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# NATIONAL BUREAU OF STANDARDS REPORT

4396

TESTS OF PRESTRESSED CELLULAR SLABS
(Slab Nos. 1, 2, 3, and 4)

by

M. Chi and D. Watstein

Report to

Bureau of Yards and Docks
Department of the Navy

NBS

U. S. DEPARTMENT OF COMMERCE
NATIONAL BUREAU OF STANDARDS

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Abstract

Four 5- by 5-ft prestressed cellular slabs were tested under concentrated loads. Deflections and strains were measured and were compared with the computed values. Maximum load carrying capacities and crack patterns were recorded. Data obtained thus far indicated that elastic theory for solid slabs is applicable to slabs of this type. Welded wire reinforcement in the blocks and good contact between blocks such as would be provided by grouted joints, were essential to insure a high load carrying capacity of such slabs. The slabs tested on a relatively short span and carrying a moderate prestress of 1000 psi failed by "punching" shear under a centrally applied load.

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## 1. INTRODUCTION

At the request of the Bureau of Yards and Docks, the Structural Engineering Section initiated a study of the properties of slabs composed of 6- by 6- by 6-in. cellular concrete blocks arranged in a checker-board fashion and prestressed in two directions. The current report presents the results obtained in tests of four 5- by 5-ft slabs made of cells procured by Preload Corporation while under contract with the Bureau of Yards and Docks.

The first two slabs were fabricated with cells assembled with ungrouted joints. It became apparent upon assembling the ungrouted slabs that the dimensional tolerances of these cells were such that close fitting and tightly closed joints could not be obtained. Accordingly, it was decided to grout the joints in subsequent test specimens in order to secure uniformly intimate contact between all bearing surfaces and the second pair of slabs described in this report were fabricated in this manner.

## 2. DESCRIPTION OF TEST SPECIMENS

### 2.1 Cells

The cells were hollow 6 in. cubes having an opening 4.5- by 5-in. in cross section. There was a 1- by 2-in. elliptical hole on each web through which the prestressing unit could pass. The principal dimensions of the cellular unit are shown in figure 1.

The cells utilized in the tests of these four slabs were furnished to the National Bureau of Standards by Preload Corporation which procured these units from a commercial concrete products plant. For these reasons, the proportions of the concrete used in the cells are not known.

The outside dimensions of the cells selected at random were found to vary as much as 1/8 in. from the specified dimensions. The reinforcement in cells consisted of a single layer of 1- by 1-in. galvanized welded wire fabric of 15/15 gage.

A 3/4- by 2- by 6-in. strip of concrete removed from a cell was tested to determine the modulus of elasticity and compressive strength of concrete. The results of sonic and static tests are given in the following table:

| Method of testing | Modulus of elasticity | Compressive strength | Poisson's ratio |
|---|---|---|---|
| | $10^6$ psi | psi | |
| Static | 2.91 | 4710 | 0.12 |
| Sonic | 2.57 | -- | -- |

## 2.2 Prestressing steel

The prestressing units in the test slabs were 5/8 in. "Stressteel" bars. The anchorage for these bars consisted of hexagonal nuts 1 1/4 in. long which bore on 5 3/4- by 5 3/4- by 3/4-in. anchor plates.

The tensile strength of unthreaded Stressteel bars was found to be 163,000 psi, whereas threaded bars supported by fully tightened nuts developed a tensile strength of 152,000 psi. The yield strength of the steel, determined by the "offset" method (offset = 0.2%) was 142,000 psi. The stress-strain characteristic was a straight line up to 65,000 psi, giving a Young's modulus of $30 \times 10^6$ psi; the secant modulus at 100,000 psi was $28.2 \times 10^6$ psi. The reduction in area at point of fracture was 35 percent.

## 2.3 Description of prestressed slabs

Each slab consisting of 100 cells was nominally 5- by 5-ft in plan and 6 in. thick. The cells were arranged in such a fashion that the axis of the cell through the open ends was perpendicular to the axis of each adjacent cell. This arrangement resulted in equal strength in both directions of the slab. The holes in the cell webs were also arranged in such a manner as to enable the prestressing tendons to be staggered with respect to the midplane of the slab. Thus, the resultant prestressing force produced axial prestressing in both directions of the slab.

The dimensions of the slabs with ungrouted joints were approximately 5- by 5-ft. The first slab with ungrouted joints designated as Slab No. 1 was made of cells wich appeared to be sound in texture and free from shrinkage cracks. It was quite evident that the cells had very poor contact with one another. Some of the joint openings were as wide as 1/4 inch.

The second slab with ungrouted joints, designated as Slab No. 2, was made of cells which were selected with greater care as to their dimensions and shape. However, on account of limited stock of units, some of the blocks were honeycombed and had shrinkage cracks. Nevertheless the assembled slab presented a better appearance and its joint openings were much narrower than was the case in Slab No. 1.

The first grouted joint slab, designated as Slab No. 3, was constructed in two stages. First, the cells were assembled into 5-ft beams with about 1/8 in. mortar joints. The beams were then assembled together with 1/8 in. mortar joints between them. The finished slab was approximately 61.5 by 62 inches. The shorter dimension was the length of the beams. The slab was a true plane before it was prestressed. The second slab with grouted joints, designated as Slab No. 4, was made of cells which were inferior in concrete quality compared with those in previous slabs. Its general appearance, however, was similar to Slab No. 3.

After prestressing, the slabs became slightly warped with one corner curled up.

## 2.4 Prestressing procedure

Experience in prestressing operations indicates that it is neither necessary nor practical to apply prestress simultaneously to all bars. Large amounts of prestress may be safely applied if the prestressing force is applied in small, equal increments to all tendons.

In the current tests, a portion of the prestress was applied by tightening the anchorage nuts with a wrench. Approximately half of the specified prestress of 1000 psi was applied with a wrench and the remainder was applied with a hydraulic jack. In order to maintain the stress in the slab as uniform as possible, the following sequence of prestressing with a jack was adopted:

1. The third bars from the edges of slab.
2. Edge bars.
3. Center bars.
4. The rest of bars symmetrically.

The tensioning force was applied to the prestressing tendons through an adaptor having suitable threaded connections. Two SR-4 strain gages were mounted longitudinally and two transversely on the adaptor bar. Each pair was connected in parallel and the two pairs served as measuring and compensating gages, respectively. The calibration curve for the adaptor is shown in figure 2.

# 3. TESTING PROCEDURE

## 3.1 Test set-up

All slabs were tested in a 600,000 lb capacity hydraulic testing machine to failure. The specimen was simply supported on 3/4 in. pipe rollers which rested on suitable frames. Two- by 6-in. timbers were used for the frame in testing Slab No. 1. This frame was found to be lacking in rigidity and accordingly it was replaced by one consisting of 6 in. I-beams in tests of Slabs Nos. 2 and 3. With this arrangement, however, it was impossible to observe the cracking pattern in the bottom of specimens during the tests. It was decided, therefore, to test Slab No. 4 elevated about 5 ft above the machine platen. The frame for supporting the slab was made of 6- by 12-in. reinforced concrete members which, in turn, were supported by seven 6-in. concrete cylindrical pillars about 4 ft high. This permitted not only visual observation of the bottom face of the specimen during the test, but also the installation of Tuckerman optical gages to measure tensile strains and widths of cracks.

## 3.2 Instrumentation

In the test of Slab No. 1 only the deflections were measured. During this test 0.0001-in. micrometer dial gages were used. However, in the tests of Slabs Nos. 2, 3, and 4, 0.001-in. dial gages were used and were found to be sufficiently accurate. The location of the dial gages is shown in figure 3.

The variation of strain in prestressing bars was measured by SR-4 strain gages attached to the bars. Four bars in each direction in Slabs Nos. 2, 3, and 4, were each equipped with a pair of A-11 gages connected in series. They were designated as bar gages Nos. 1 through 8 and their locations are shown in figure 3. All bars with gages were calibrated in a testing machine.

Strain in concrete surfaces was measured by A-11 gages. In Slabs Nos. 3 and 4, four such gages were mounted on top and four on bottom surface of each slab. The locations of these gages are also shown in figure 3.

Four Tuckerman optical gages were installed on Slab No.4 and their locations are shown in figure 3. They were intended to measure the width of cracks. Unfortunately, cracks occurred elsewhere and these gages were used only to check the tensile strains recorded by SR-4 gages.

### 3.3 Testing procedure

All slabs were first flexed by applying a load of 5000 lb. This load was then removed, the slab was allowed to recover for several minutes and a set of readings of all gages for zero load was recorded. The load was applied to the slab through a 6- by 12- by 12-in. concrete loading block located at the center of the specimen. The load was generally applied in increments of 5000 lb, but sometimes the increments were reduced to 2500 lb when an extra set of readings appeared to be advisable. As the maximum load was approached, the indicated strains and deflections increased noticeably. As soon as these signs of impending failure were observed, readings of the gages were discontinued and the load was increased until the maximum value was reached. This value was recorded as the load carrying capacity of the slab.

## 4. TEST DATA

### 4.1 Deflections of slabs

All deflection measurements were made with reference to the lower platen of the testing machine. The four dial gages placed directly over the edges of the slab indicated the settlement of the supports. The observed deflections of the slabs were corrected for the settlement of the supports by assuming that the edges of the slab remained straight during the test and that, consequently, the settlement of the supports varied linearly between these four points.

The relation between center deflections and applied load is shown in figure 4. It is noted that Slab No. 2 had a much larger deflection than Slabs Nos. 3 and 4. This indicates that imperfect contact between cells in the ungrouted slab produced extra deflection due to the adjustment of the contact of cells.

Figures 4A and 4B show the deflection profiles of Slabs Nos. 3 and 4 obtained by averaging the deflections along two rows of dial gages disposed symmetrically and adjacent to the loading block. The location of the dial gages is shown in figure 3. The gages at the ends of these two rows served to determine the datum plane of the supported edges of the slab.

### 4.2 Variation of prestressing force during test

After the prestress is applied to the slab it may diminish slowly due to the following causes: (1) readjustment of contact of cells, (2) relaxation of steel, (3) creep of concrete, and (4) shrinkage of concrete. In view of the fact that the cells were about a yearold, concrete would be expected to shrink only a negligible amount. Since Slabs Nos. 3 and 4 had grouted joints, they were monolithic structures and no readjustment of cells occurred. The loss indicated by the decrease in strain of bars in these slabs must be attributed to causes (2) and (3). The loss was rather small because of the low prestress; this is illustrated by data obtained for Slab No. 3 shown in table 1. In the case of Slabs Nos. 1 and 2, however, the loss of prestress due to readjustment of the cells was observed from the time when the anchorage nuts were tightened by wrench. When the full prestress was applied to Slabs Nos. 1 and 2, most of the readjustments of the cells had already taken place but the process continued for a few days before it tapered off. Since SR-4 gages were installed on some prestressing bars in Slab No. 2 these losses were measurable. It was noted that the loss of prestress in Slab No. 2 due to its being moved from the working platform to the test frame constituted 25 percent of total strain due to prestressing.

Since the prestressing bars were placed either at the neutral plane or 0.5 in. above or below it, it was expected that the application of transverse load to the slab would have little or no effect on the strain in the bars. The observed increments of strain in the bars in Slabs Nos. 3 and 4 were all under 4 percent of the initial prestress. The variation of prestress is given in table 2.

In Slab No. 2 the maximum loss of strain during the test was $96 \times 10^{-6}$ in./in., which was more than four times as much as that for corresponding load in Slabs Nos. 3 and 4. The larger loss of prestress in Slab No. 2 can be attributed to the readjustment of the cells. This excessive loss of prestress probably had a direct bearing on the extraordinarily large deflection mentioned in 4.1 and low load carrying capacity as will be described in 4.5.

## 4.3 Concrete strain

Concrete strains due to prestressing were measured in Slab No. 3. The average strain corresponding to 1000 psi prestress was observed to be 313 x $10^{-6}$ in./in.

The variation of strain on the surface of the concrete with the applied load is illustrated in figures 5 and 6 for Slabs No. 3 and 4, respectively. During the test of Slab No.3, it was observed that the strain gages on the bottom face of the slab, that were connected to a common ground lead had a low resistance to ground. The tensile strain data obtained for Slab No. 3 were, therefore, discarded. All strain gages in Slab No. 4 had separate leads and the data obtained appeared reasonably good.

## 4.4 Width of cracks

Tuckerman gages were installed to measure the width of cracks on the bottom face of Slab No. 4 and they were located directly under the edges of the loading block where the maximum moment was expected to occur. Since the slab presumably failed due to punching shear, the crack occurred about a foot from the loading block. However, the Tuckerman gages served to check the tensile strains measured by SR-4 gages. The concrete strains measured by Tuckerman gages are also shown in figure 6.

## 4.5 Load carrying capacity

Slabs Nos. 1 and 2 which were assembled with ungrouted joints supported maximum loads of 11,800 and 16,250 lb, respectively. It will be recalled that Slab No. 1 contained units which made poor contact with one another and had joint openings as wide as 1/4 in. The units used in Slab No. 2 were selected with greater care as to their dimensions and shape and its joint openings were much narrower than in Slab No. 1.

Slabs Nos. 3 and 4 were grouted and were both fabricated in the same manner. During the test, Slab No. 3 was loaded to 35,000 lb at which point signs of approaching failure were indicated. The load was then removed, and readings after recovery were recorded. The slab was then loaded again and failed at a load of 30,500 lb. Slab No. 4 supported a maximum load of 34,250 lb.

## 4.6 Crack pattern

In Slab No. 1 the four cells under the loading block were pushed down about 3/4 in. The cells adjacent to them had crushed corners. This can be attributed to the failure of frictional forces between the cells to resist the shear. Figure 7 showing bottom view of slab, clearly illustrates this type of failure.

In Slab No. 2 the four cells directly under the loading block along with two rows of cells surrounding them were pushed down. There was some slipping between cells but the bottom surface formed essentially a convex surface. Cracks were irregular and did not form any definite pattern on the surface. The cells adjacent to the loading block and having a web parallel to its edge developed horizontal cracks near the top shell of the unit; for cells further away from the loading block, the horizontal cracks developed near the bottom shells. Webs of cells perpendicular to the edge of the loading block had diagonal cracks. Cells one foot or more away from the loading block appeared undamaged except for the presence of some horizontal hair cracks in the webs. The top surface of the slab remained intact but the loading block was forced into the slab about 1/4 inch. (See figures 8 and 9).

In Slabs Nos. 3 and 4 a circular crack approximately 3 ft in diameter appeared at the bottom surface. Figure 10 shows the crack pattern observed in Slab No. 4. As in Slabs Nos. 1 and 2, the loading block was pushed approximately 1/4 in. into the slab. The top shells of the cells adjacent to the loading block cracked slightly. There was a circular hair crack in the top surface of Slab No. 3 with a diameter of approximately 3 ft. (See figure 11). No similar crack was observed in Slab No. 4. On the other hand, a hair crack approximately 14 in. in diameter was observed in the bottom face of Slab No. 4, but not in Slab No. 3. Cracks in the webs of cells were essentially similar to those in Slab No. 2. Typical diagonal crack patterns in sections of a slab are shown in figures 12 and 13; diagonal cracks in individual units are shown in figure 14.

## 5. COMPARISON OF TEST DATA WITH THEORY

### 5.1 Theory

The deflection of a square plate under a partially distributed load is given in S. Timoshenko's "Theory of Plate and Shells," p. 153, as follows:

$$w = \frac{4qa^4}{D\pi^5} \sum_{m=1,3,5,\cdots}^{\infty} \frac{(-1)^{\frac{m-1}{2}}}{m^5} \sin\frac{m\pi a_1}{2a} \left\{ 1 - \frac{\cosh\frac{m\pi y}{a}}{\cosh\alpha_m} \right.$$

$$\left[ \cosh(\alpha_m - 2\gamma_m) + \gamma_m \sinh(\alpha_m - 2\gamma_m) + \alpha_m \frac{\sinh 2\gamma_m}{2\cosh\alpha_m} \right]$$

$$\left. + \frac{\cosh(\alpha_m - 2\gamma_m)}{2\cosh\alpha_m} \frac{m\pi y}{a} \sinh\frac{m\pi y}{a} \right\} \sin\frac{m\pi x}{a}$$

The following notation is used in the equation above and the following discussion:

| | | |
|---|---|---|
| w | = | deflection |
| q | = | intensity of a uniformly distributed load per unit of loaded area. |
| a | = | length of edge of square slab. |
| D | = | flexural rigidity of the slab. |
| $a_1$ | = | edge length of the loaded square. |
| m | = | positive odd integers ranging from 1 to infinity |
| $\alpha, \beta, \gamma$ | = | dimensionless constants. |
| P | = | total load on the slab |
| $\nu$ | = | Poisson's ratio |
| E | = | Young's modulus |
| x, y | = | coordinates of a point in the slab. |
| $M_x$ | = | bending moment along x - axis |
| $M_y$ | = | bending moment along y - axis |

$h$ = thickness of a solid slab with flexural strength equal to that of the cellular slab.

$m_x$ = moment along x - axis due to a unit load.

$m_y$ = moment along y - axis due to a unit load.

Maximum deflection occurs at the center point of the slab where the coordinates are,

$$x = \frac{a}{2} \cdot y = 0$$

In this case, $a = 54$ in.

$a' = 12$ in.

and $\alpha_m = \dfrac{m\pi}{2}$

$\gamma_m = \dfrac{m\pi}{18}$

and $\alpha_m - 2\gamma_m = \dfrac{7}{18} m\pi$

Substituting into the equation and taking only first two terms of the series, we obtain

$$w = 0.0416 \frac{4 q a^4}{D \pi^5}$$

and substituting $D \doteq \dfrac{E h^3}{12(1-\nu^2)}$

$$w = 0.131 \frac{P a^2}{E h^3} (1-\nu^2)$$

For $\nu = .3$ , $w = .119 \dfrac{Pa^2}{Eh^3}$

which is comparable with $w = .1265 \dfrac{Pa^2}{Eh^3}$ for point load as given by Timoshenko in table 20, p. 158.

For $\nu = .15$, $w = .128 \dfrac{Pa^2}{Eh^3}$

Also, $M_x = -D \left( \dfrac{\partial^2 w}{\partial x^2} + \nu \dfrac{\partial^2 w}{\partial y^2} \right)$

Combining the two expressions for $M_x$ and $w$, the bending moment at any point can be found. The final expression for maximum moment can be reduced to the following equation:

$$(M_x)_{max} = \beta a_1^2 q = \beta P$$

The value of $\beta$ can be found in table 17 of the above reference, for $\nu = 0.3$, as approximately 0.2. Values of coefficient $\beta$ can also be found in Reynolds' "Reinforced Concrete Designer's Handbook", p. 233, from which

$m_x = 0.15$ is obtained for

$$\frac{a_1}{a} = \frac{12}{54} = 0.222$$

hence, $M_x = M_y = P(m_x + 0.15\, m_y)$

$$= 0.17\, P$$

This value is applicable to material with Poisson's ratio of 0.15 which is a commonly used value for concrete.

It is noted that the foregoing formulas are valid under the following conditions:

(1) Material in the slab is homogeneous and elastic.

(2) The rigidity of the loading block is negligible

and (3) The four edges of slab are restrained against vertical movement but are free to rotate.

In the laboratory tests of the slabs, it can hardly be claimed that any of these conditions were completely fulfilled. Nevertheless, it was assumed that the deviations from ideal conditions during these tests produced only minor changes in the behavior of the test specimen. It was also assumed that the deflections and strains measured at a point 3 in. from the loading block were essentially the same as those values at the center of the slab.

In the following discussion of the properties of the slabs, the section modulus of the cell was computed to be 22.05 in.$^3$. From this, the thickness of equivalent solid slab was found to be 4.7 in. Young's modulus determined for a 3/4- by 2- by 6-in. concrete strip, cut from a cell was $3 \times 10^6$ psi. The theoretical maximum deflection of the slab based on these properties of its section was calculated and is shown in figure 4.

It is recalled that the relation between the bending moment M and the applied load P is given by the expression

$$M = 0.17P$$

From this expression and the flexure formula $M = f_c \frac{I}{c}$,

the load P corresponding to $f_c = 1000$ psi is 21,600 lb.

The strain in concrete corresponding to 1000 psi with $E = 3 \times 10^6$ psi is $333 \times 10^{-6}$ in./in. The average strain observed during the prestressing operation was $313 \times 10^{-6}$ in./in.

The relationship between the applied load and the strain in the concrete is illustrated in figures 5 and 6.

Comparing the calculated strains and deflections with observed data, it can be seen that they agree fairly well for Slabs Nos. 3 and 4 which had grouted joints. The observed load-deflection relationship for ungrouted Slab No. 2 was markedly different from the computed relationship.

## 5.2 Discussion of test results

From the data given in section 5.1, it is evident that prestressed grouted slabs do not exhibit any sharply defined transition from uncracked to cracked slabs at the point where the initial prestress at the bottom fibers is reduced to zero. There is, rather, a gradual transition as evidenced by both the load-deflection and load-strain relationships in figures 4, 5, and 6. In Slabs Nos. 3 and 4 there was a gradual increase in the center deflection up to a load of about 30,000 lb, or about 85 percent of the load carrying capacity. As the loads approached the maximum values, cracking progressed over the tension face of the slab and deflections increased rapidly. It is probable that at this stage of loading the cracking around the elliptical holes in the cells was extensive. Examination of the cells after test revealed the presence of diagonal tension cracks extending into two or three rows of blocks adjacent to the loading block. This is illustrated in figures 12 and 13.

The failure of the slabs was primarily caused by punching shear around the periphery of the loaded area. The failure in every case was accompanied by the loaded area being "punched" out. This is strikingly demonstrated in figure 7 where the central group of four ungrouted cells is shown displaced about 3/4 in. below the adjoining cells and a similar view of the top is shown in figure 8. The displacement of the central loaded area is not quite as severe for the grouted slab as can be seen in figure 12.

At ultimate load, the prestress was completely nullified and the slab behaved as an ordinary non-prestressed concrete slab. The diagonal tension in this case is numerically equal to vertical shear. The effective web area of the cells immediately adjacent to the central loaded area was 3 in.$^2$ for each cell, or 36 in.$^2$ for the entire periphery of the loading block. The shear strengths of the grouted Slabs Nos. 3 and 4 were, thus, 980 and 950 psi, respectively. The Slabs Nos. 3 and 4 were able to develop diagonal tensile stresses of nearly 1000 psi because of the presence of closely spaced welded wire reinforcement in the webs and flanges of the cells.

## 6. CONCLUSIONS

Although the study of prestressed cellular slabs is still in progress the following conclusions may be drawn from the test results obtained so far:

(1) A prestressed slab made of precast hollow cells may be expected to behave like a homogeneous material in flexure and shear.

(2) A simply supported slab made of these cells, subjected to concentrated load, with short span and nominal prestressing fails in punching shear.

(3) Welded wire reinforcement in the cell webs contributed a considerable amount to the load carrying capacity of the slab.

(4) For a structure of this type, intimate contact between the cells is of utmost importance.

(5) With reasonably rigid anchorage plates for prestressing bars, solid end blocks are not necessary, provided the anchorage plates are nearly equal in area to the cell with which they are in direct contact.

Table 1. Loss of strain in steel and concrete with time in Slab No. 3.

| Condition of slab | Prestressing Tendons Nos. | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| Initial strain, $10^{-6}$ in./in. (slab on working platform) | 1263 | 1291 | 1331 | 1264 | 1285 | 1299 | 1285 | 1300 |
| Strain increment, $10^{-6}$ in./in. (slab simply supported 1 day) | 0 | +3 | 0 | +4 | +5 | +3 | +10 | +5 |
| Strain increment, $10^{-6}$ in./in. (slab simply supported 6 days) | -8 | -13 | -11 | -12 | -13 | -2 | -9 | -3 |

| Condition of slab | Gages on Concrete | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| Initial strain, $10^{-6}$ in./in. (slab on working platform) | 312 | 375 | 334 | 332 | 245 | 385 | 253 | 274 |
| Strain increment, $10^{-6}$ in./in. (slab simply supported 1 day) | +17 | +6 | +4 | -6 | +125 | +241 | +65 | +100 |
| Strain increment, $10^{-6}$ in./in. (slab simply supported 6 days) | +17 | +20 | +3 | 0 | +17 | +19 | +15 | +18 |

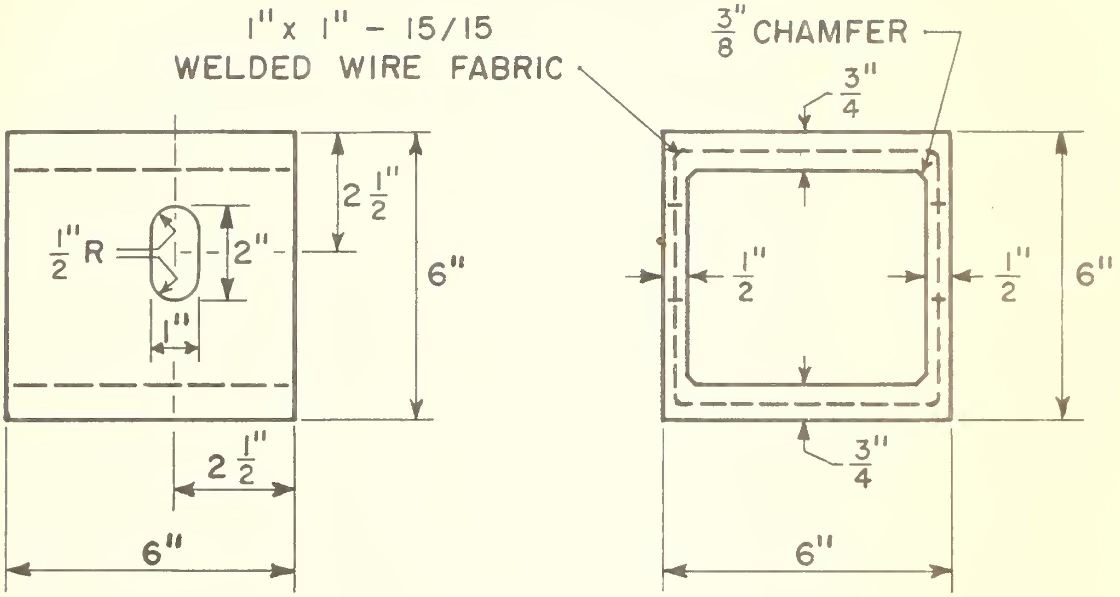
\+ indicates an increase in strain.
\- indicates a decrease in strain.

Table 2. Strain increments in prestressing bars due to applied loads.

| Slab No. | Load kips | Strain increments, $10^{-6}$ in./in., in bars Nos. | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| 2 | 5 | 5 | 8 | - | 3 | 13 | -5 | 15 | -1 |
| | 10 | 0 | 5 | - | 0 | 23 | -24 | 19 | -21 |
| | 13 | -19 | -11 | - | -20 | 16 | -60 | 14 | -57 |
| | 15 | -43 | -32 | - | -39 | 11 | -96 | 7 | -85 |
| 3 | 5 | 4 | 7 | 4 | 3 | 6 | -1 | 8 | 0 |
| | 10 | 3 | 5 | 4 | 3 | 10 | -6 | 9 | -6 |
| | 15 | 3 | 5 | 3 | -1 | 13 | -10 | 14 | -10 |
| | 20 | -2 | 4 | 3 | -6 | 17 | -17 | 19 | -20 |
| | 25 | 0 | 8 | 4 | -6 | 21 | -17 | 24 | -21 |
| | 30 | 6 | 14 | 10 | 4 | 27 | -26 | 35 | -31 |
| | 32.5 | 13 | 22 | 18 | 0 | 36 | -26 | 45 | -32 |
| | 35 | 34 | 44 | -- | -- | -- | -- | -- | --- |
| 4 | 2.5 | 2 | -8 | -- | -7 | 0 | -9 | 0 | -7 |
| | 5 | 1 | -4 | -- | -3 | 8 | -19 | 0 | -7 |
| | 7.5 | 2 | -3 | -- | -4 | 8 | -10 | 9 | -4 |
| | 10 | 3 | -3 | -- | -5 | 7 | -16 | 8 | 23 |
| | 15 | 10 | -4 | -- | -7 | 18 | -14 | 10 | 29 |
| | 20 | 12 | -3 | -- | -8 | 19 | -19 | 18 | -- |
| | 25 | 18 | 0 | -- | -5 | 29 | -19 | 20 | -28 |
| | 30 | 42 | 10 | -- | 2 | 6 | -20 | 40 | -22 |

Note: Total strain due to prestressing was approximately $1200 \times 10^{-6}$ in./in.

Bar 5 and 7 were 1/2 in. below centerline.
Bar 6 and 8 were 1/2 in. above centerline.




EFFECTIVE SECTION PROPERTIES OF A CELL IN A SLAB:

$A = 11.25 \text{ IN.}^2$

$I = 66.22 \text{ IN.}^4$

$\frac{I}{C} = 22.05 \text{ IN.}^3$

$Q = 13.1 \text{ IN.}^3$

$h$ = THICKNESS OF EQUIVALENT SOLID SLAB

= 4.7 IN.

FIGURE 1. DIMENSIONS AND SECTION PROPERTIES OF A CELLULAR BLOCK.

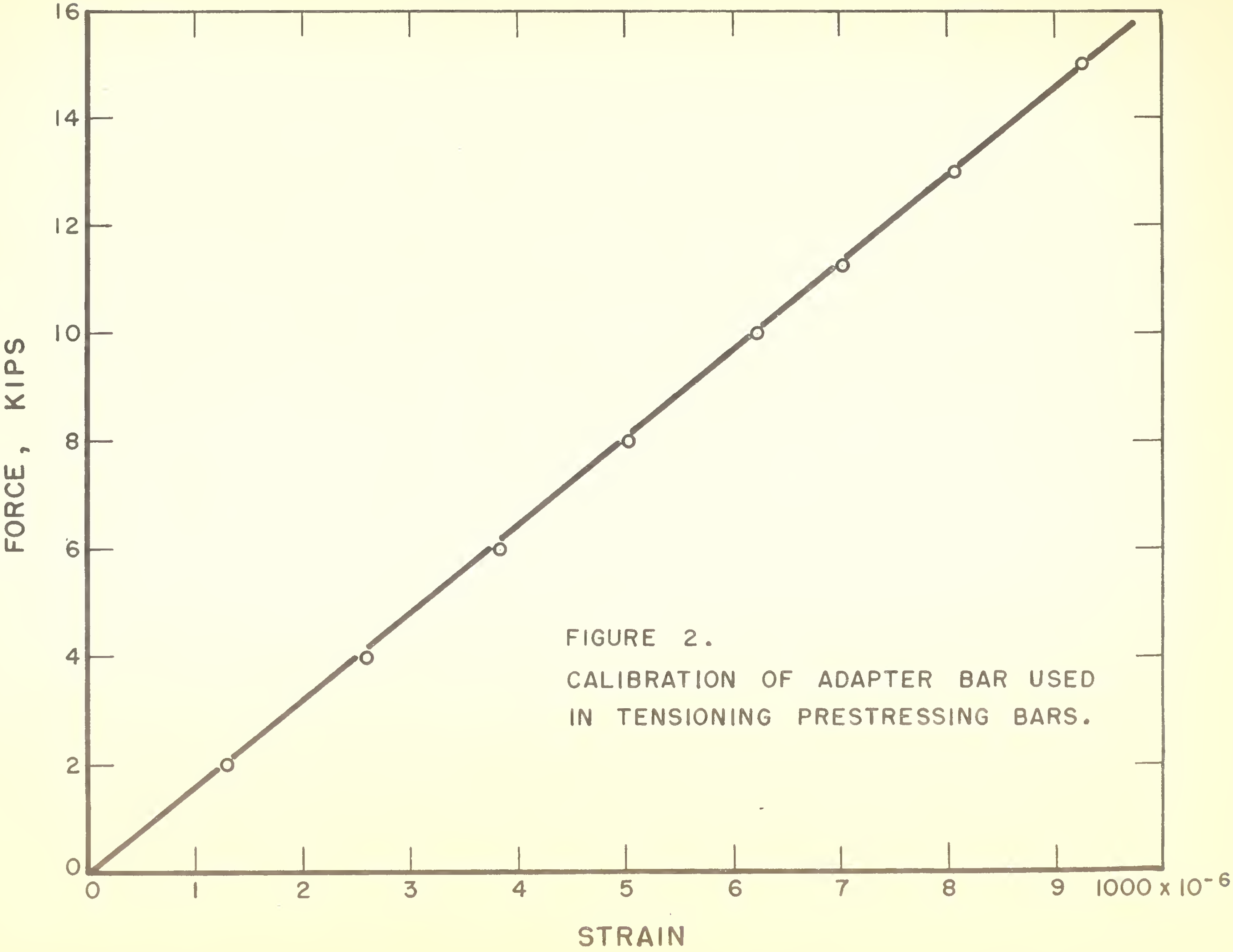
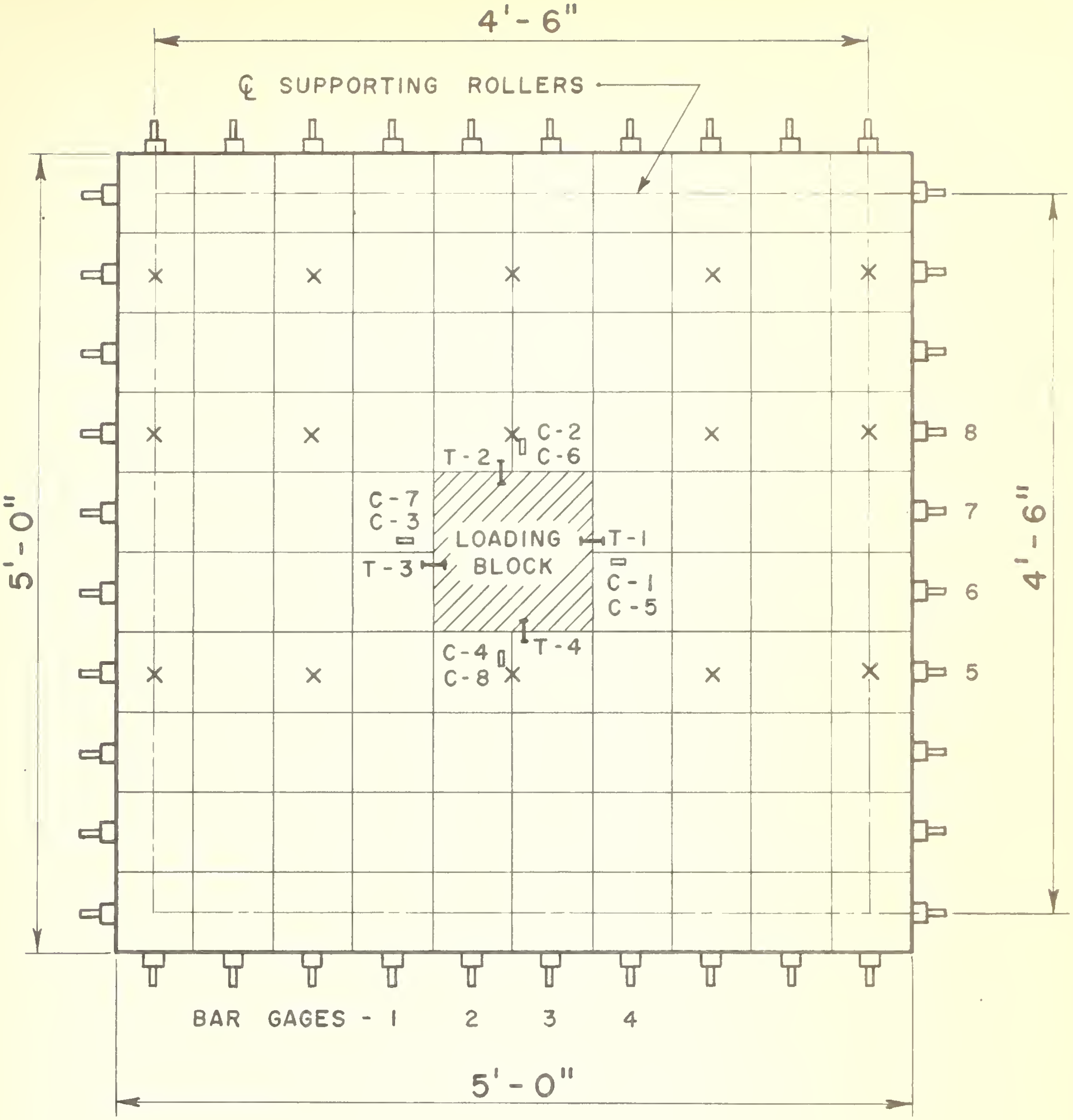



FIGURE 2.
CALIBRATION OF ADAPTER BAR USED IN TENSIONING PRESTRESSING BARS.




LEGEND

X INDICATES MICROMETER DIAL GAGES
T " TUCKERMAN GAGES
C " SR-4 GAGES
GAGES 1 THROUGH 4 ON BOTTOM SURFACE OF SLAB
" 5 " 8 " TOP " " "

FIGURE 3. LOCATION OF MICROMETER DIAL DEFLECTION GAGES, STRAIN GAGES ON CONCRETE AND PRESTRESSING TENDONS, AND TUCKERMAN GAGES

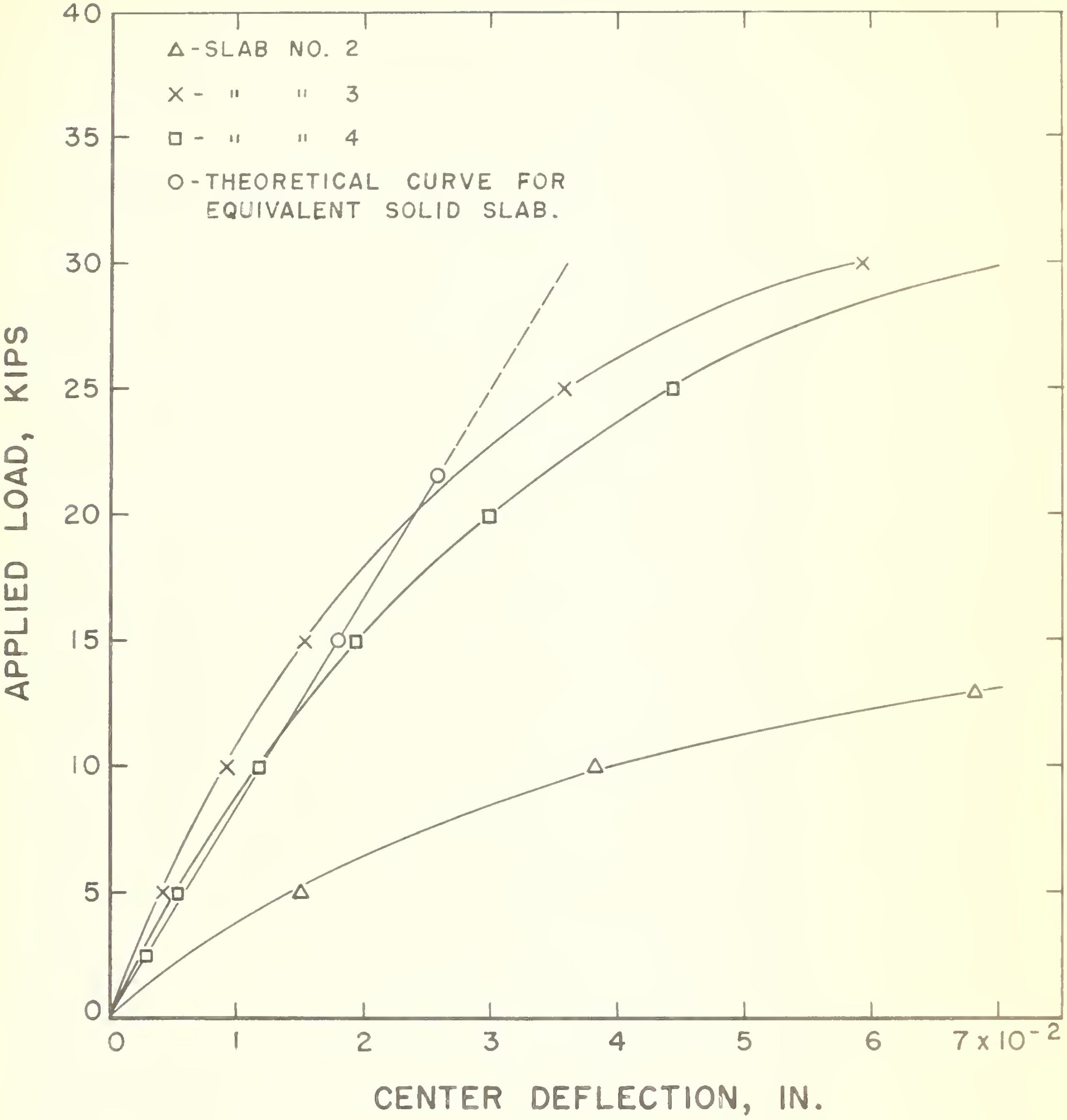



FIGURE 4. COMPUTED AND OBSERVED CENTER DEFLECTION IN SLABS NOS. 2, 3, AND 4.

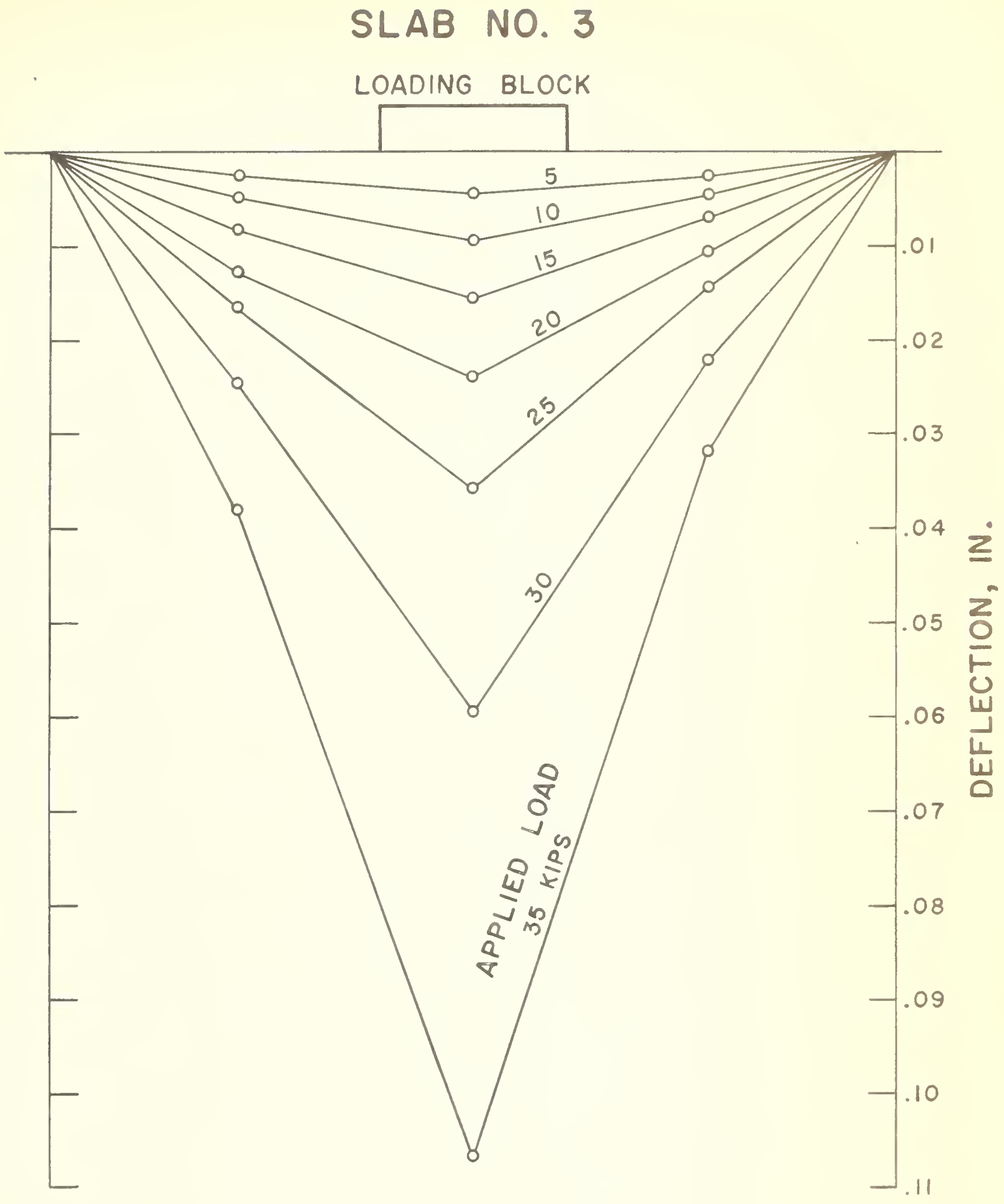



FIGURE 4A. DEFLECTION PROFILE OF SLAB NO. 3 AT A SECTION ADJACENT TO THE LOADING BLOCK.

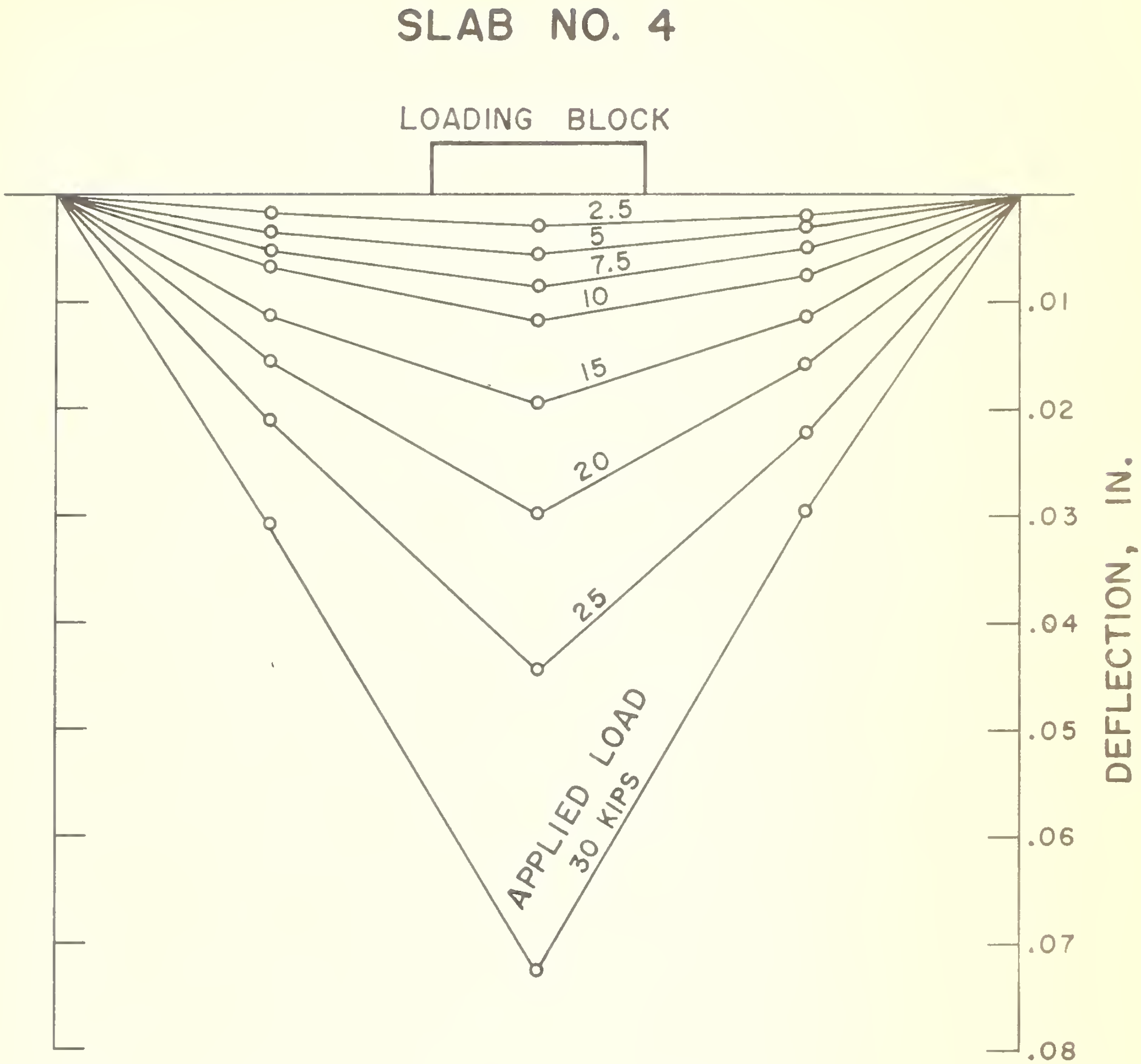



FIGURE 4B DEFLECTION PROFILE OF SLAB NO. 4 AT A SECTION ADJACENT TO THE LOADING BLOCK.

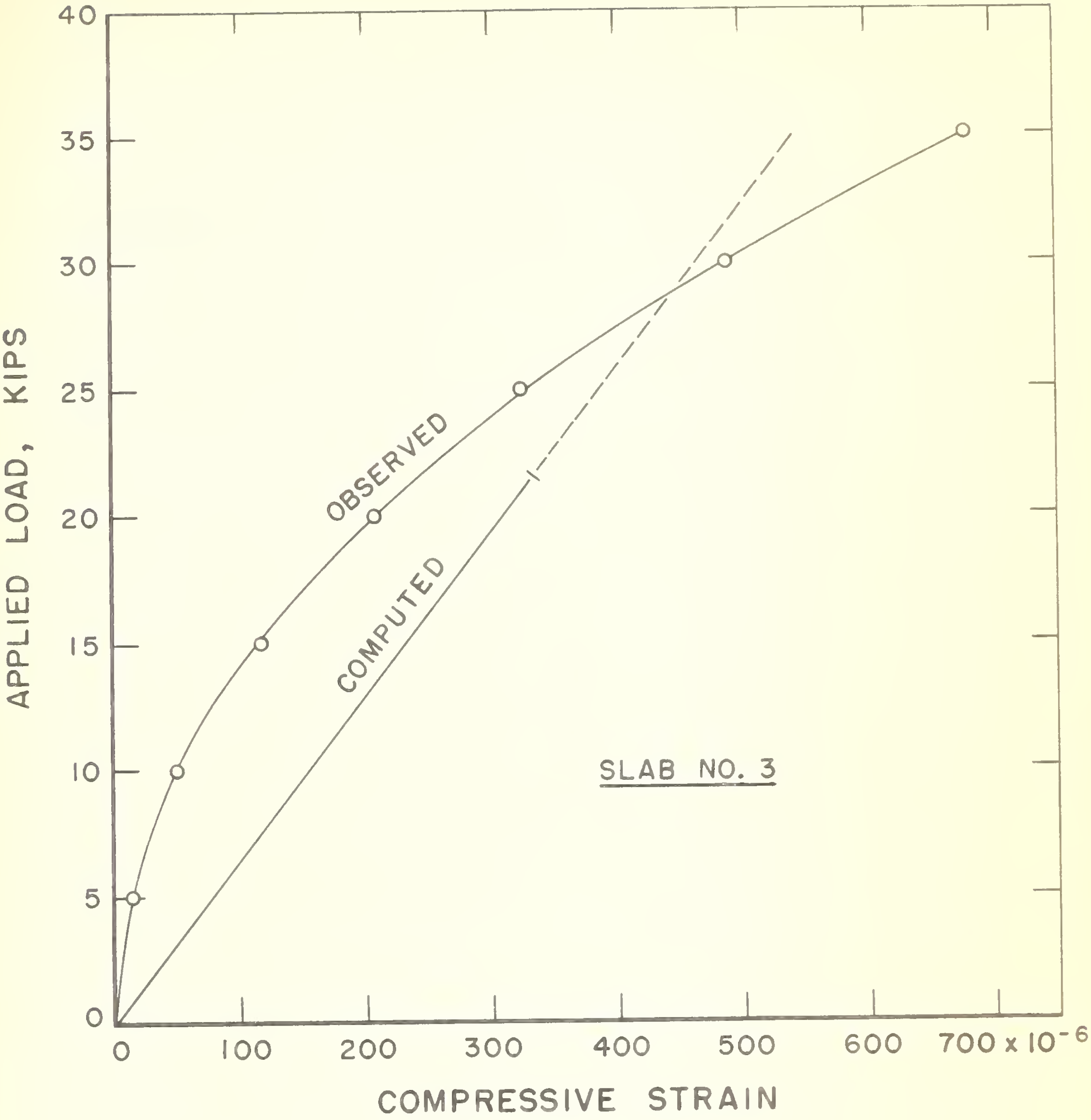



FIGURE 5. COMPUTED AND OBSERVED STRAINS IN CONCRETE IN SLAB NO. 3.

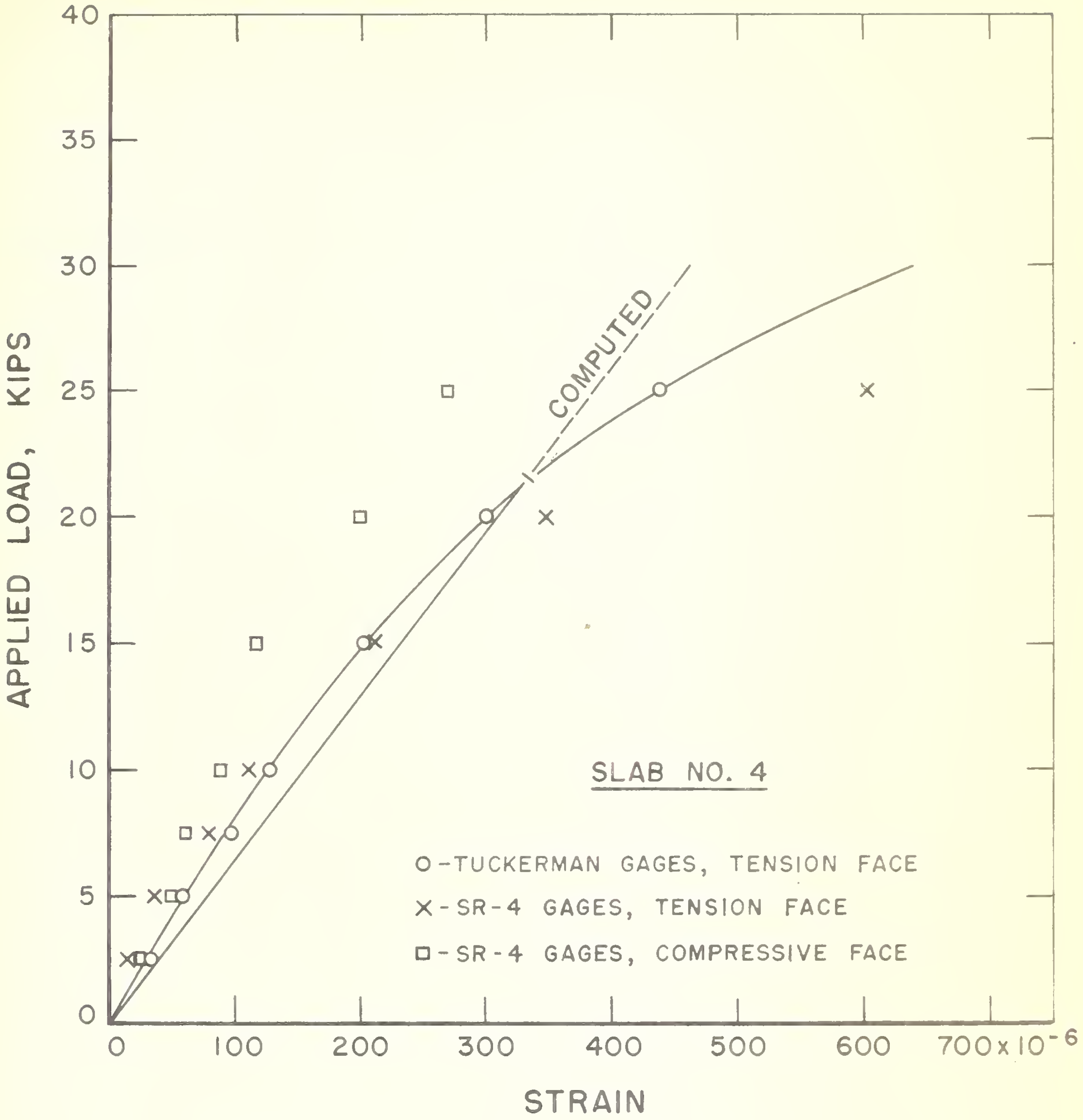



FIGURE 6. COMPUTED AND OBSERVED STRAINS IN CONCRETE IN SLAB NO. 4.

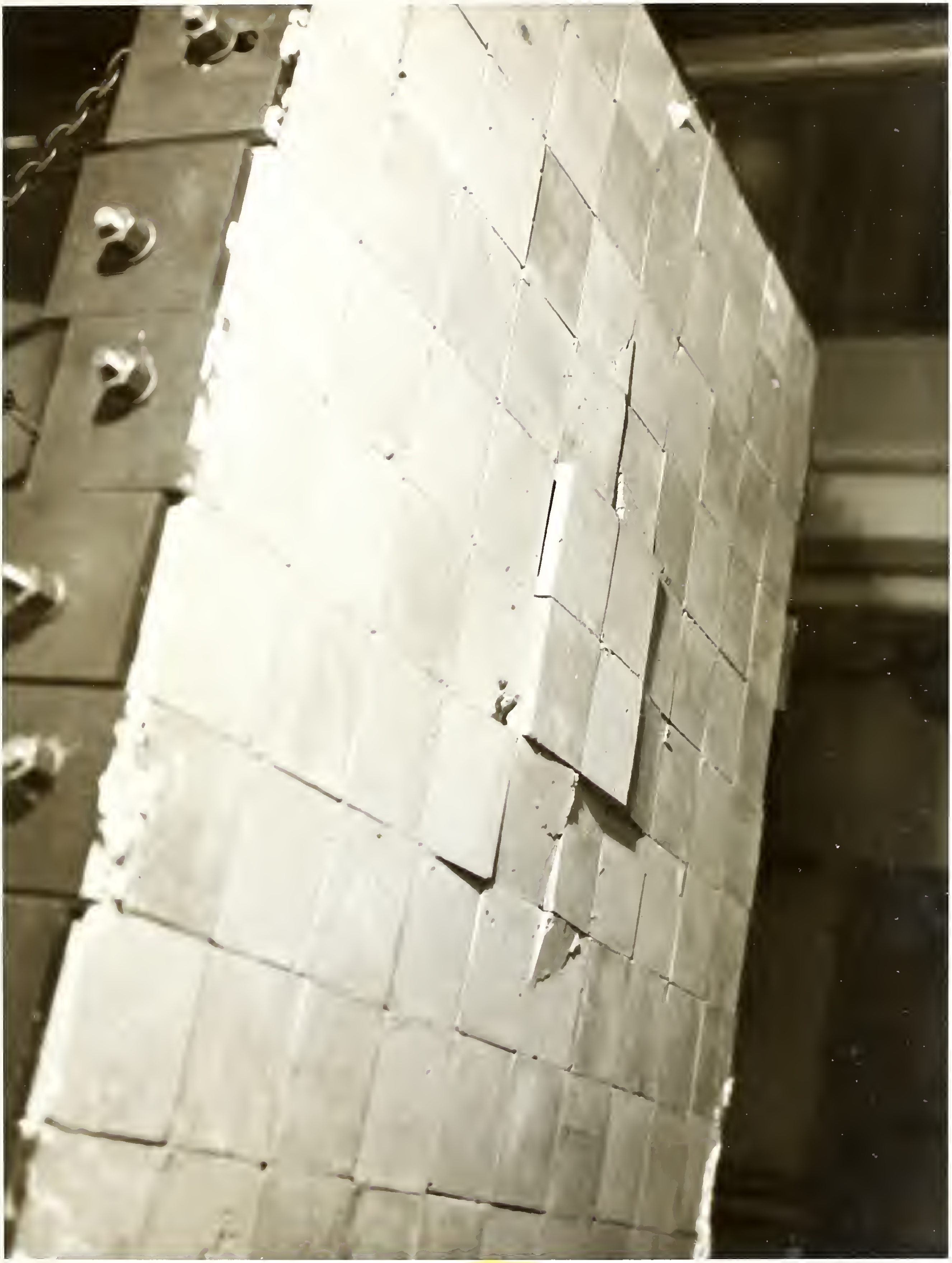


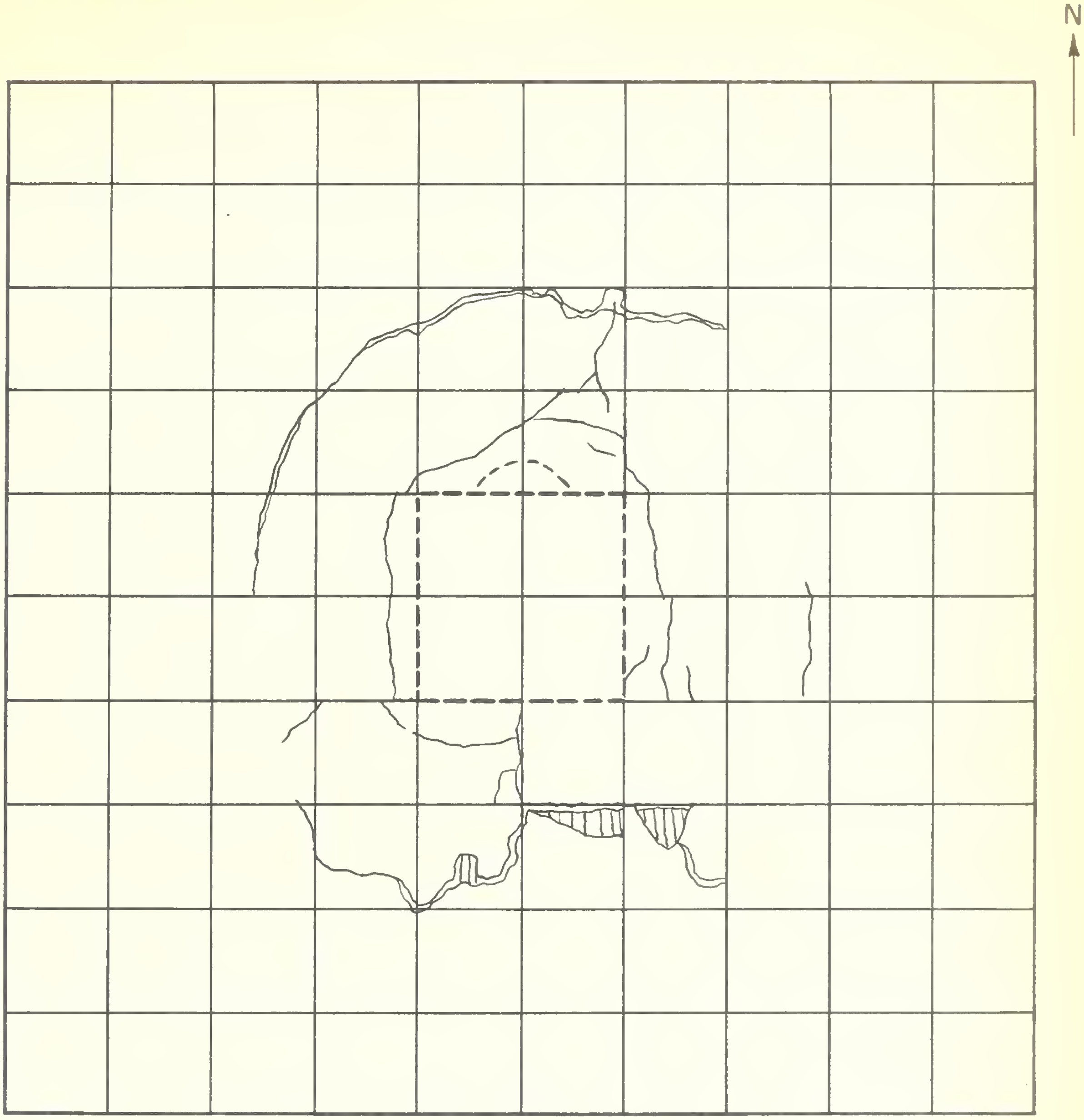
Fig. 7. Bottom of slab No. 1 after failure.

Fig. 8. Top of slab No. 2 after failure.

Fig. 9. Bottom of slab No. 2 after failure.

NOTE:

LOADING BLOCK PENETRATED 1/4 IN. BELOW TOP SURFACE OF SLAB, (SHOWN BY DOTTED LINE). THERE WAS LITTLE OTHER DAMAGE ON TOP OF SLAB.




BOTTOM VIEW OF SLAB NO. 4

FIGURE 10. CIRCULAR CRACK PATTERN IN BOTTOM OF SLAB NO. 4.

Fig. 11. Circular crack in top of slab No. 3.

Fig. 12. Crack pattern in cross section of a slab.

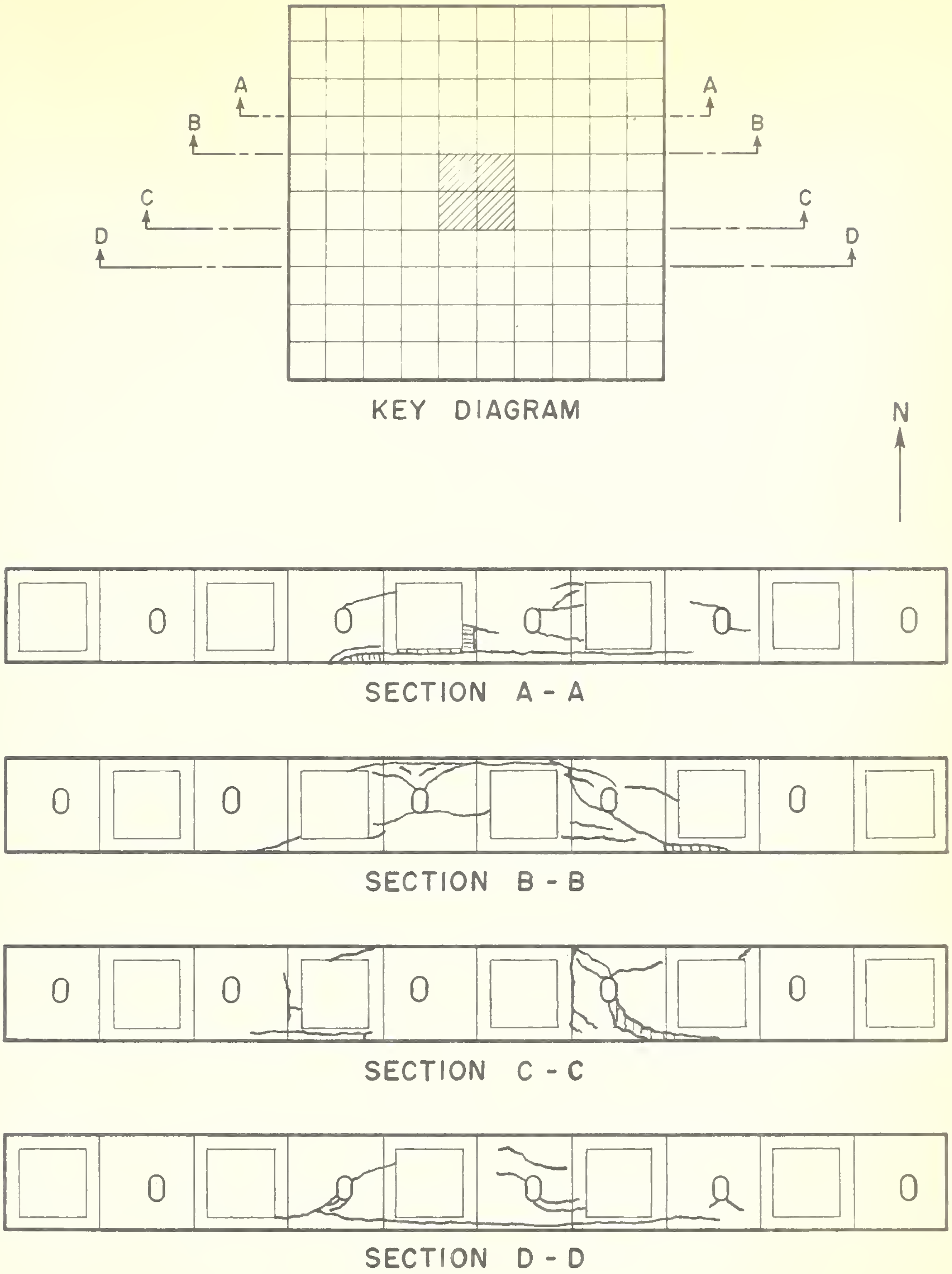



FIGURE 13. CRACK PATTERN IN SECTIONS OF SLAB AT VARIOUS DISTANCES FROM THE LOADED AREA.

Fig. 14. Diagonal crack pattern in individual units.

## THE NATIONAL BUREAU OF STANDARDS

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