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NATIONAL BUREAU OF STANDARDS REPORT

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FLEXURAL TESTS OF PRESTRESSED CELLULAR SLABS

by

A. F. Kirstein

Report to

Bureau of Yards and Docks
Department of the Navy

NBS

U. S. DEPARTMENT OF COMMERCE

NATIONAL BUREAU OF STANDARDS

THE NATIONAL BUREAU OF STANDARDS

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NBS PROJECT **NBS REPORT**

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To further the investigation of the properties of prestressed cellular slabs, two specially designed specimens were subjected to pure bending. These slabs were made of unreinforced NBS blocks, and were prestressed to 1000 psi in both directions. Deflections, strains, crack patterns, and maximum bending moments were recorded. Both slabs exhibited abrupt compression failures in the top flange of the blocks. A comparison with previously tested slabs subjected to a concentrated load at midspan failing in shear shows that the slabs reported herein failed at bending moments 39 and 63 percent greater than the maximum moments resisted by those slabs that were supported along two and four edges respectively. This indicated that a sizeable increase in load-carrying capacity could be realized in the previously tested slabs by increasing the resistance of the prestressed slabs to diagonal tension and punching shear.

1. INTRODUCTION

In the previous tests carried out in the investigation of properties of prestressed cellular slabs, the specimens were 5- by 5-ft slabs supported either on four or two edges and subjected to a concentrated load at the center. The information developed in these tests served as the basis for modification of the design of the cellular blocks and the method of jointing which resulted in substantial increases in the shear resistance of these slabs.

In the current phase of the study of these slabs, the object is to determine the flexural strength of prestressed assemblies. The specimens selected for these tests were 2- by 10-ft slabs loaded at two points so as to have a section 5 feet long subjected to pure bending. The results obtained in these tests will serve to plan future flexural tests and also shear tests of specimens in which the ratio of shear span to depth of slab will be a variable.

2. DESCRIPTION OF TEST SPECIMENS

2.1 Cellular Blocks

The cellular concrete blocks used in this investigation were the regular unreinforced NBS blocks that were hollow six inch cubes having an opening 4.5 by 4.5 in. in cross section. There was a 1- by 2-in. elliptical access hole in each web to permit the passage of the prestressing tendon. The principal dimensions of the cellular blocks are shown in figure 1.

The blocks were made of a mix proportioned of one part Type III cement and three parts of sand, by weight, with a water-cement ratio of 0.57. This mix had a 7-day compressive strength of approximately 6000 psi as determined by tests of 2 in. cubes. However, the actual units were moist-cured for a considerably longer period of time and then placed in dry storages before they were assembled into slabs. Therefore, the compressive strength of the concrete in the individual units should be expected to be somewhat greater.

The Young's modulus and Poisson's ratio of the concrete were determined in previous tests of the concrete units and other specimens. Axial compression tests and sonic modulus tests indicated an average Young's modulus of 4×10^6 psi with a variation of $\pm$ 10 percent. Poisson's ratio was found to be approximately 0.15.

2.2 Prestressing Steel

"Elastuff" steel prestressing tendons were used in the slabs. Tensile tests of this material indicated a stress-strain relationship that was essentially linear up to 95,000 psi, and exhibited a Young's modulus of approximately 30×10^6 psi. The yield strength of the bar was found to be 120,000 psi as determined by the 0.2 percent offset method and the tensile strength was found to be 133,000 psi. Although "Elastuff" bars are made of cold-worked high carbon steel, they are fairly ductile and can be machined easily.

2.3 Description of Prestressed Slabs

The overall dimensions of slabs were 2 ft by 10 ft 3 in. (fig. 4).

The nominal dimensions of the portion of each slab which was in pure bending and which contained 40 cellular blocks, were 2 ft by 5 ft by 6 in. The blocks were arranged in a crisscross pattern so that the axis through the open ends of one block was perpendicular to the axis of each adjacent block. The holes in the webs of the block were arranged to permit the longitudinal prestressing tendon to be located along the mid-plane of the slab, and the transverse tendons were staggered above and below the mid-plane. Thus, the resultant prestressing force produced an axial compression of 1000 psi in two directions through the slab.

There were two slabs tested in this investigation. Both slabs were constructed as specified above, except that. Slab No. F-1 had epoxy resin and Slab No. F-2 had neat cement paste as jointing materials.

Both slabs were equipped with concrete end spans in the longitudinal direction that withstood the shearing forces of the load and served as anchorage blocks. With the use of these end spans and the loading technique described later it was possible to subject the test section of the specimen to pure flexure.

2.4 Prestressing Procedure

The tensioning force was applied to the prestressing bars by means of a hydraulic jacking rig that was equipped with a dynamometer to determine the amount of prestressing that was applied to the tendons. The calibration curve for the dynamometer is shown in figure 2.

Approximately one-half of the prestress was applied to the slab in small increments by tightening the anchorage nuts with a wrench. The remaining prestressing force was applied by means of the hydraulic jacking rig. This final stage of the prestressing operation was accomplished by using a suitable sequence of stressing the tendons so that no unduly large differences in strain would be induced in the blocks.

3. TESTING PROCEDURE

3.1 Test Setup

A 600,000-lb capacity hydraulic testing machine was used to test the slabs. The specimens were simply supported by rockers that rested on the testing machine platen, and the loads were applied through a knife edge at one end of the test section and rollers at the other end. The loads were distributed across the end of the concrete shear spans with stiffened I sections, and all loads and reactions were transmitted to the specimen through steel plates that were firmly set in plaster. Figure 3 is a photograph of one of the slabs and the test setup.

3.2 Instrumentation

The deflection measurements of these slabs were made with 0.001-in. micrometer dial gages that were attached to steel angles resting onthe slab directly over the joints between the test section and the end shear spans, thus placing the datum plane on the top surface of the specimen and at the ends of the test section. Figure 4 shows the symmetrical arrangement of these gages.

The strains in the concrete on the top and bottom surfaces of the slabs were measured with bonded electrical resistance wire gages of the A-3 type. The location of these gages are also shown in figure 4.

Bonded electrical resistance wire gages of the A-3 type were also placed at the center of the two prestressing tendons indicated in figure 4 to determine the strains in the steel due to the applied load.

3.3 Test Procedure

Both specimens were loaded as shown in figures 3 and 4. The load was applied in increments of 1000 lb, and gage readings were made for each increment until the maximum load was reached.

4. TEST DATA

4.1 Deflection and Load-carrying Capacity of Slabs

Slab No. F-1 carried a maximum applied load of 16,000 lb and Slab No. F-2 carried a maximum of 15,700 lb which correspond, respectively, to total bending moments of 213.6 and 210 in-kips when the dead load is considered. During the test, signs of distress were noted in both slabs at loads considerably below the maximum. At about 130 in-kips of moment, audible cracking of the webs in Slab No. F-1 was noted, and shortly after that the epoxy joints on the tensile surface of the slab cracked open. Slab No. F-2 also cracked at about 130 in-kips, but the neat cement joints opened at the same time as the cracking occurred in the webs. Some crushing of the top surface of Slab No. F-2 was noted at about 200 in-kips. However, it continued to carry load up to 210 in-kips when it failed very abruptly. It appears that no strength advantage is gained from using the stronger epoxy resin as a jointing material. Both slabs failed by flexural compression.

The relationship between deflections and applied moments is shown in figure 5.

4.2 Concrete and Steel Strains

The concrete strains on the tensile and compressive surfaces of the slabs are plotted against the total bending moment in figure 6. The open circles represent data from Slab No. F-1, and the solid circles represent data from Slab No. F-2.

The relationship between the total bending moment and the steel strains are given for both slabs in figure 7.

4.3 Crack Patterns

The crack patterns exhibited by these two slabs were quite different from those observed in the previous tests where the slabs failed in diagonal tension or punching shear. The cracks that formed in the webs were for the most part horizontal as shown in figure 8. The shaded and missing portions of the blocks indicate the location of the compression failure.

The more widespread cracking shown in the webs of the blocks in Slab No. F-1 is probably due to the use of a quick setting plaster in the joint between the test section and the concrete end spans. It is believed that the rapid setting of the plaster resulted in nonuniform contact between the shear span and the test section of slab. A slower setting neat cement paste was used for these joints in Slab No. F-2, which produced a more uniform joint.

5. DISCUSSION

As mentioned in the preceding section both specimens carried approximately the same maximum bending moment, and showed signs of distress prior to failure at approximately the same stage of loading. Thus, it would appear that no real advantage was derived from the use of the stronger epoxy resin. However, it was also pointed out that the use of a quick setting plaster as a jointing material between the test section and the end shear spans of Slab No. F-1 having epoxy resin joints was the cause of the widespread premature cracking of the block webs in that slab. This widespread cracking shows up in the relationships between total bending moment and deflection, concrete strain, and steel strain as seen in figures 5, 6, and 7 respectively. It may have prematurely transferred the ultimate compressive stress to the top flanges of the blocks, thus causing a premature failure.

It is important to note that the apparent lack of rigidity shown in Slab No. F-1 can be traced to the imperfect plaster joints. The concrete and steel strains also reflect this action in greater compressive strains in the concrete and greater tensile strains in the steel.

Since both the epoxy resin and neat cement jointing materials are fairly brittle and the cracks in the joints formed at about 60 percent of the maximum moment, it is believed that greater usable strengths may be obtained in marine structures if a tough and more extensible joint material is used.

SUMMARY

The preceding work can be summarized as follows:

1. Both slabs failed at approximately the same load in abrupt compression failures of the concrete in the compressive flanges of the blocks.

2. Both slabs showed signs of distress prior to final failure at approximately 60 percent of the maximum load when cracks began to form.

3. There is a possibility that the use of a tougher and more extensible jointing material might increase the maximum usable strength of a marine or other structures where corrosion of the steel is a problem.

4. The apparent lack of rigidity of Slab No. F-1 was attributed to the use of a quick setting plaster as a jointing material between the test section and the concrete end shear spans, which might have resulted in an imperfect contact between the two portions of the slab.

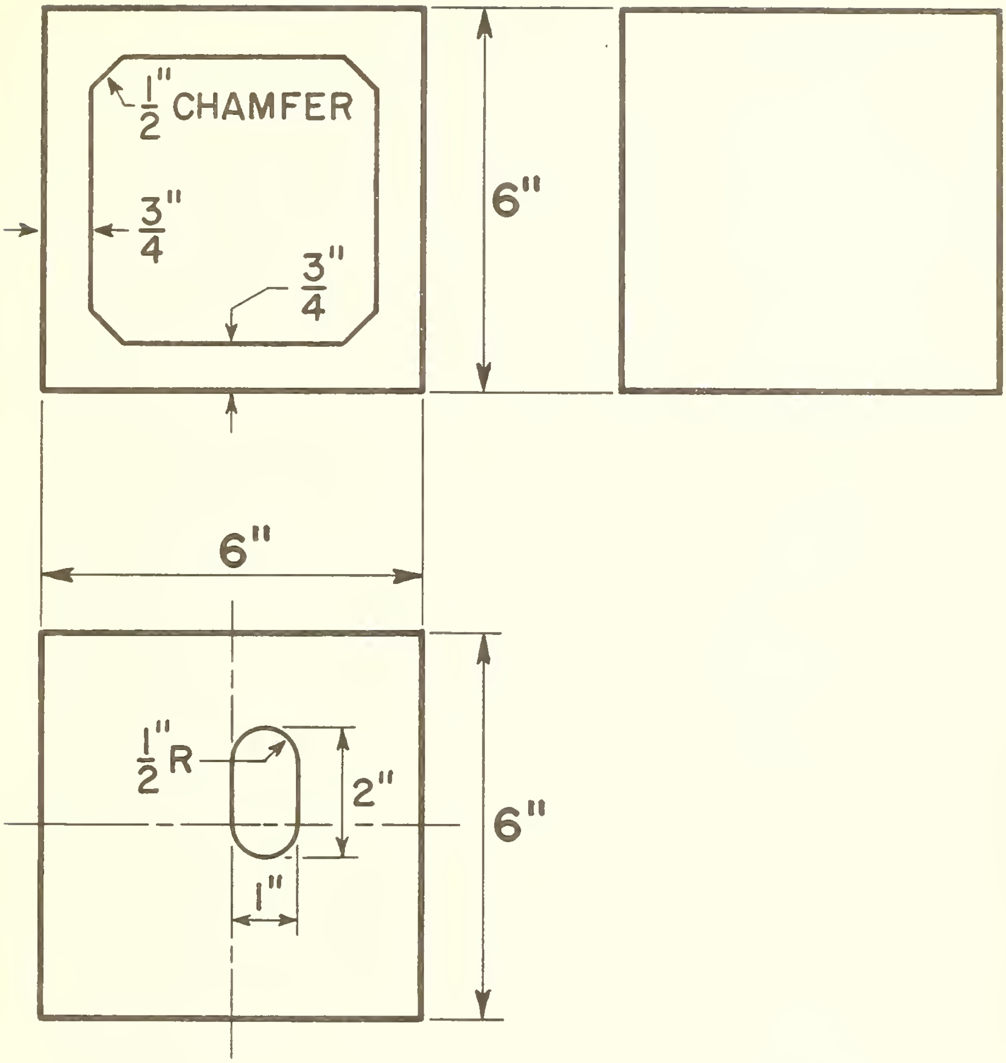


FIGURE I. NOMINAL DIMENSIONS OF NBS UNREINFORCED BLOCKS.

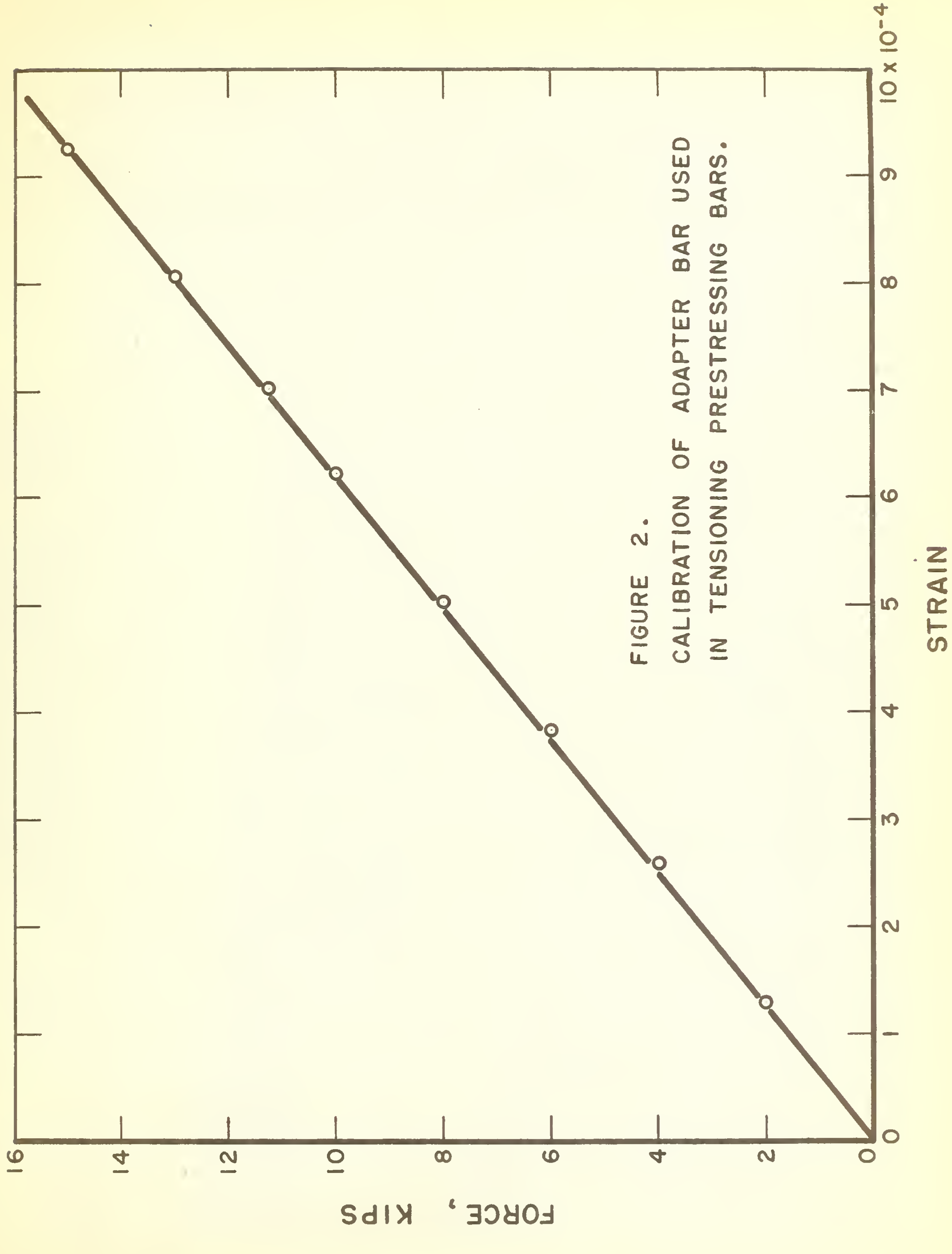


FIGURE 2.
CALIBRATION OF ADAPTER BAR USED
IN TENSIONING PRESTRESSING BARS.
FORCE, KIPS
0
2
4
6
8
10
12
14
16
STRAIN
0
1
2
3
4
5
6
7
8
9
10 x 10^-4

FIGURE 2.

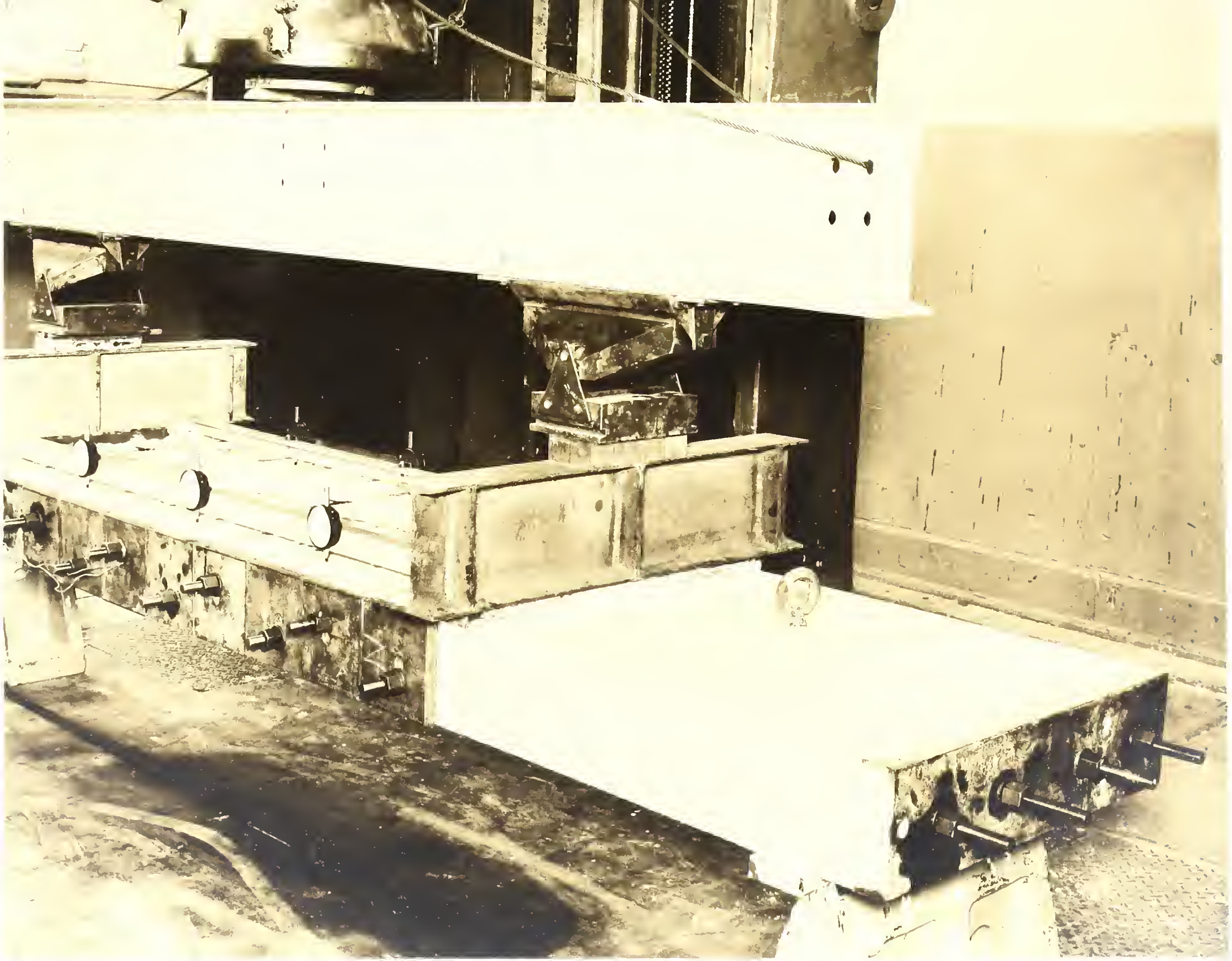
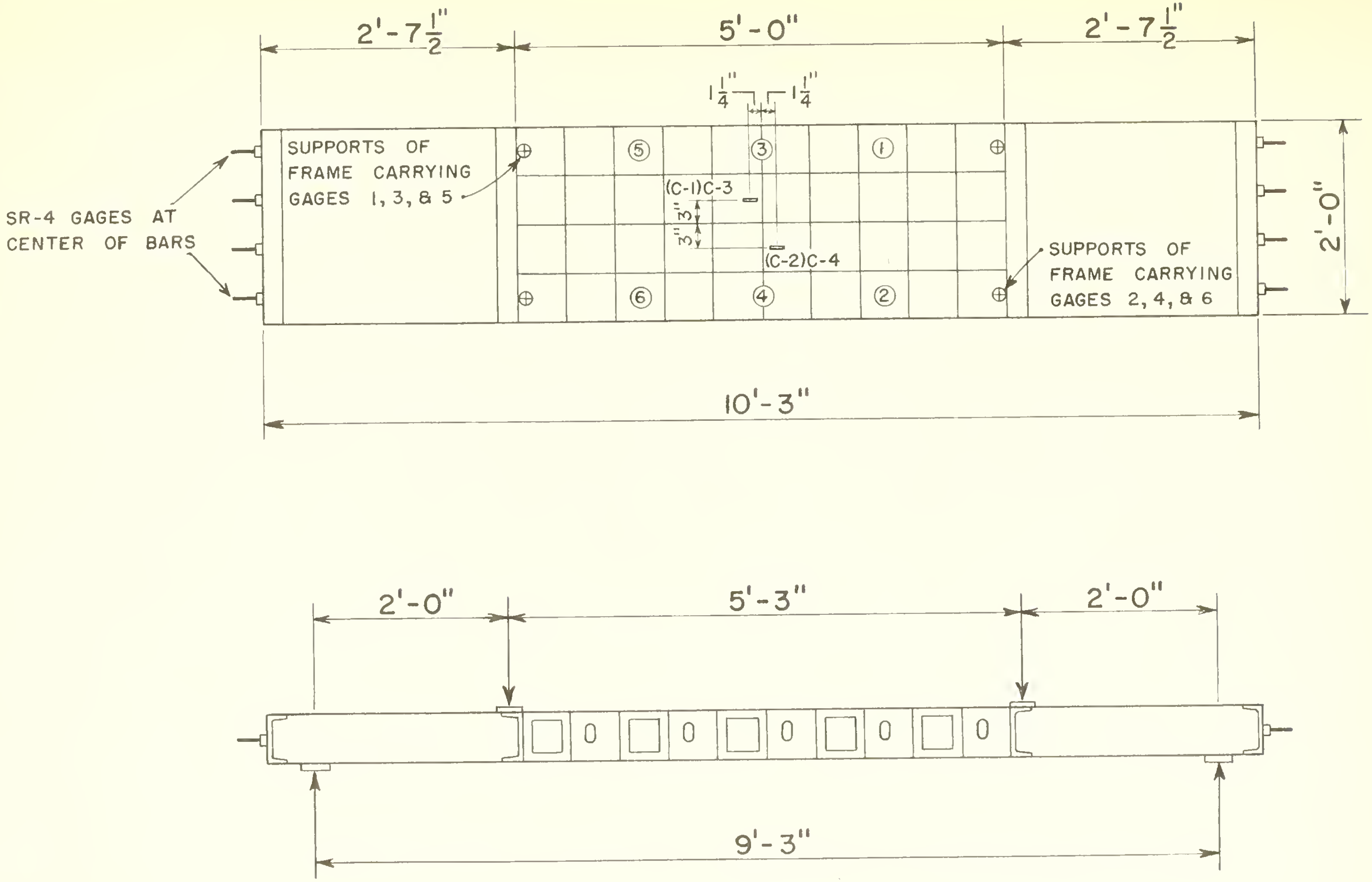


FIGURE 3. PHOTOGRAPH OF SPECIMEN AND TEST SETUP.



DIAL GAGES 1 THRU 6 (DEFLECTION)
SR-4 GAGES C-1 AND 2 CONCRETE STRAIN (BOTTOM)
SR-4 GAGES C-3 AND 4 CONCRETE STRAIN (TOP)

FIGURE 4. INSTRUMENTATION AND LOADING DIAGRAM OF SLABS

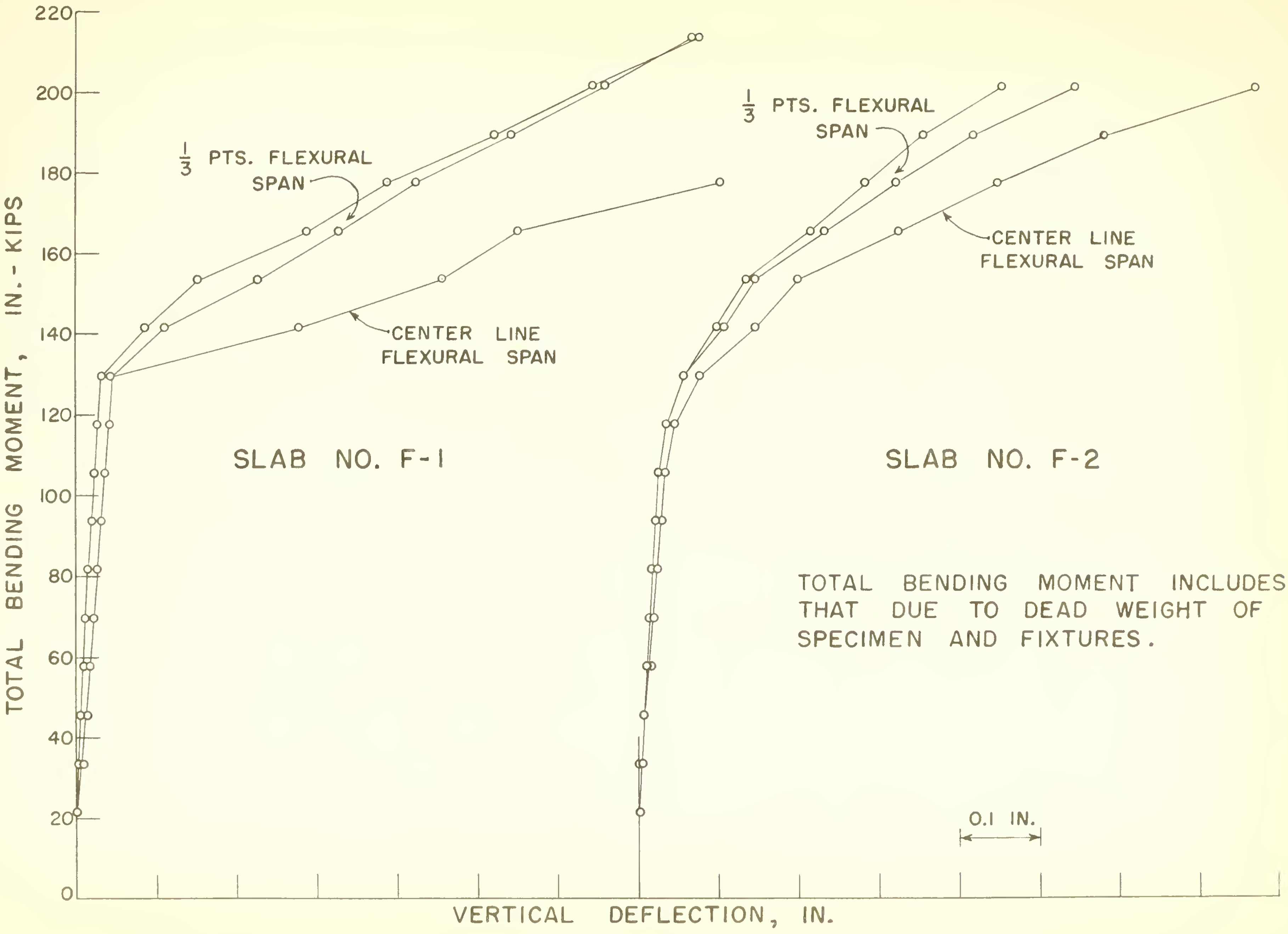


FIGURE 5. BENDING MOMENT AND DEFLECTION RELATIONSHIP OF SLABS

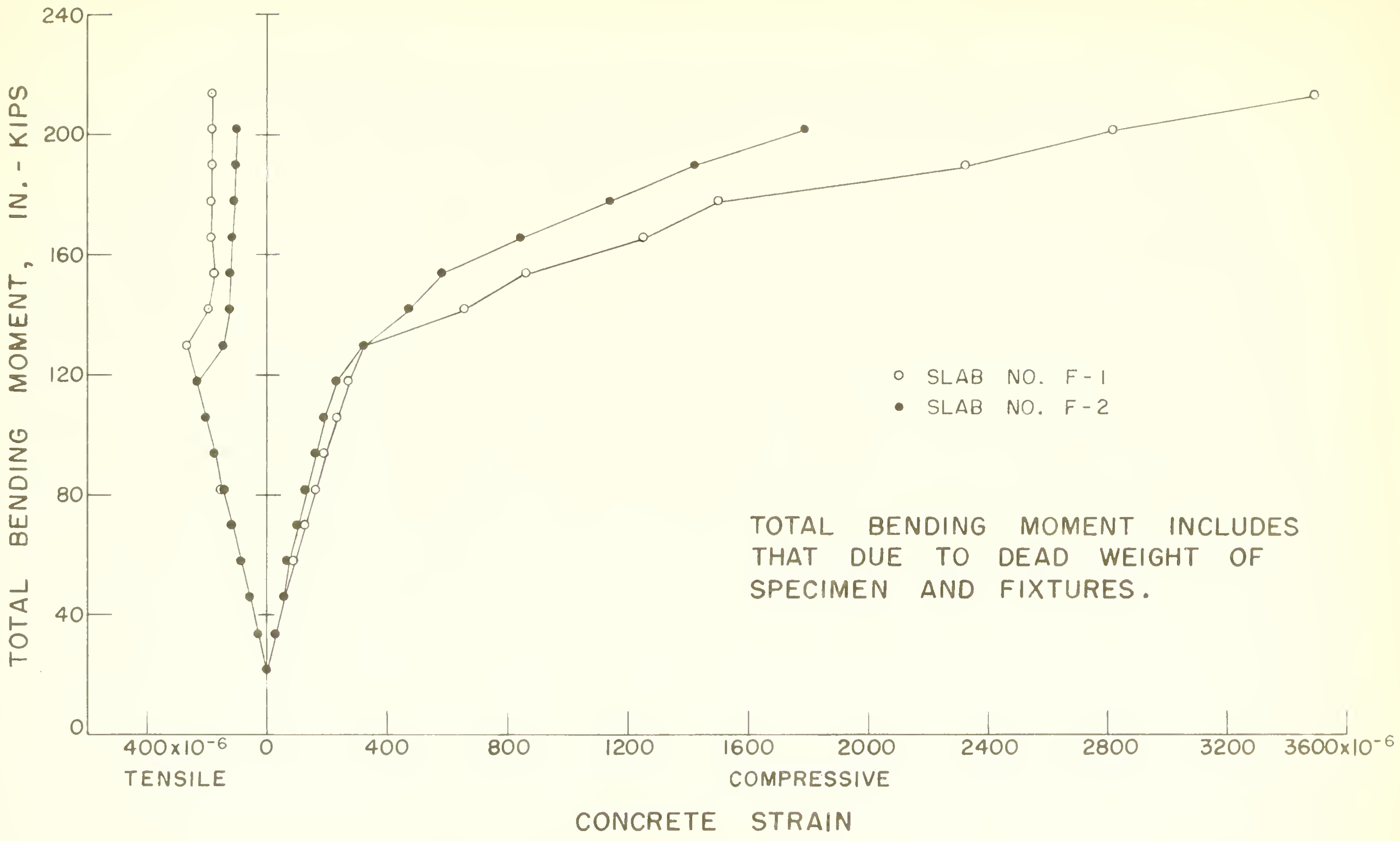


FIGURE 6. CONCRETE STRAINS ON THE TENSILE AND COMPRESSIVE SURFACES OF THE SLABS

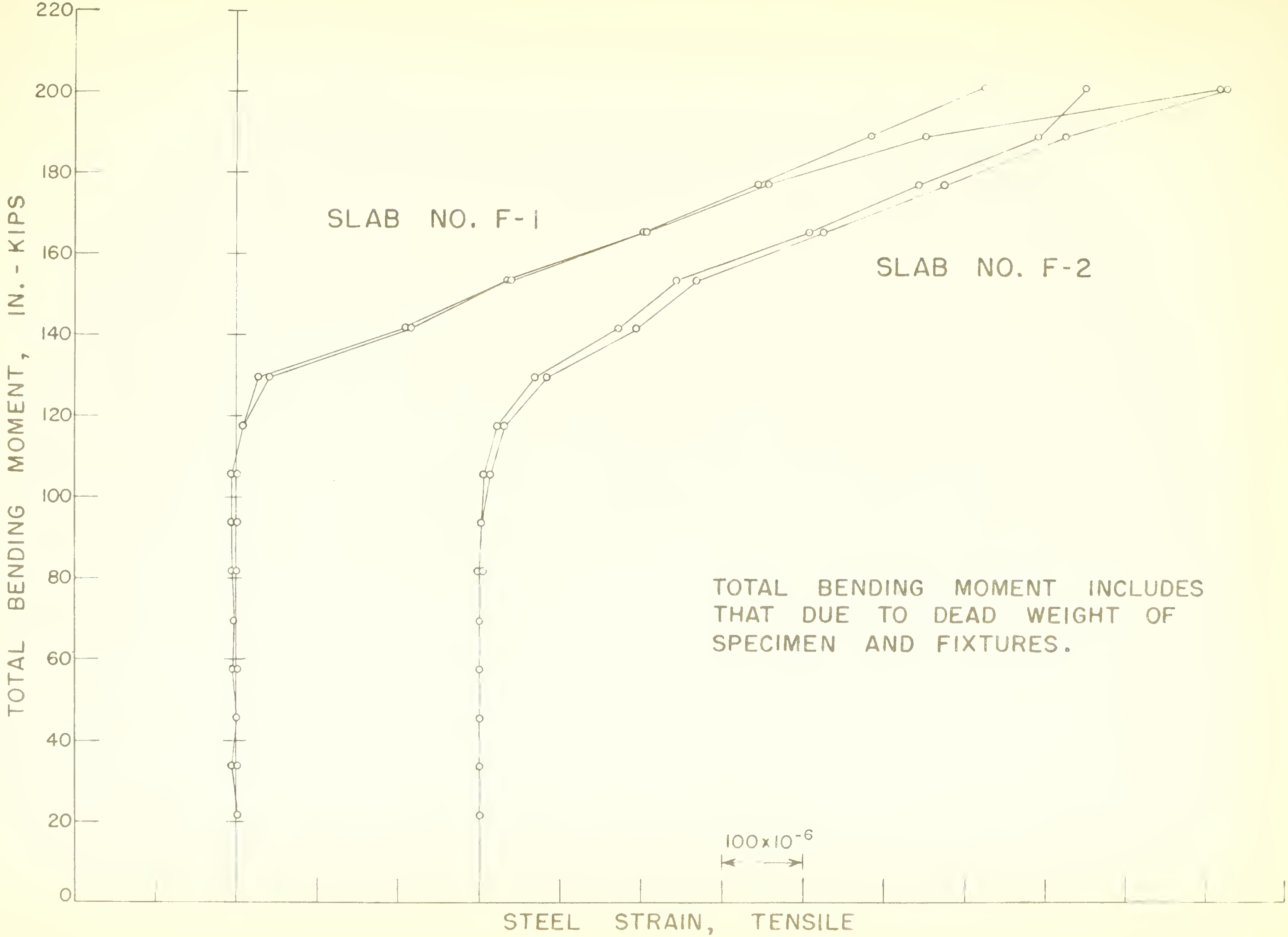
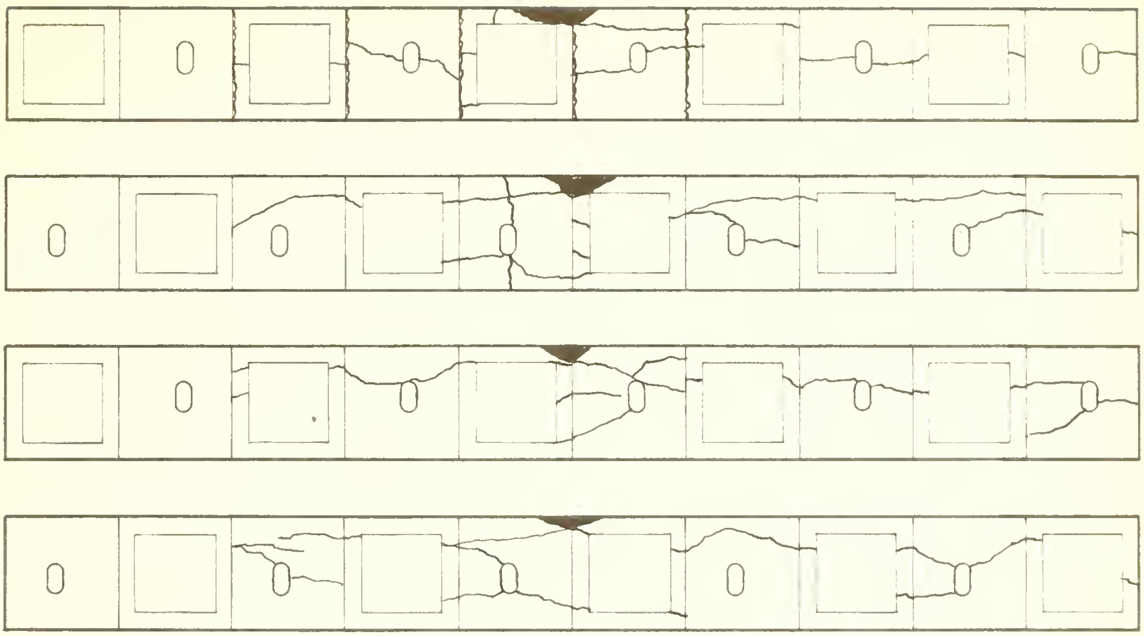
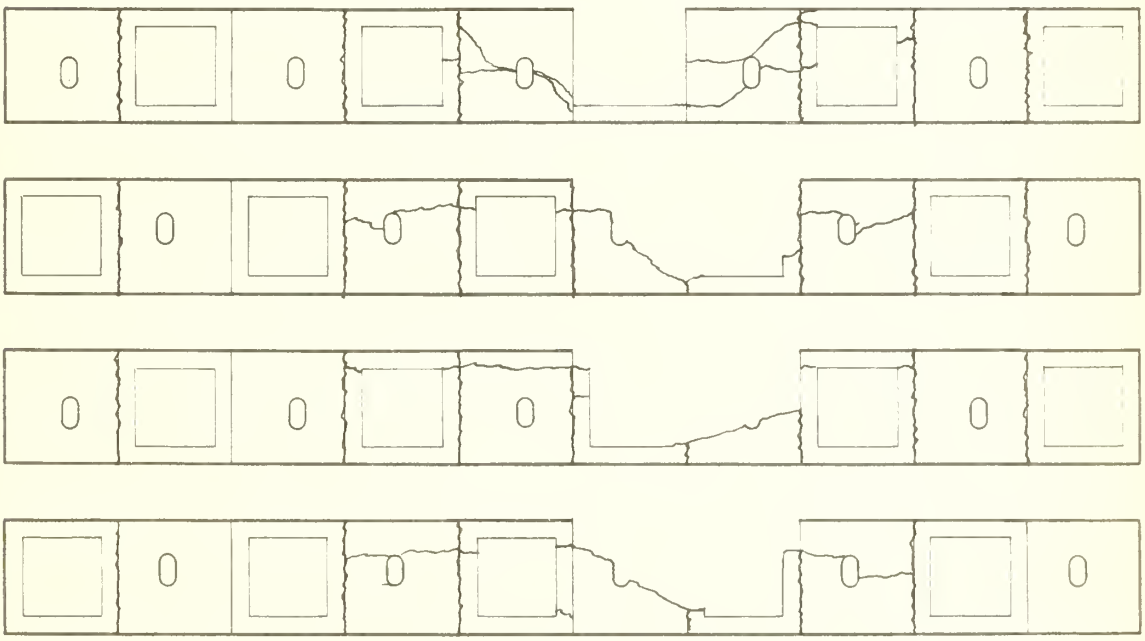


FIGURE 7. RELATIONSHIPS BETWEEN BENDING MOMENT AND STEEL STRAIN IN THE SLABS.



SLAB NO. F-1



SLAB NO. F-2

FIGURE 8. CRACK PATTERNS

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Sinclair Weeks, *Secretary*

NATIONAL BUREAU OF STANDARDS
A. V. Astin, *Director*

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