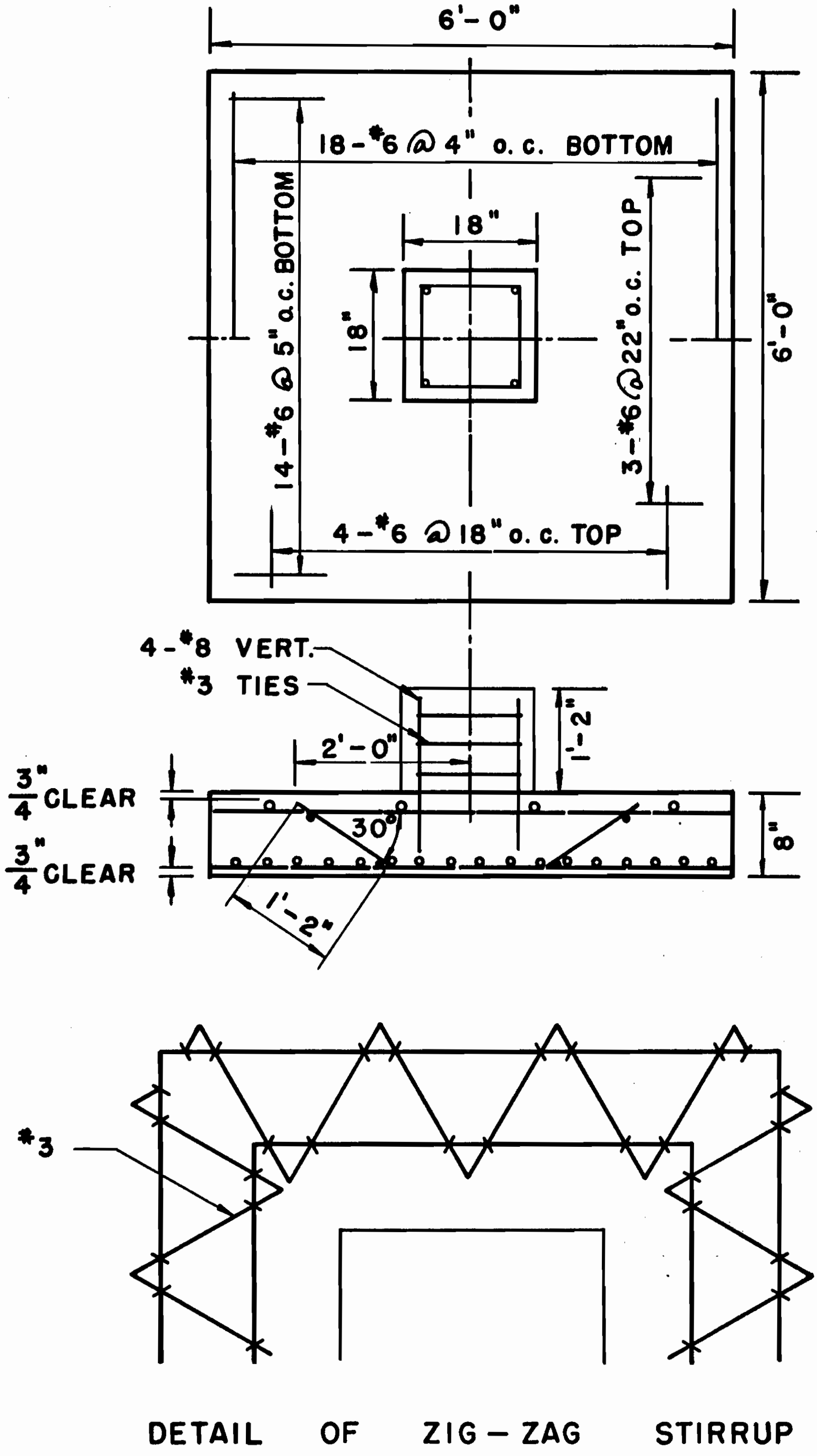
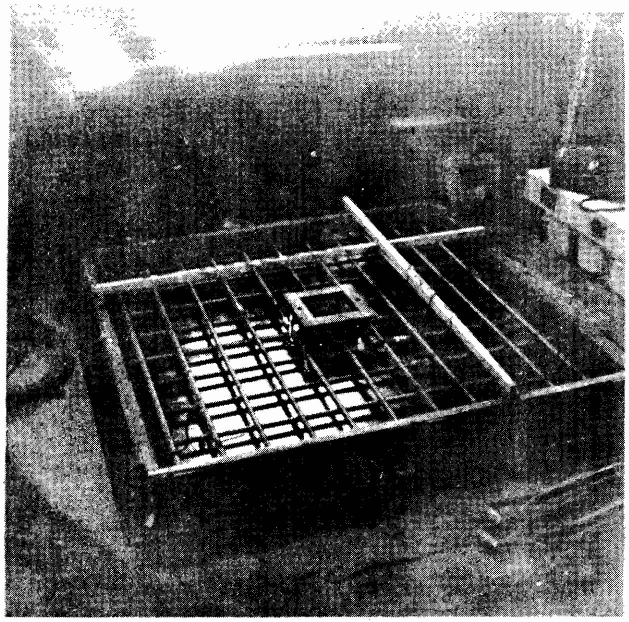
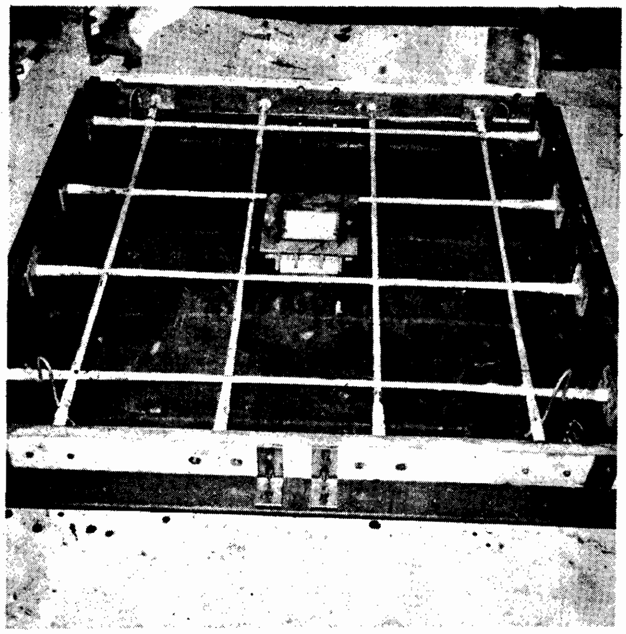


FIG. 4 LAYOUT OF SPECIAL REINFORCED CONCRETE SLAB

(S - 16)



REINFORCED CONCRETE SLAB



PRESTRESSED CONCRETE SLAB

FIG. 5 PHOTOGRAPHS OF SLAB ARRANGEMENTS JUST TO PRIOR TO CASTING

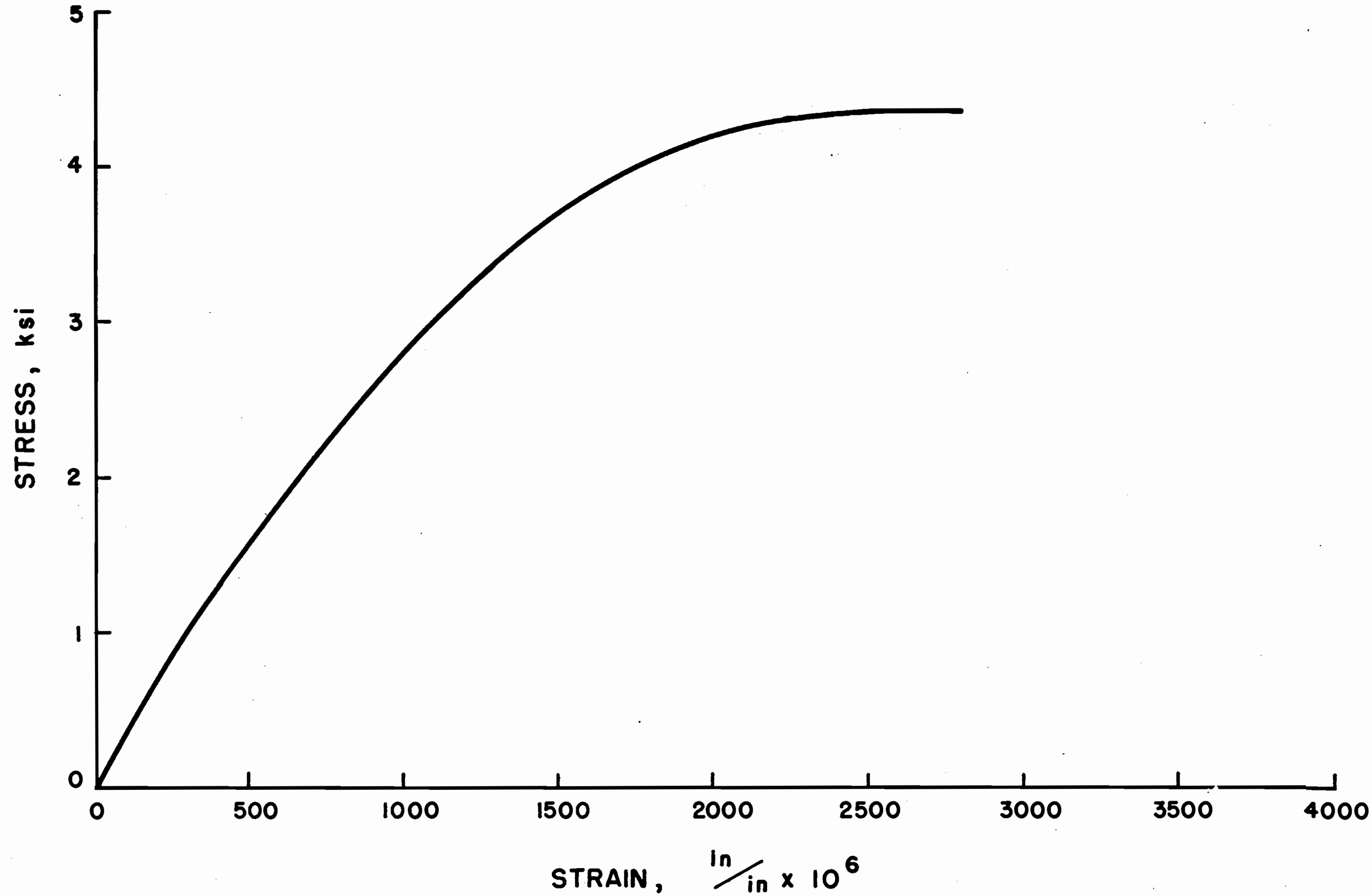


FIG. 6 TYPICAL STRESS STRAIN CURVE FOR 6" x 12" CONCRETE CYLINDER AT AGE OF 14 DAYS

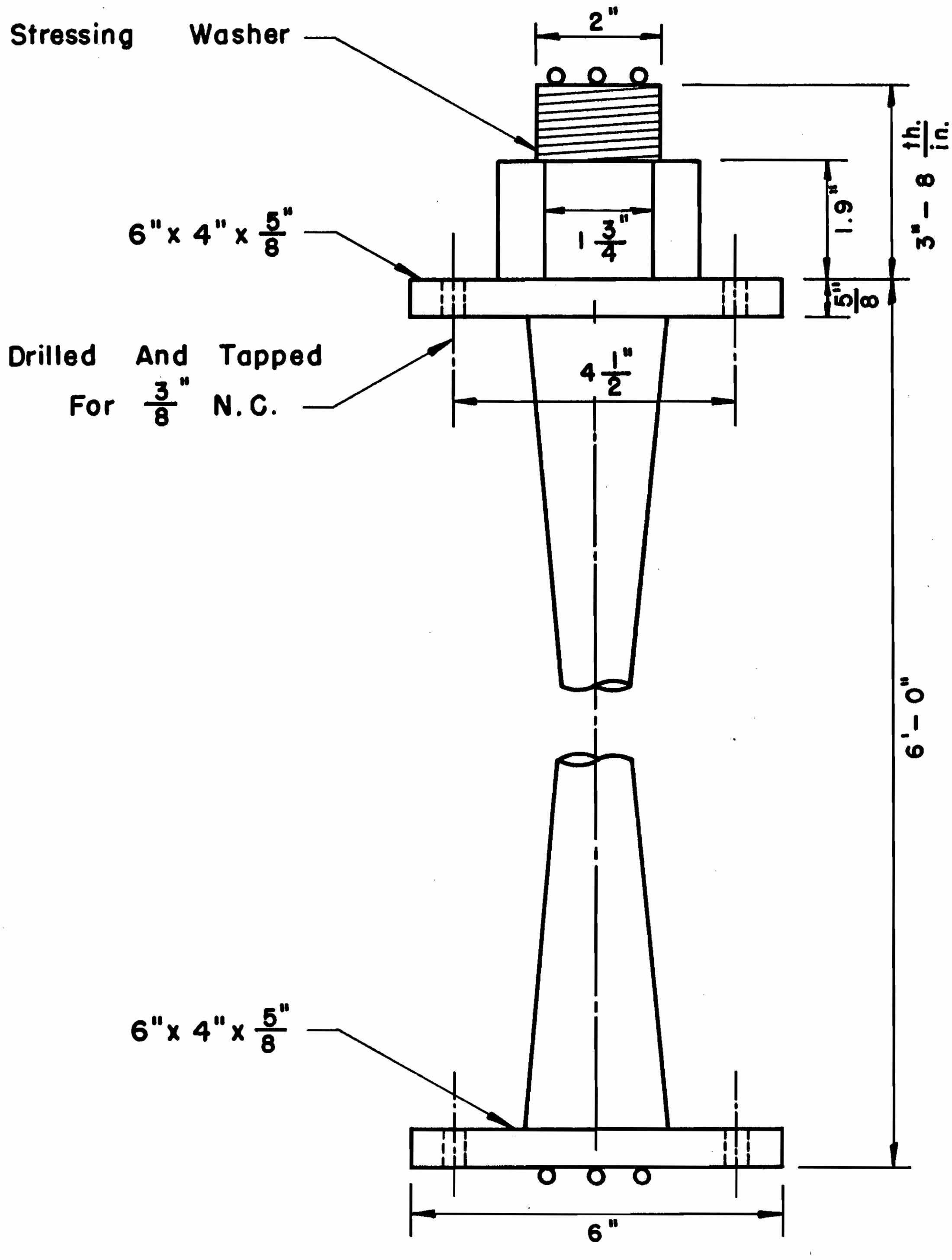


FIG. 7 DETAILS OF CABLE ASSEMBLY AND ANCHORAGE

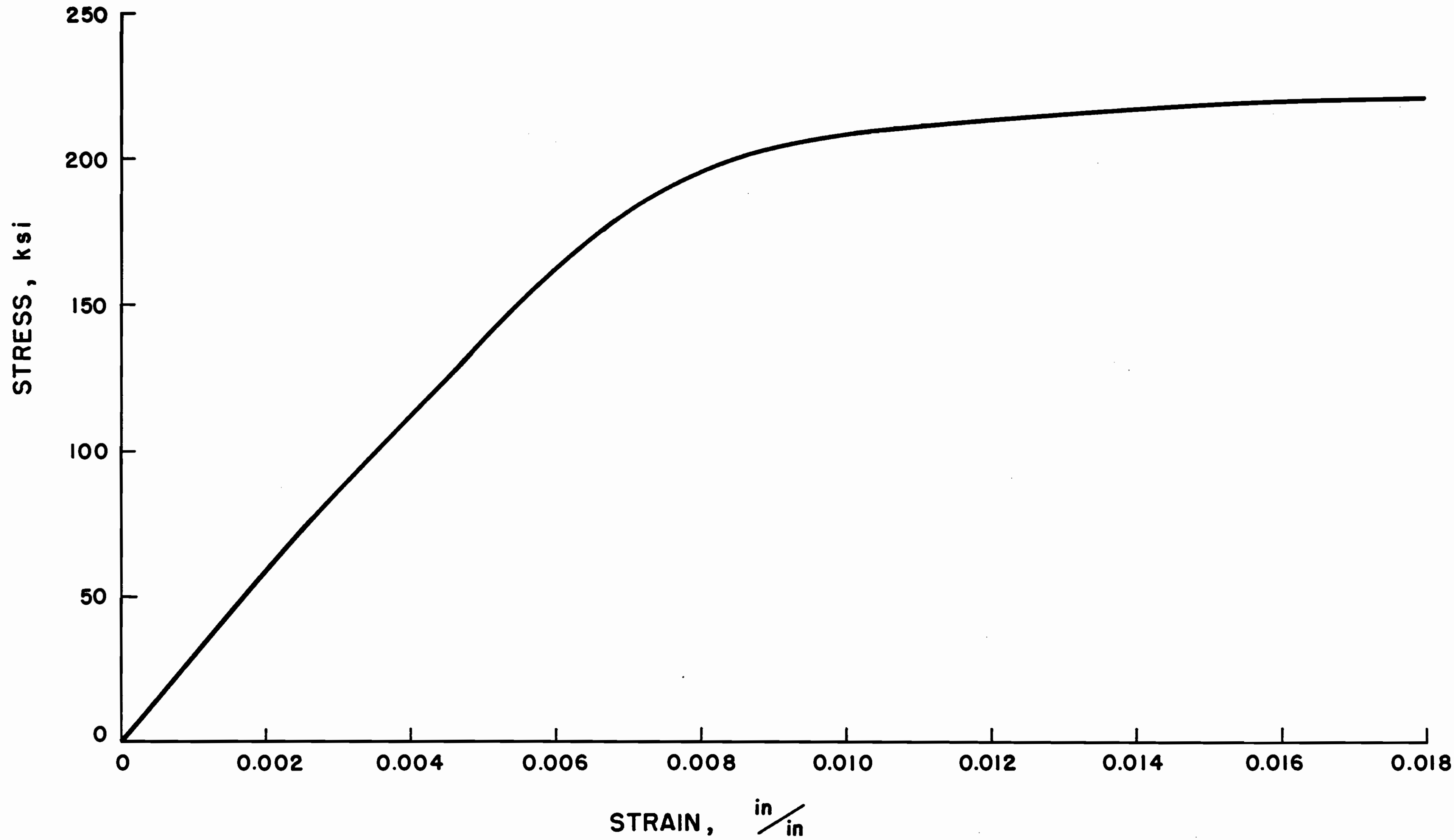


FIG. 8 TYPICAL STRESS STRAIN CURVE FOR $\frac{1}{4}$-in. DIA. STEEL PRESTRESSING WIRE

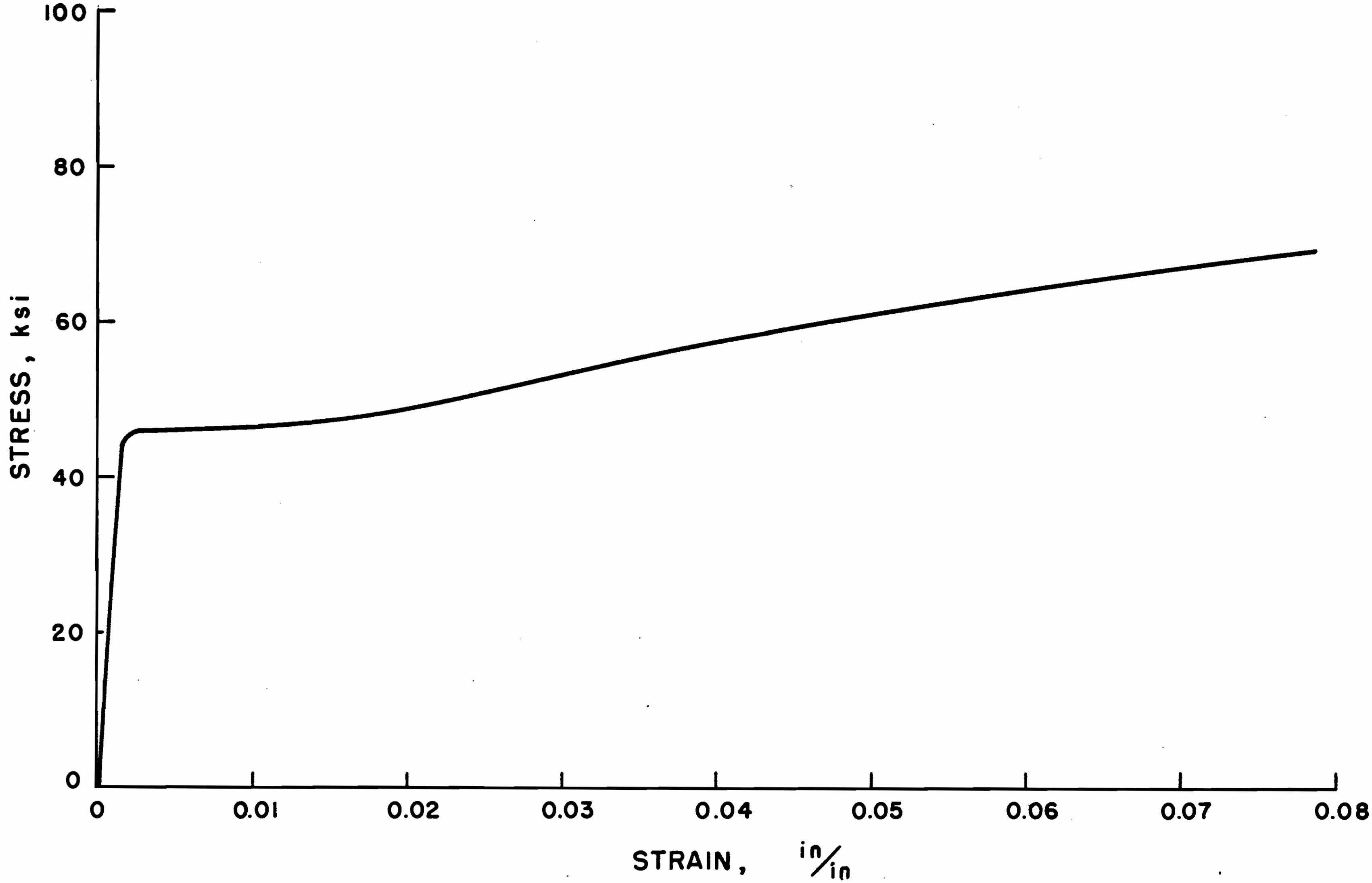


FIG. 9 TYPICAL STRESS STRAIN CURVE FOR INTERMEDIATE GRADE STEEL REINFORCING BARS

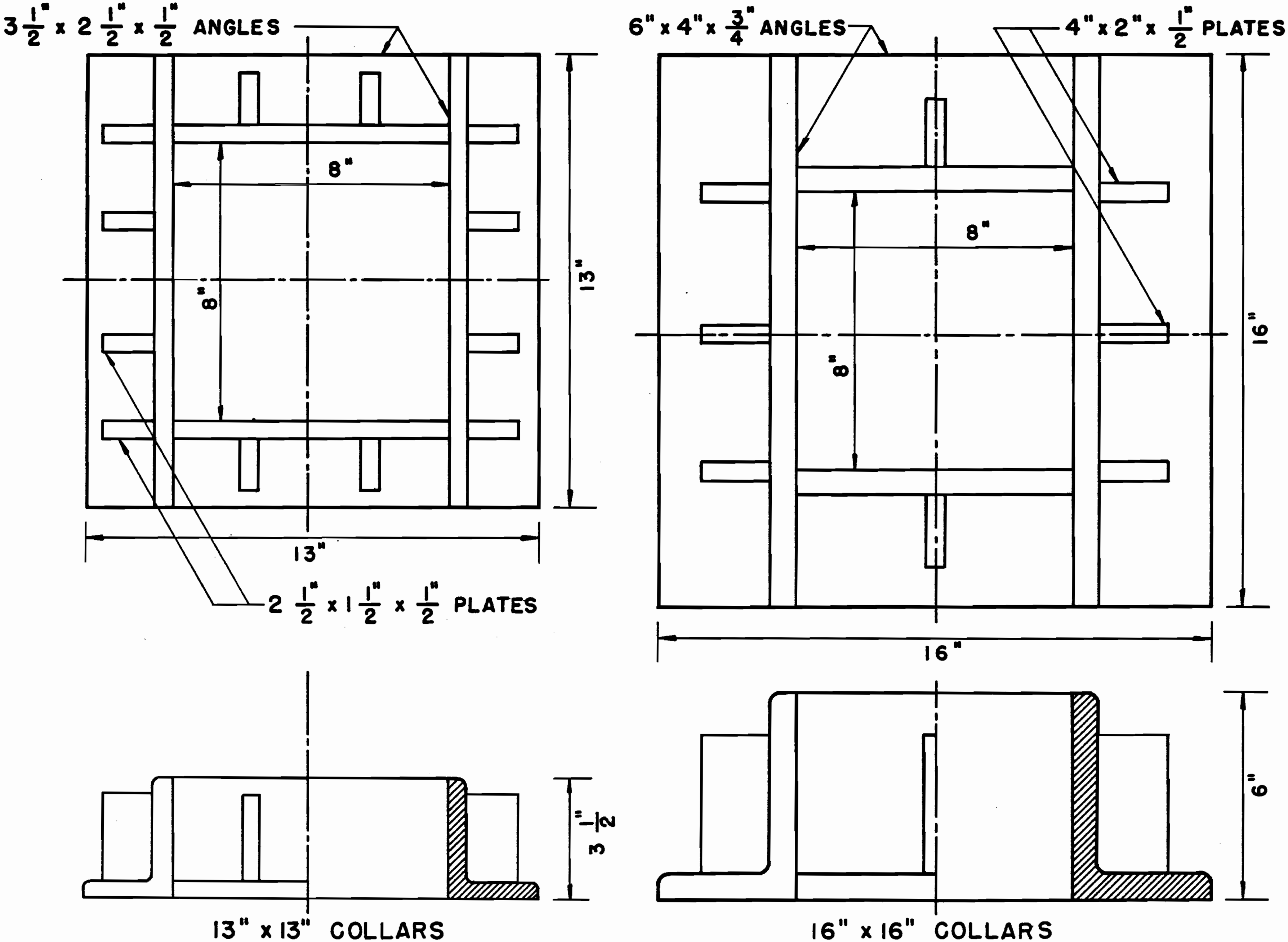


FIG. 10 LIFT SLAB COLLARS

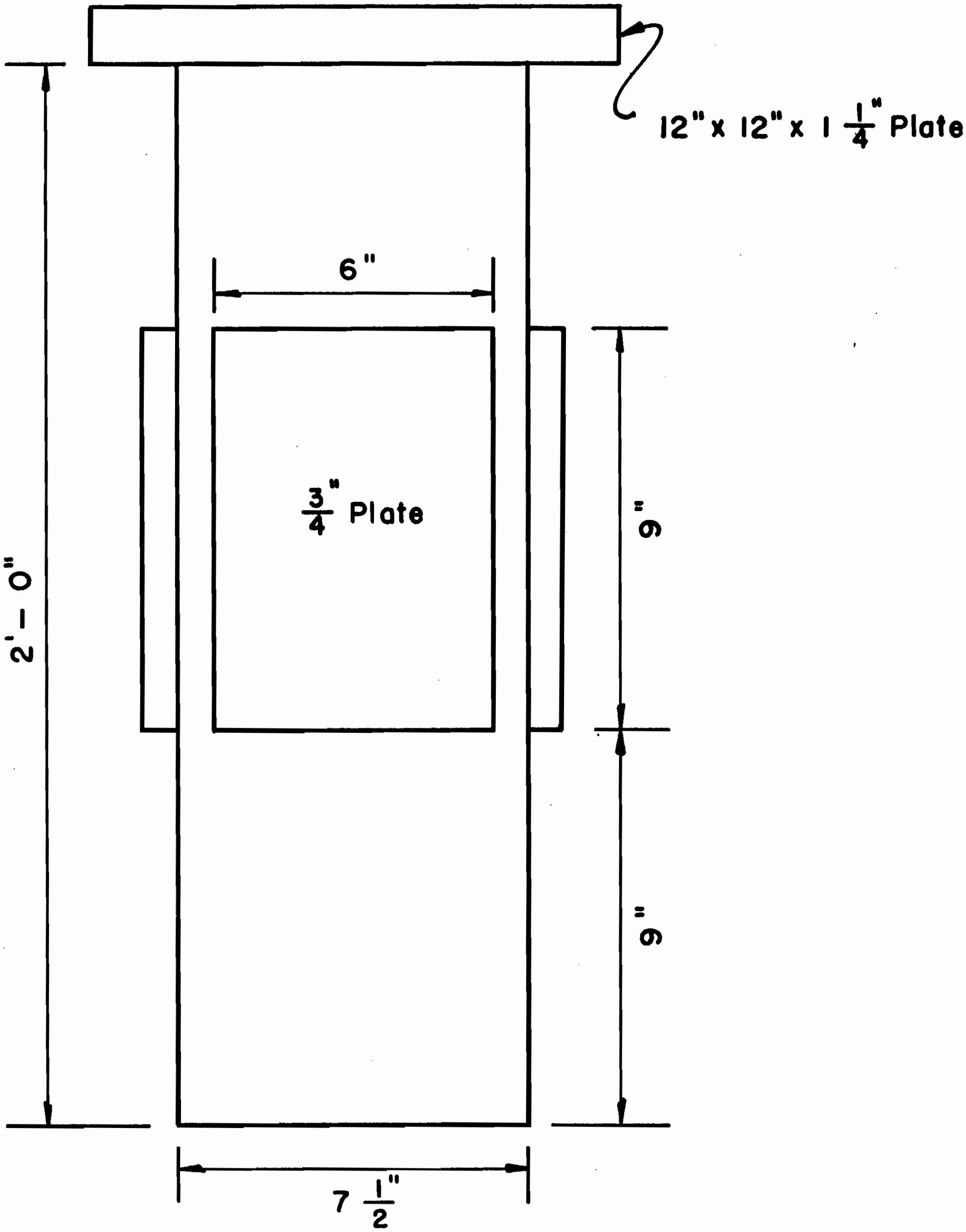


FIG. 11 LOADING COLUMN

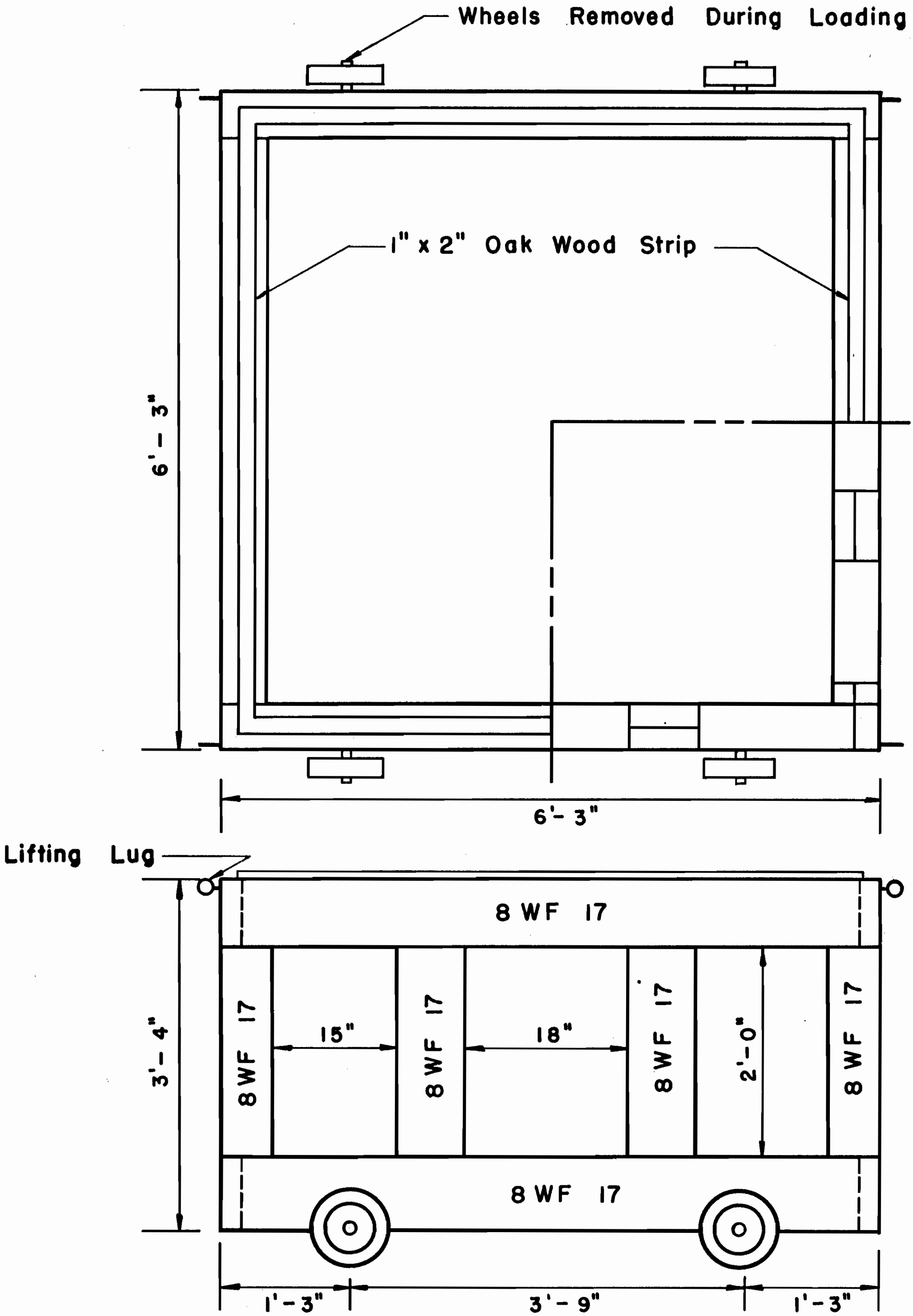
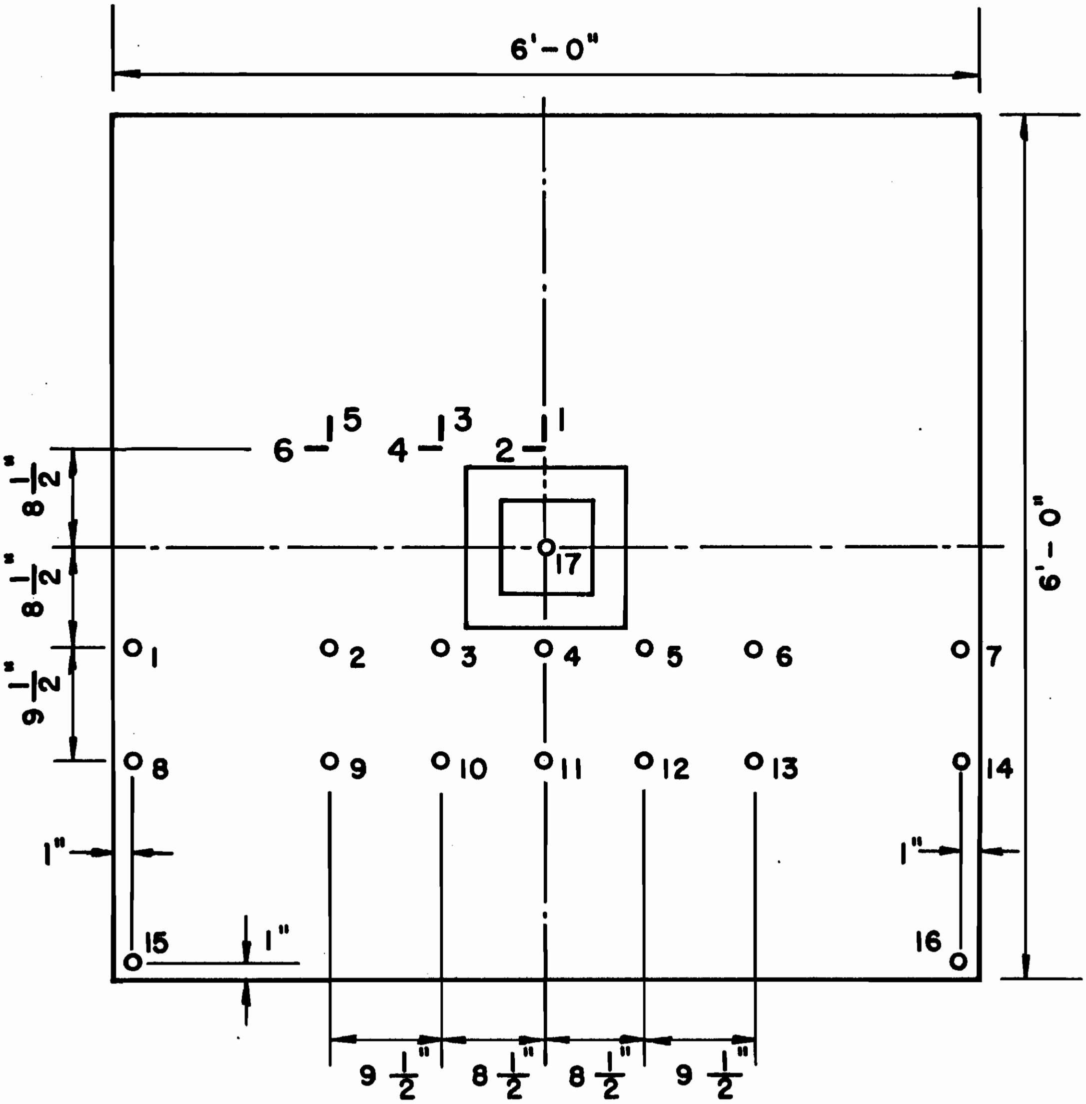


FIG. 12 SLAB SUPPORTING FRAME



— SR-4 Type A-1 Gages
o 0.001" Dial Gages

Note: SR-4 Gages #7 – #12 Are Located On The Bottom Of The Slab Directly Undeneath #1 – #6 Respectively.

FIG. 13 LOCATION OF SR-4 GAGES AND DEFLECTION GAGES

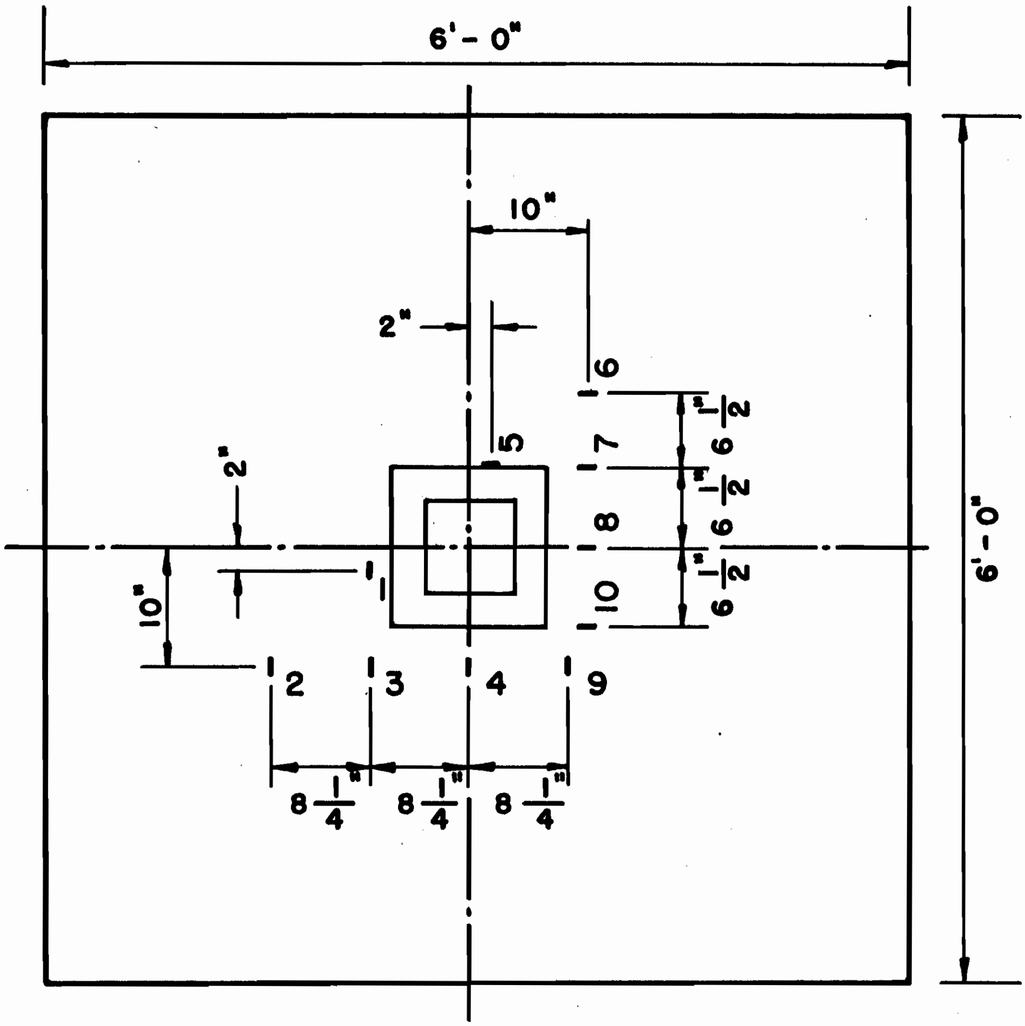
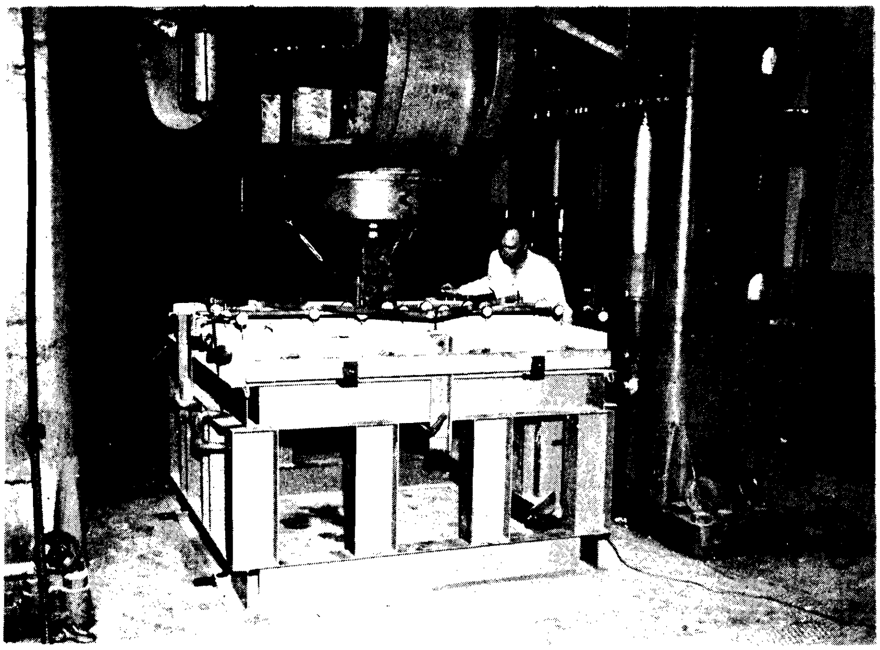
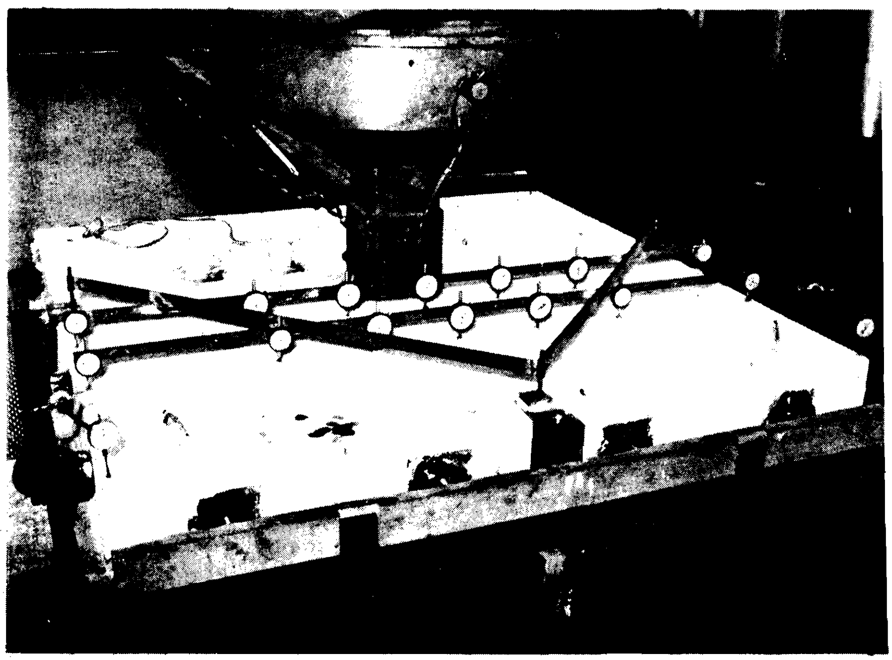


FIG. 14 LOCATION OF SR-4 GAGES ON INTERMEDIATE GRADE STEEL BARS (S-1 AND S-2)

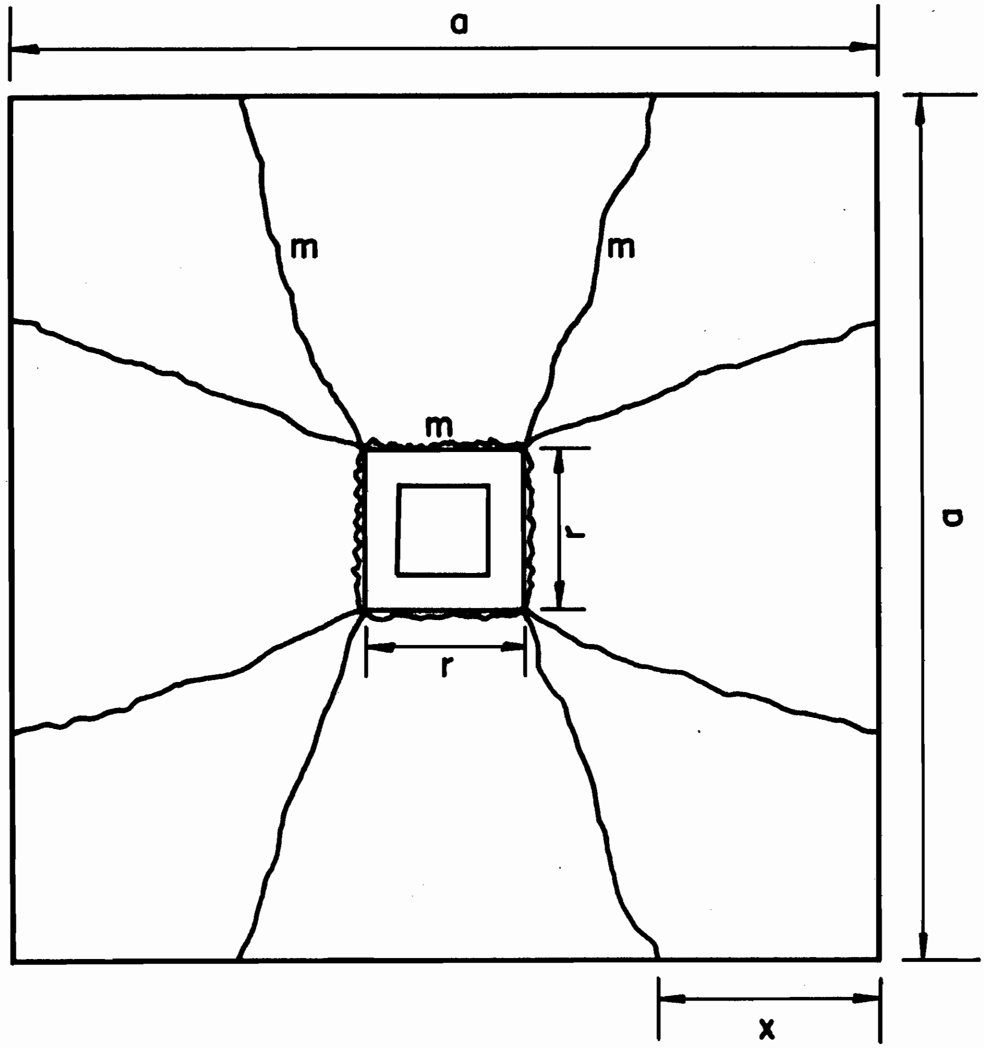


TEST ASSEMBLY



INSTRUMENTATION

FIG. 15 PHOTOGRAPHS OF TEST ASSEMBLY AND INSTRUMENTATION



$$x = \left(1 - \tfrac{1}{2}\sqrt{2}\right)(a - r)$$

Note: Slab Simply Supported Along Edges; Corners Are Free To Lift.

FIG. 16 THEORETICAL YIELD LINE PATTERN

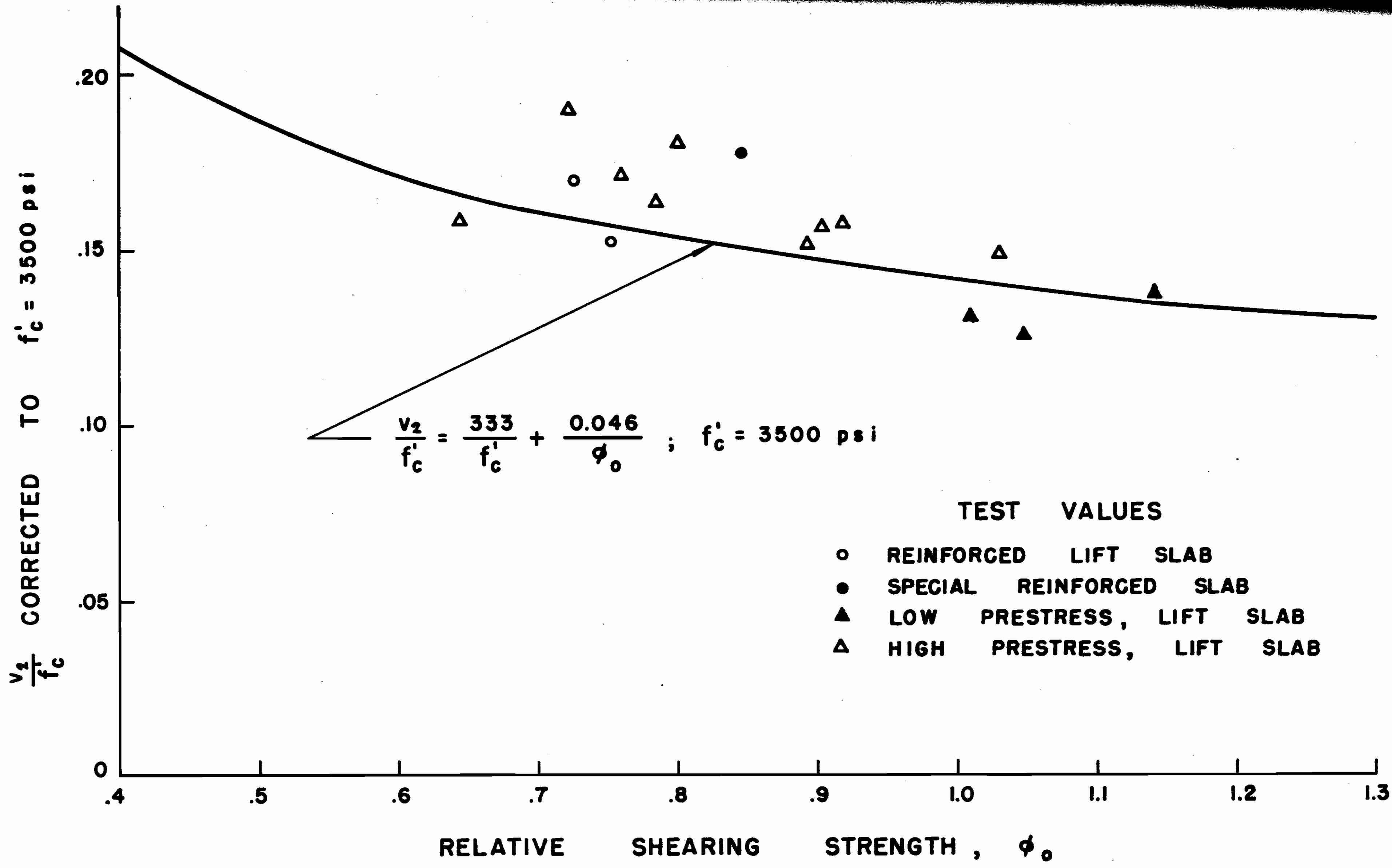


FIG. 17 INFLUENCE OF SHEAR-FLEXURAL STRENGTH RATIO USING EQUATION (3)

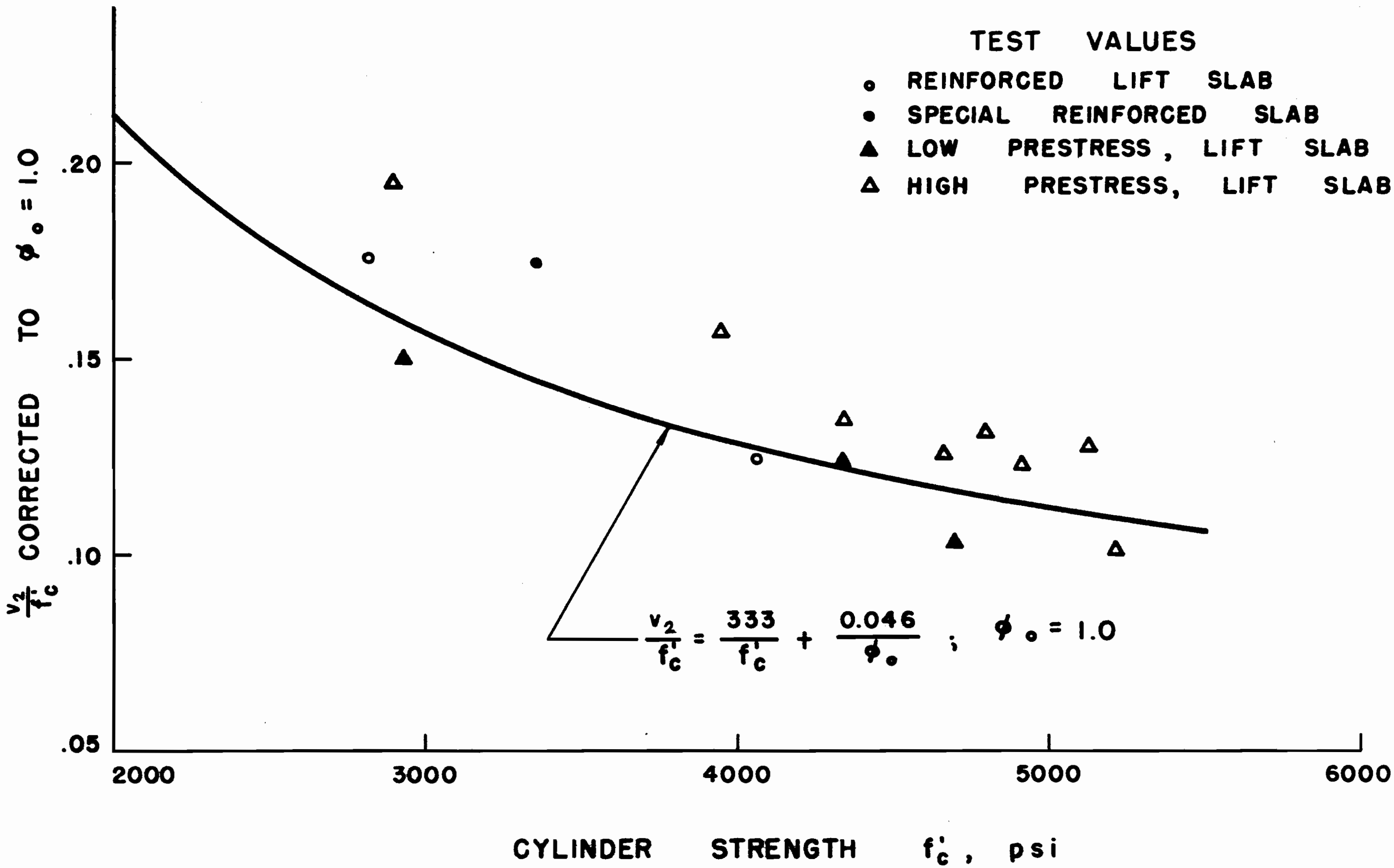


FIG. 18 INFLUENCE OF CYLINDER STRENGTH USING EQUATION (3)

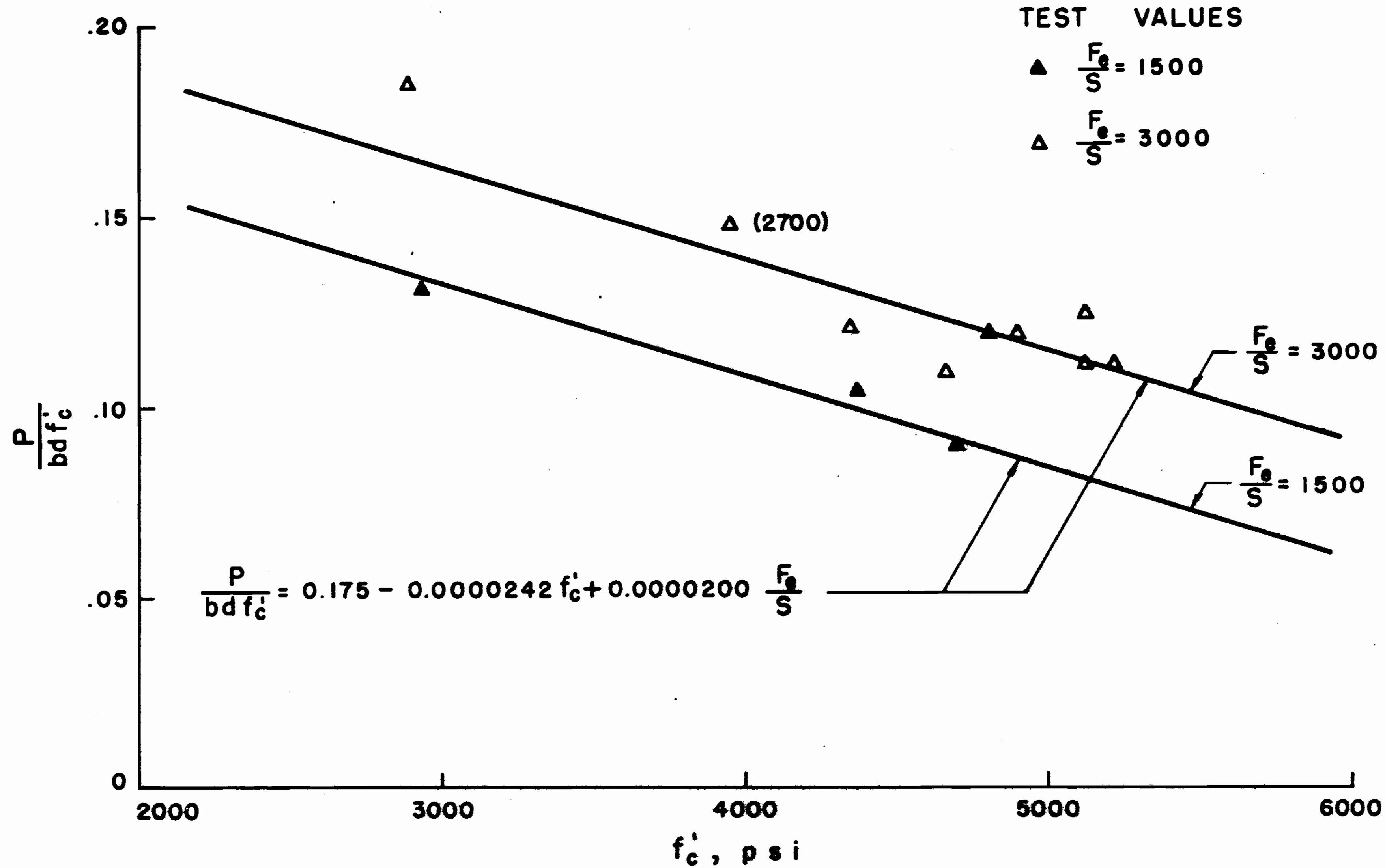


FIG. 19 TEST VALUES COMPARED WITH EQUATION (8)

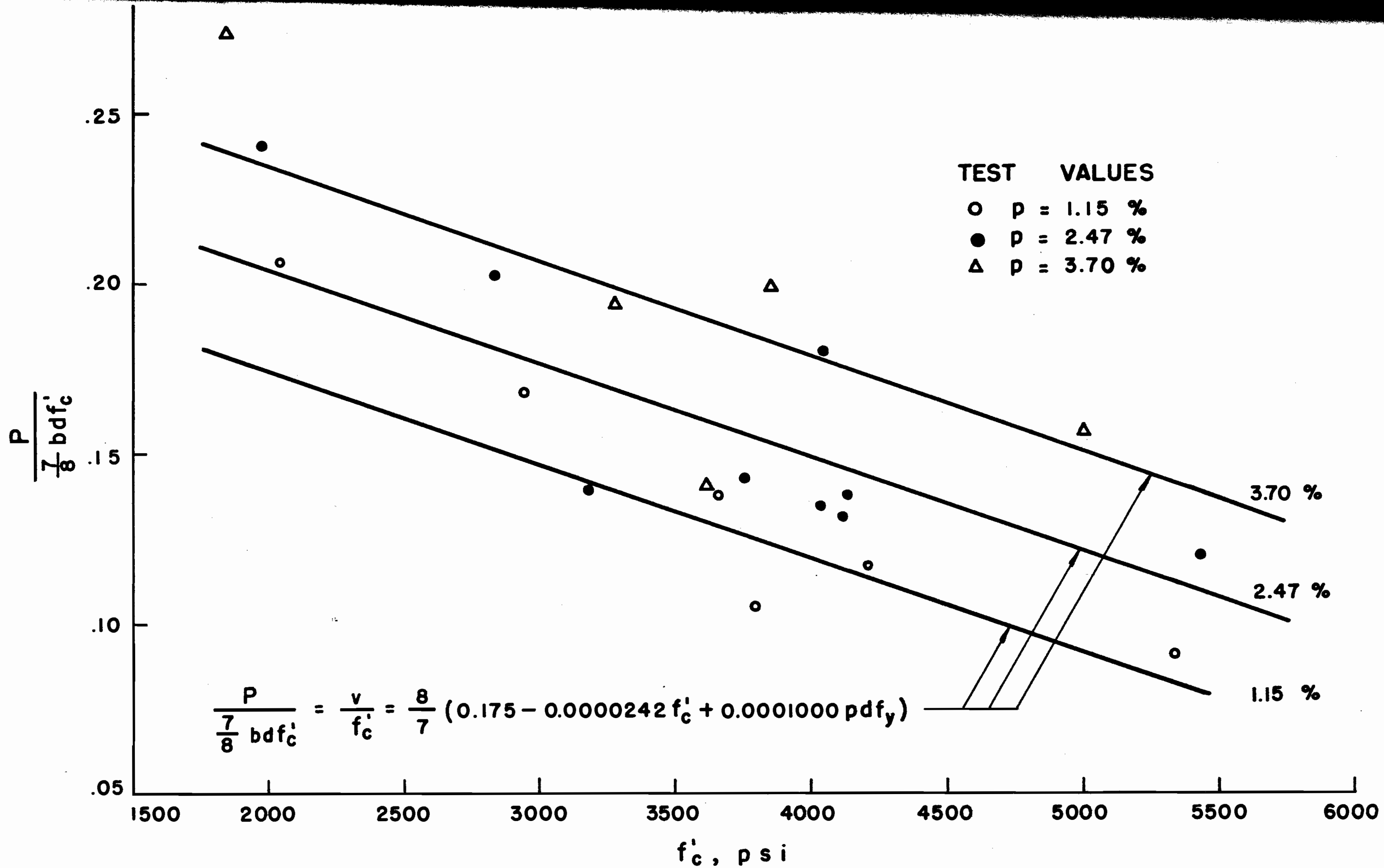
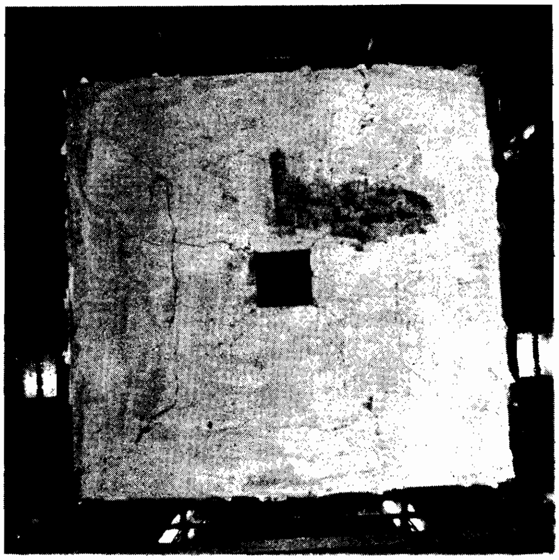
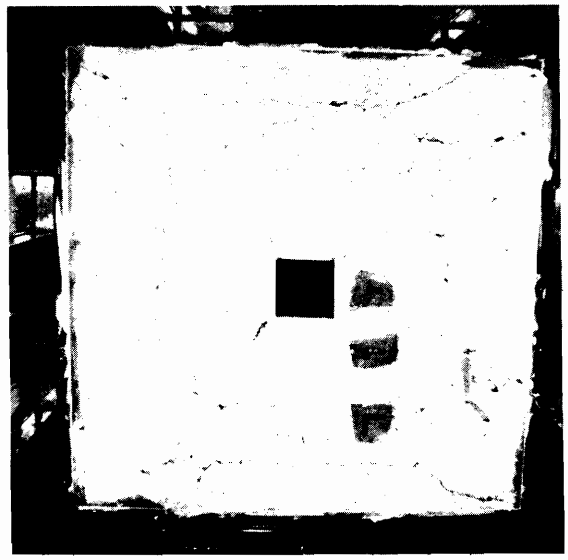


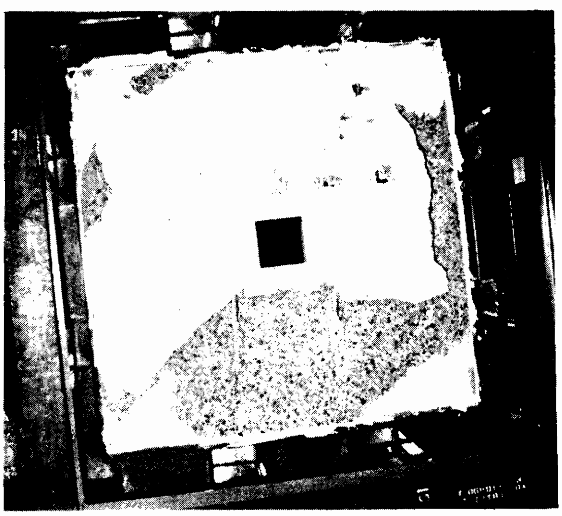
FIG. 20 TEST RESULTS OF ELSTNER AND HOGNESTAD SLABS COMPARED WITH EQUATION (8)



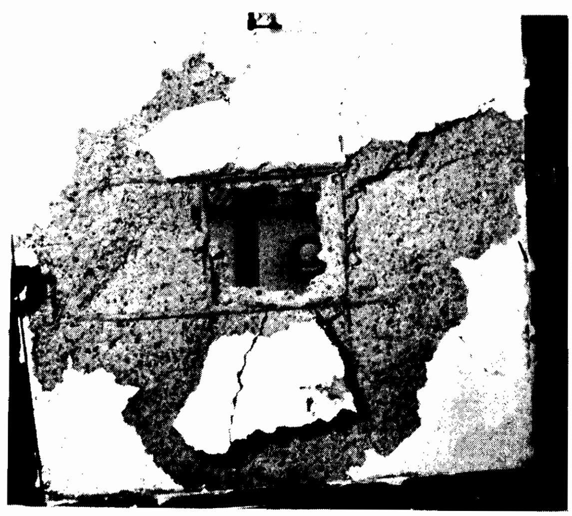
S - 1



S - 2

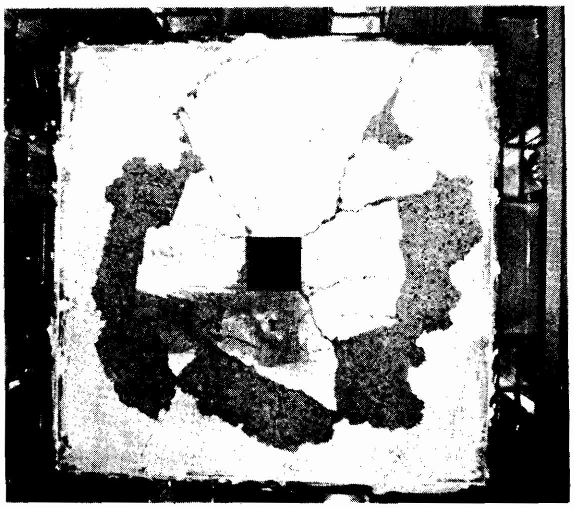


S - 4

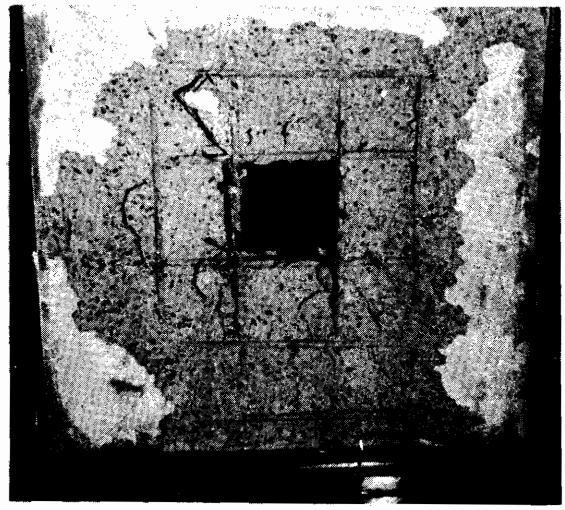


S - 5

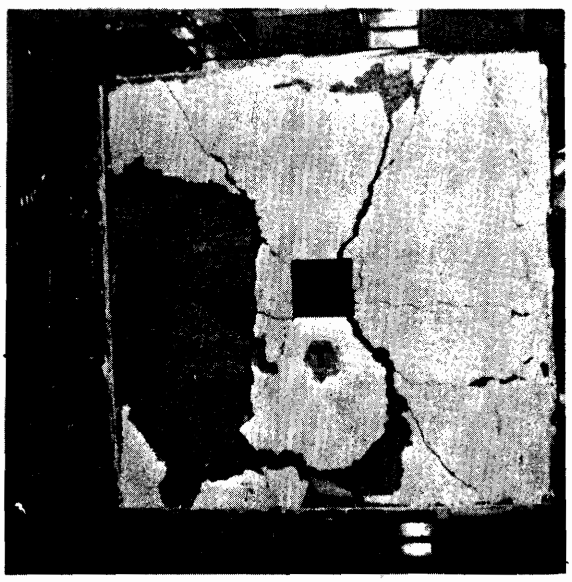
FIG. 21 BOTTOM VIEW OF SLABS AFTER FAILURE



S - 6



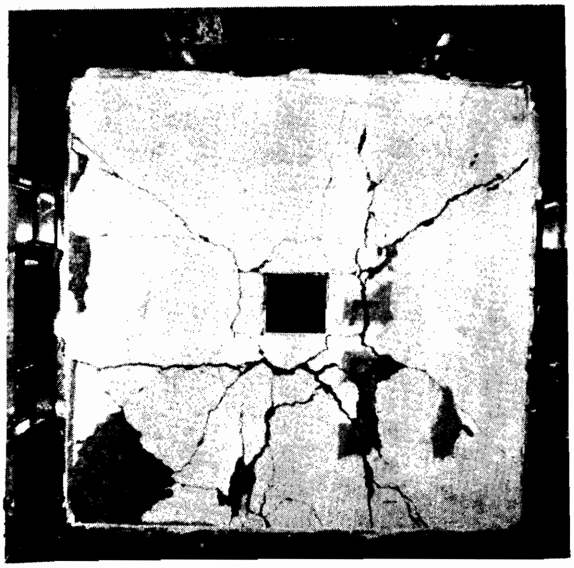
S - 7



S - 8

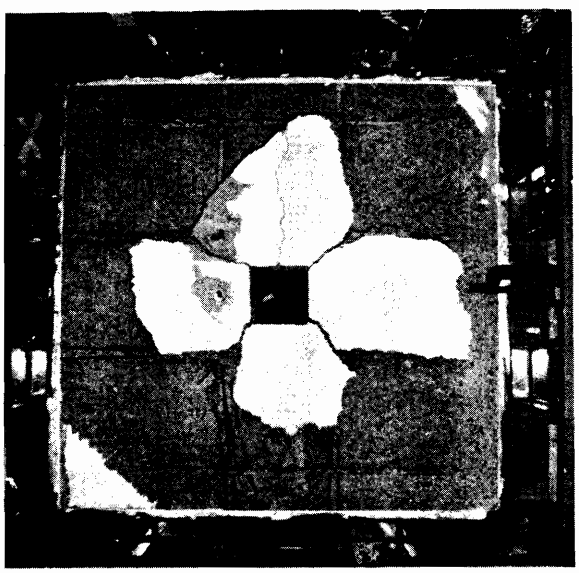
S - 9

FIG. 22 BOTTOM VIEW OF SLABS AFTER FAILURE

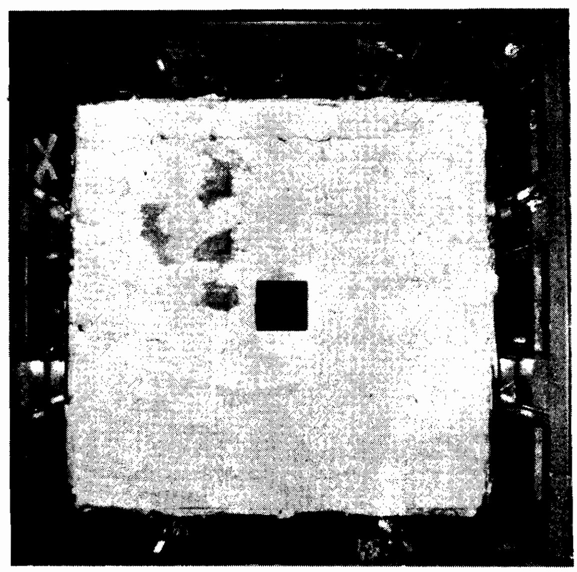


S-10

S-11

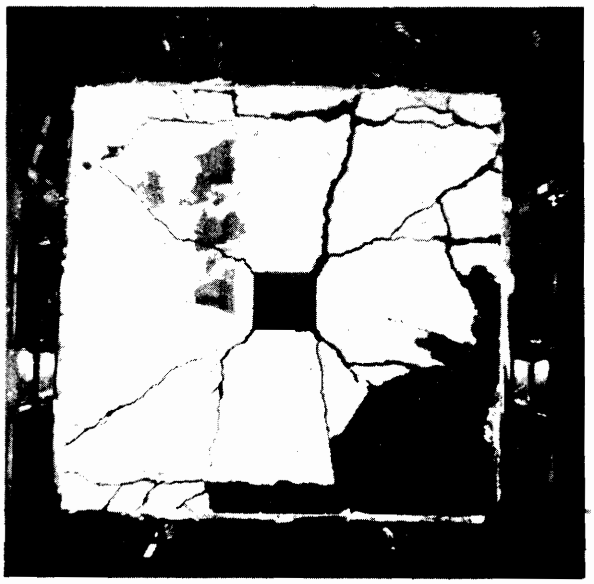


S-12

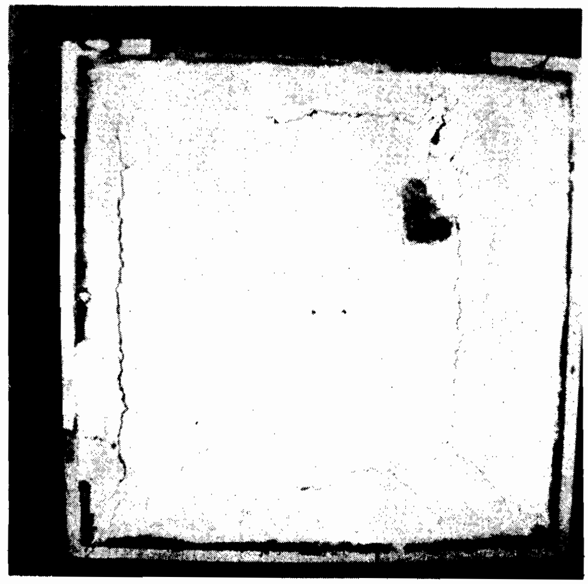


S-13

FIG. 23 BOTTOM VIEW OF SLABS AFTER FAILURE



S - 14



S - 16

FIG. 24 BOTTOM VIEW OF SLABS AFTER FAILURE

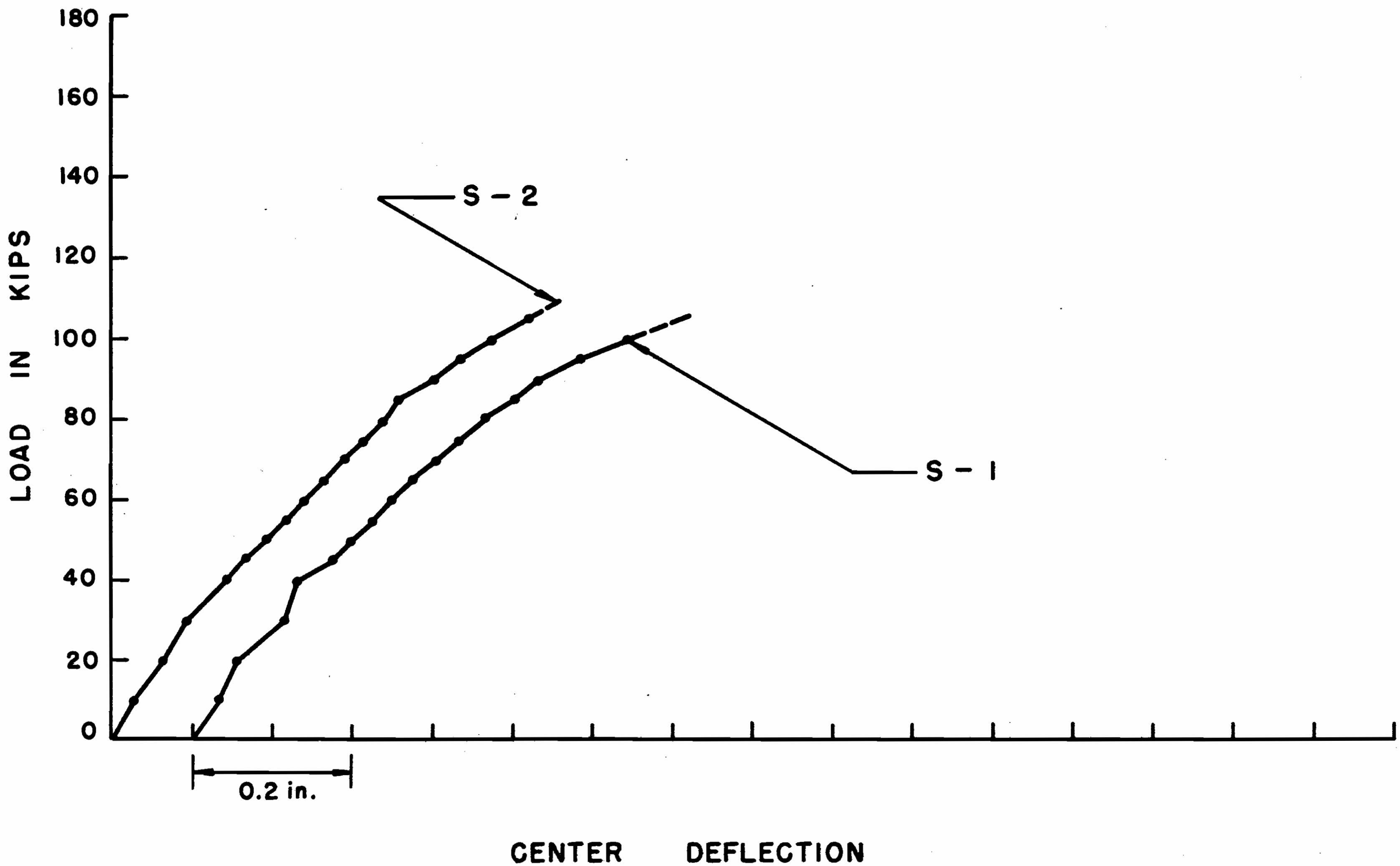


FIG. 25 LOAD-DEFLECTION CURVES FOR GROUP I SLABS

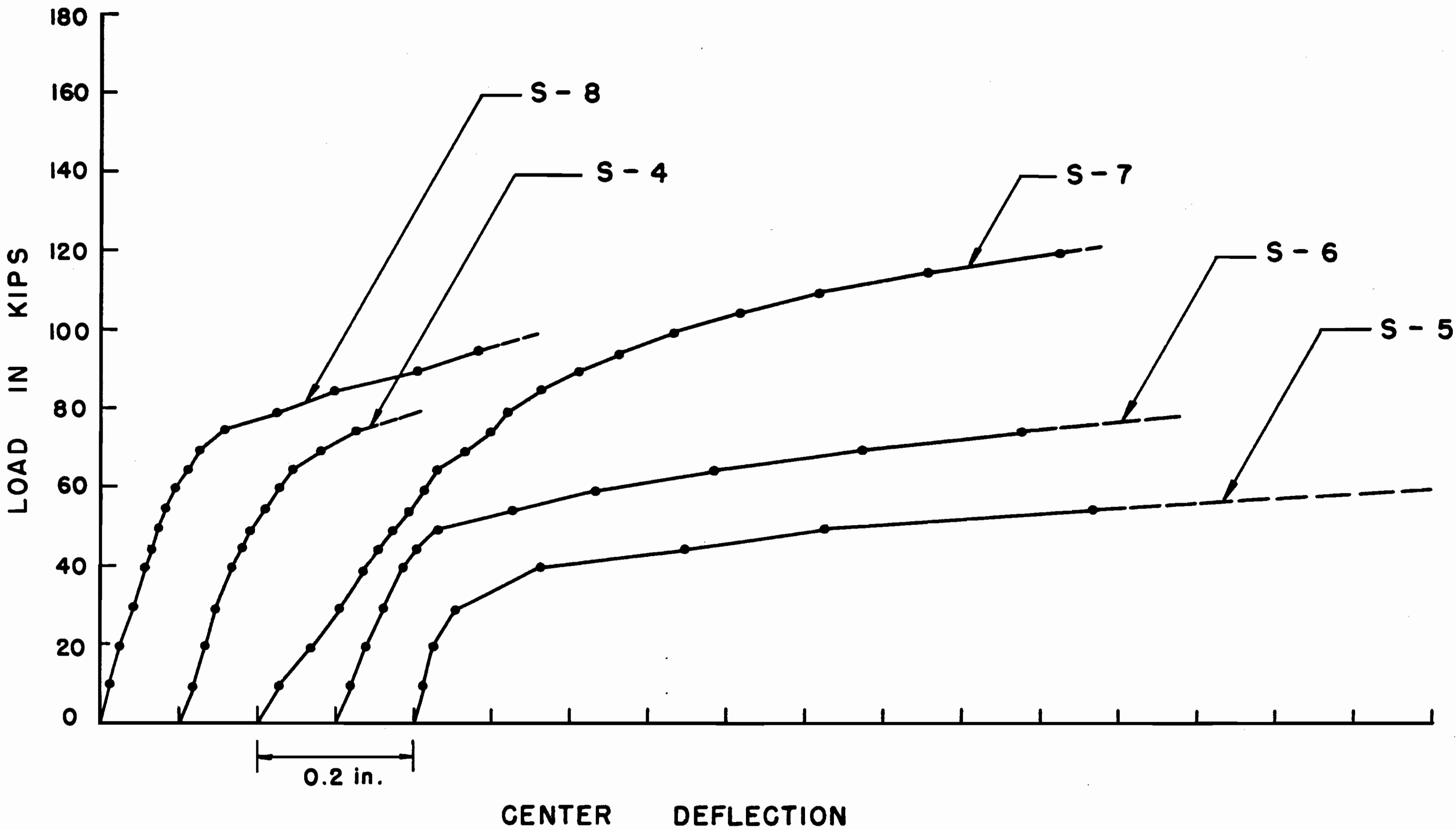


FIG. 26 LOAD-DEFLECTION CURVES FOR GROUP II SLABS

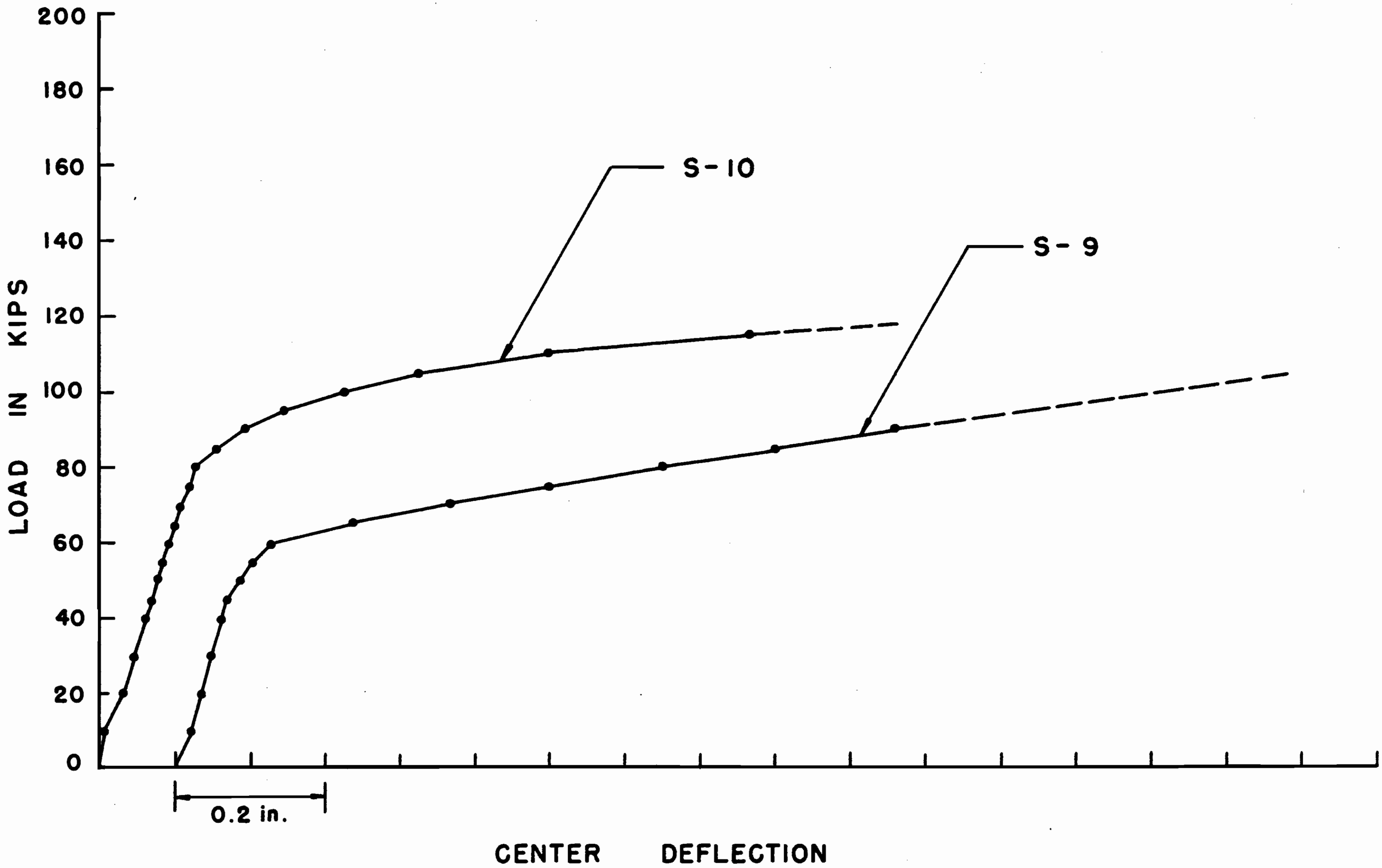


FIG. 27 LOAD-DEFLECTION CURVES FOR GROUP III SLABS

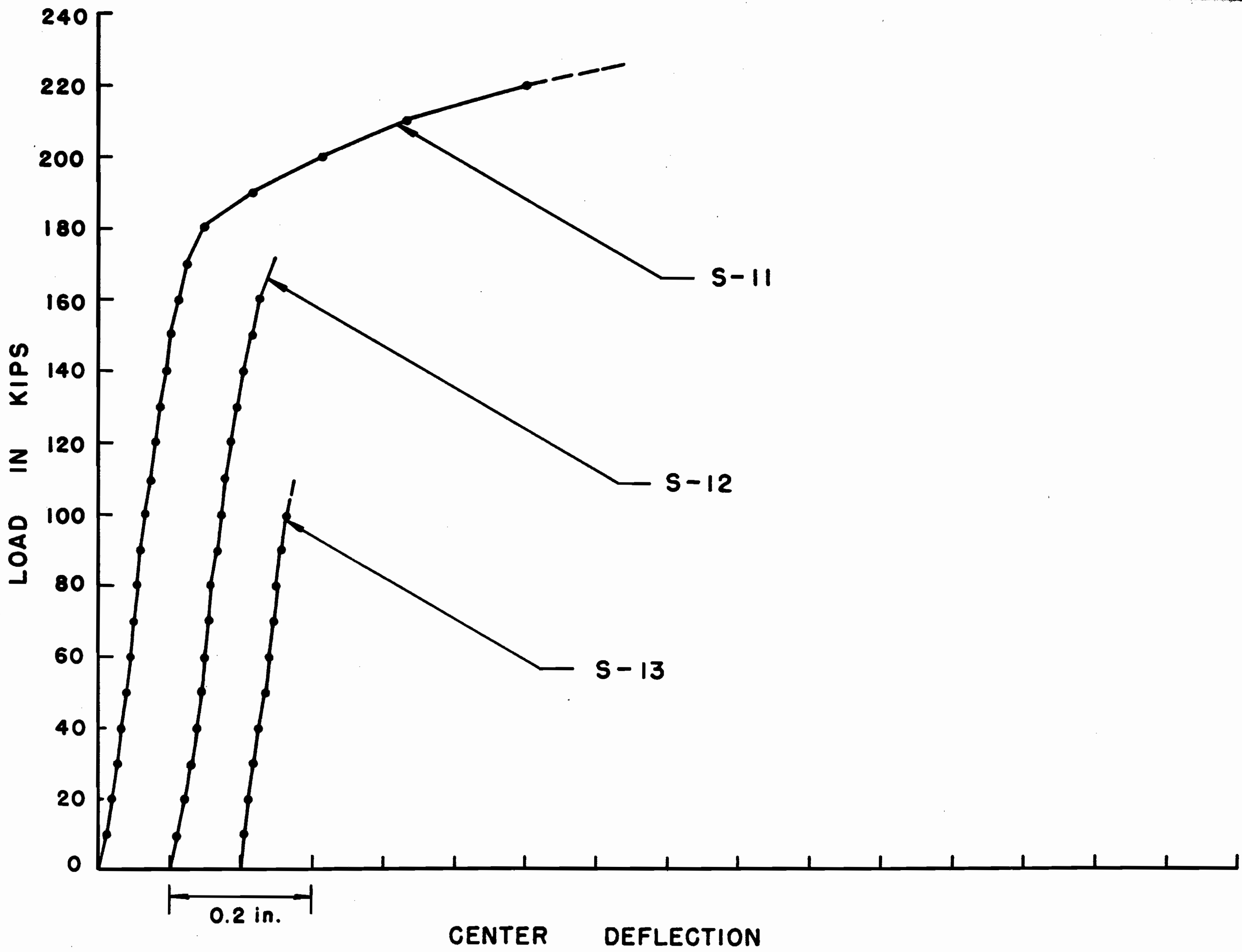


FIG. 28 LOAD-DEFLECTION CURVES FOR GROUP IV SLABS

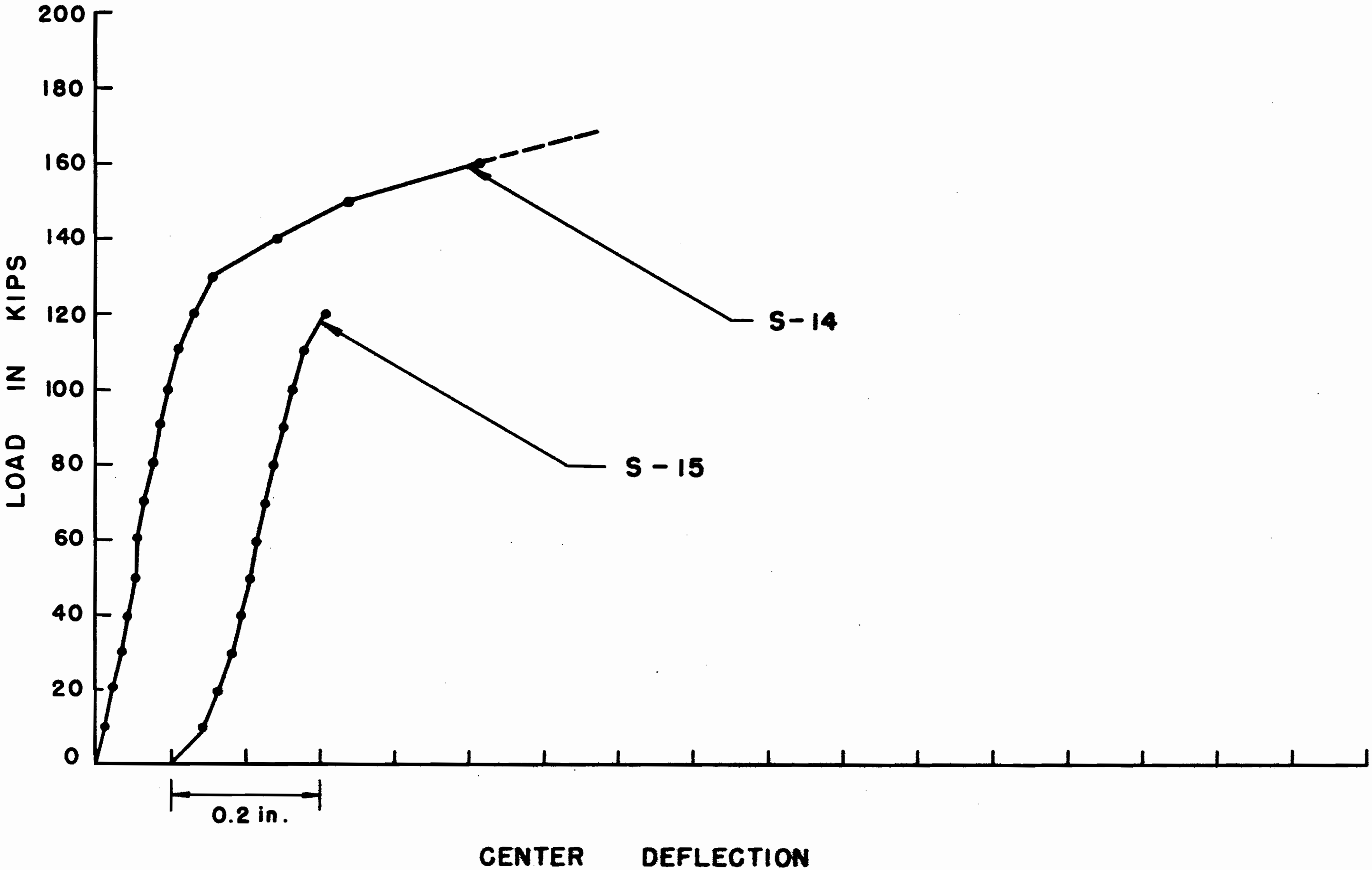


FIG. 29 LOAD-DEFLECTION CURVES FOR GROUP V SLABS

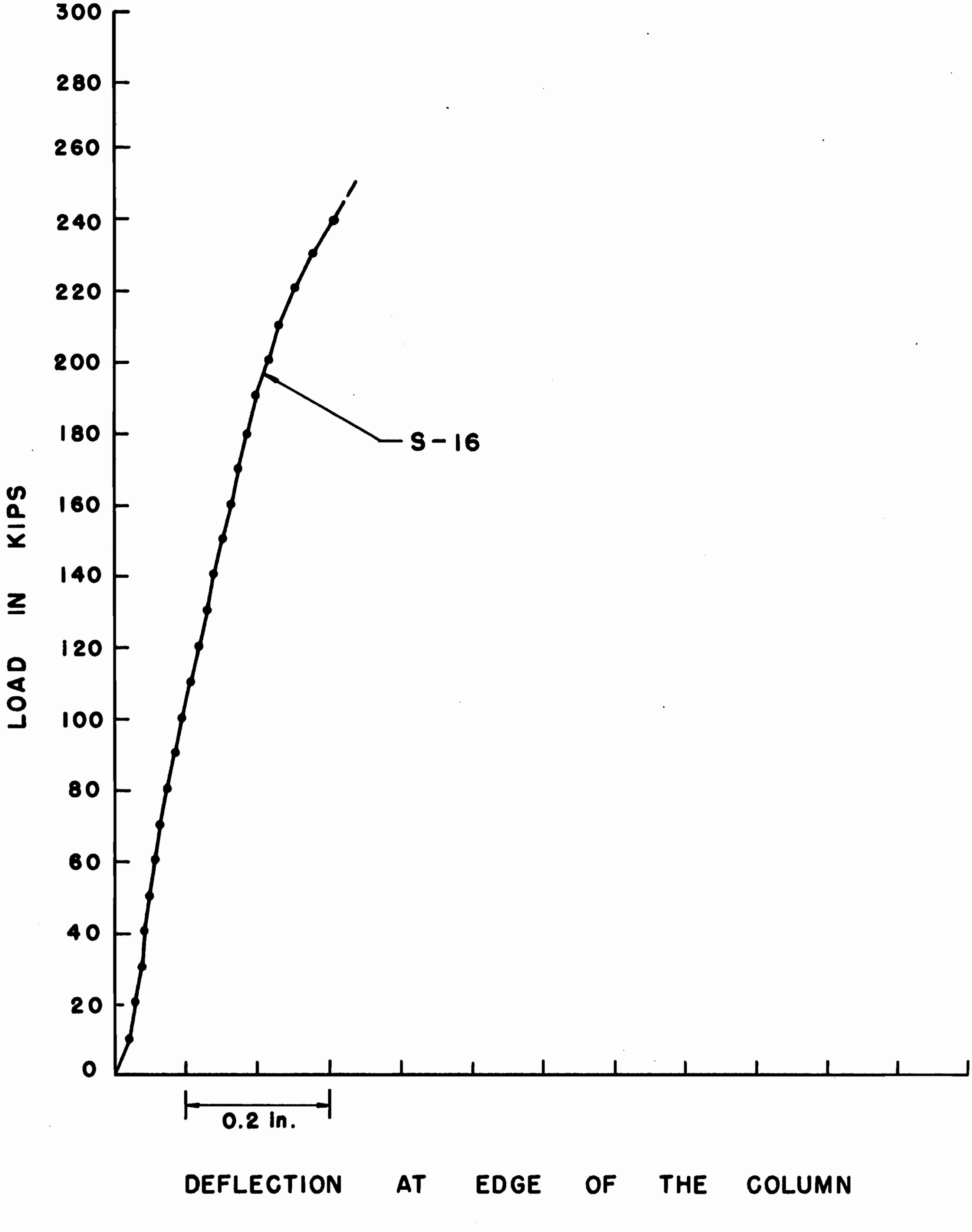


FIG. 30 LOAD-DEFLECTION CURVE FOR GROUP VI SLAB

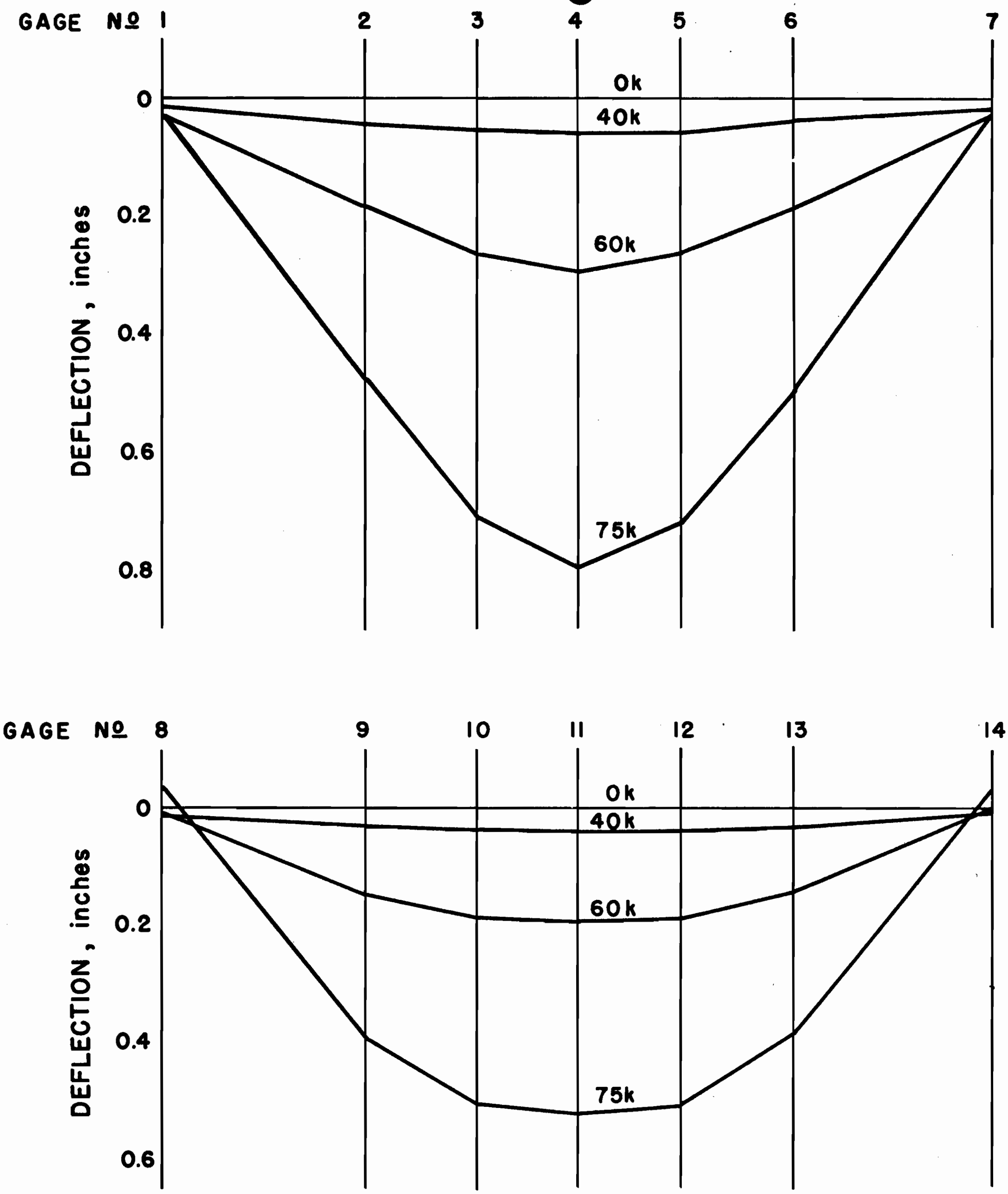


FIG. 31 TYPICAL LOAD DEFLECTION PROFILES

(S-6)