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Preliminary Investigation of Precast Prestressed Concrete Pavement

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PRELIMINARY INVESTIGATION OF PRECAST PRESTRESSED CONCRETE PAVEMENT

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BY

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This thesis is approved as a creditable, independent investigation by a candidate for the degree, Master of Science, and is acceptable as meeting the thesis requirements for this degree, but without implying that the conclusions reached by the candidate are necessarily the conclusions of the major department.

Thesis Adviser

Head of the Major Department

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INTRODUCTION

Merits of Prestressed Concrete

Prestressing is a simple method of introducing an initial compressive stress in any material possessing high compressive strength. This procedure enables the design engineer to better utilize the high compressive strength possessed by concrete and the exceptionally high tensile strength possessed by high strength steel. This means that a force tending to cause a tensile stress in a material must first overcome the compressive stress introduced during prestressing before subjecting the material to a tensile stress. Since concrete is about ten times stronger in compression than in tension, the logical design approach is to fully utilize this high compressive strength by combining it with a material possessing high tensile strength.

Historical Background

Prestressed concrete, as it is known today, is a relatively recent development. It was not until the development of high strength steel that concrete could be prestressed efficiently. One of the earliest large scale applications of prestressed concrete pavement was at Orly Airport near Paris, France. This test slab was poured in 1946 and gave satisfactory performance under test loads. Since that time the use of prestressed concrete pavements has been increasing. Military air fields include prestressed concrete pavement at Biggs Air Force Base

at El Paso, Texas and Patuxent River Naval Air Station[1]. A prestressed overlay slab[2] was used for the airport runway at San Antonio, Texas. At a site near Pittsburg, Pennsylvania, an 800 foot test section of prestressed concrete highway pavement[3] was constructed. The test results revealed exceptional load carrying capacity. The most recent prestressed pavement constructed is an airport pavement constructed at the Bonn-Cologne Airport[4] in Germany. The Bureau of Public Roads, the Missouri State Highway Commission, and the University of Missouri are now cooperating on a field study of 16 prestressed slabs to observe creep and warping under field conditions. A complete description of this field test was given at the 37th. annual meeting of the Highway Research Board.

These and other prestressed concrete pavement projects have shown that this method is a feasible approach to the current problem of providing an economical pavement for todays increasing wheel loads and traffic volumes. This method also appears to greatly reduce the problem of tension cracks caused by temperature changes or moisture.

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[1]Journal of American Concrete Institute, "Prestressed Pavement; a World View of It," vol. 30, p. 829-838, Detroit, Michigan, February, 1959.

[2]M. M. Lemcoe and C. H. Mohla, "Prestressed Overlay Slab for San Antonio Airport," Journal of American Concrete Institute, vol. 31, p. 25-35, Detroit, Michigan, July, 1959.

[3]American City, "Industry Experiments with Prestressed Concrete Pavements," Buttenheim Publishing Company, p. 11, New York, New York, September, 1957.

[4]Engineering News Record, "Germans Prestress a Concrete Runway for Jets," McGraw-Hill Publishing Company, vol. 168, p. 92, New York, New York, April 26, 1962.

Scope of Study

A study was initiated at South Dakota State College by Prof. Emil R. Hargett to determine the feasibility of the use of precast and prestressed concrete panels for highway and airport pavements. Professor Hargett proposed the production of precast and prestressed panel sections in a centrally located casting yard for post-tensioning at the job site. It was further proposed that the panel sections be post-tensioned in "pull up" spans of about 100 feet; but extending as a continuous post-tensioned pavement. The panel sections are then to be covered with a thin course of asphaltic concrete so as to smooth out the joints, provide the necessary crown, and protect the panel sections against weathering and impact loads. The individual panels are to have a device such as tongue and grove joints to provide the necessary vertical alignment and vertical shear resistance. Figure I shows a typical panel section.

The problems of casting and handling panel sections of this type were discussed with Gage Brothers Concrete Products in Sioux Falls, South Dakota. It was decided that this plan had merit and was worthy of laboratory investigation. Information regarding cost was also favorable. Gage Brothers demonstrated a keen interest in this study by furnishing six half scale panel sections free of charge for this laboratory investigation. Figure II shows the pouring of the panel sections and Figure III shows four of the panel sections after curing and prestressing.

It was the primary purpose of this study to determine the

structural behavior of the assembled panels. To achieve this, strains and deflections of the slab under various loads were recorded and analyzed in the laboratory investigation.

TEST PROCEDURE

Description of Half Scale Test Panels

The dimensions of the half scale test panels are 12' x 2' x 2". High strength concrete was used in casting the panels. This concrete had a 28 day standard test cylinder strength greater than 6000 psi. High strength $\frac{1}{4}$"-7 strand steel cables were used for prestressing. Design tension in the prestressing cables in both directions was 140,000 psi. Two strands were pretensioned in the lateral direction $\frac{1}{2}$ inch below the neutral axis of the cross section of the panel. They were spaced 12 inches apart and six inches from the panel edge. Twelve 5/8 inch longitudinal conduits were inserted in the panel at the time of casting for post-tensioning cables. The conduits were on the neutral axis of the cross section of the panel and extended the panel width. These conduits were spaced 12 inches apart starting six inches from the panel end. One inch by 13/16 inch inside diameter steel sleeves were centered on both ends of conduit $\frac{1}{2}$, $5\frac{1}{2}$, $6\frac{1}{2}$, and $11\frac{1}{2}$ feet from the panel end. As shown in Figure IV, these sleeves are recessed one inch into the panel edges at the said locations. The steel sleeves were replaced by 12' x 2" x $\frac{1}{4}$" steel bearing plates on one edge of the end panels. These plates receive the thrust of the anchorage cones when the panels are assembled and post-tensioned. Figure IV shows details of a test panel section.

Preparation for Loading

The six half-scale panels were aligned and supported $3\frac{1}{2}$ inches

off the laboratory floor at the ends and middle by 2" x 6" x 16' wood planks. The alignment shear keys, shown in Figure V, were then inserted and the steel prestressing strands were threaded through the 5/8 inch conduit. The panels were then post-tensioned longitudinally with a force resulting in 175,000 psi in each strand. The center strand was post-tensioned first. This was followed by post-tensioning the two outside strands. This procedure was then repeated. The remaining strands were post-tensioned alternating from one side of the center line to the other. This was done starting at the far strands and working toward the center. Figure VI shows a jack in position for post-tensioning a longitudinal strand. The middle support was then removed and the end supports were aligned six inches from the edge. This resulted in two opposite edges being simply supported in the longitudinal direction, with the remaining two edges free.

As loads would be placed along the longitudinal axis of symmetry, stresses resulting from the loads could be expected to form a mirror image about this axis. For this reason, strain gages were distributed more heavily on one side of the axis than the other. The strain gage distribution is shown in Figure VII.

Sixty SR-4 strain gages of type A-1 were placed in position on the carefully prepared top surface of the slab using Epoxy 150 cement. These are paperbacked wire gages of 13/16 inch length. They were then connected to a common ground and the remaining leads grouped in three groups of twenty. These three groups were labeled A, B, and C and numbered 1 through 20 for ease of reference. The gage labeling is

shown in Figure VII. Each group was connected to a Baldwin 20 point switch unit. As there were only two 20 point switch units available, only two groups were connected at one time during a loading series. Strains were measured by using a type L Baldwin strain indicator.

It was desired that the deflections under load be measured. For this reason a grid system, shown in Figure VIII, was developed using the paths of the prestressing strands. A black cross was painted on the surface to mark the location of strand crossings. An engineer's level was set up near one side of the slab and rod reading were taken to the nearest sixteenth of an inch during the loading series. A wooden cat walk was erected on rollers to facilitate rod readings on the interior of the slab. This gave access to the interior of the slab while not disturbing the slab under load.

Loading Procedure

The panel sections were assembled and post-tensioned under the loading frame in such a manner as to allow the loads to be placed anywhere along the longitudinal axis of symmetry. Figure VII shows the assembly of panel sections forming a 12' x 12' slab for testing. Since the edge supports of the slab were not connected to the loading frame, but rested on the concrete floor of the lab, the maximum load that could be applied was the weight of the loading frame. This dead load amounted to 8300 pounds and was considered sufficient for this project. The loading frame is shown in Figure IX.

The jack head was two inches in diameter. To prevent extreme

stress concentrations from developing due to the small area of the jack head, a 12 x 12 x 1/8 inch steel bearing plate was centered under the jack to distribute the load. This bearing plate and loading jack can be seen in Figure X.

Loads were applied from the loading console which gave the load in gage pressure. Actual load in pounds was 2.18 times the gage pressure. The console is partially shown in Figure XI.

A certain amount of deflection and strain was initially present due to the 25 pounds per square foot dead load of the slab. This initial condition was used as a base datum. The loading jack was placed over the center of the slab. Groups A and B strain gages were connected to the strain indicator. These strain gages provided adequate coverage for the half of the slab adjacent to the Baldwin switch unit. Initial level rod readings were taken of all 144 points to establish a datum before any loads were applied. During the test only a selected number of rod readings were taken to determine configuration. All 144 points were not read again until the yield point had been reached.

A loading series was first applied at the center of the slab. These loads were 872, 1744, 2180, and 2485 pounds. The procedure in this series was to take initial strain gage and level rod reading before applying the load, take the readings under load, and then take final readings after the load was released. Figure X shows this procedure in process. The object of this procedure was to observe any permanent deflection or yield resulting from the applied load.

The yield point was reached at 2485 pounds. This resulted in a

small permanent deflection of $\frac{1}{8}$ inch at the center of the slab. The loading jack was then transferred along the longitudinal axis to the edge of the slab, as shown in Figure XII. Strain gage group C was connected to the strain indicator in place of group A. Loads of 875, 1090, 1308, and 1526 pounds were applied. The loading procedure in this load series differed from the former in that the load was not released after each individual loading, but continued to the final load. Initial and finally readings showed that there was no permanent deflections resulting from this load series.

The loading jack was then moved along the longitudinal axis to an approximate quarter point and a single load of 1526 pounds was applied. This load did not result in any permanent deflection.

Another loading series was then applied at the center of the slab. These loads were 2616, 3052, and 3488 pounds applied in a continuous loading manner.

ANALYSES OF TESTS

The study on six half scale panels was designed to produce test data regarding the structural performance of this type pavement. This was done by gathering and analyzing data regarding the stress distribution and deflections of the slab under load. The analysis was to show whether the slab was acting as individual panels or acting as a complete unit. No attempt was made to analyze the stress and deflections caused by the dead load of the slab. This dead load would be nullified by ground support under actual pavement conditions.

It was originally intended to simply support the slab on its four corners. However, the resulting large deflections precluded this approach. It was then decided to simply support the slab along the two longitudinal edges. Since the strain gages were placed for a loading on corner support, they were not all in the most effective positions for edge supports. While many strain gages thus gave strains of little value, enough effectively placed strain gages remained to give an analysis of the stress distribution. The large deflection resulting from corner support was primarily due to the absence of initial negative moment in the longitudinal direction. No negative moment was present as the prestressing strands in this direction ran along the neutral axis of the cross-section. An initial negative moment was present to resist deflection in the lateral direction as the prestressing strands ran below the neutral axis of the cross-section.

Center Loading

A linear relationship was established between load and deflection throughout the loading range. It was noted at the yield point that the slope of the actual load verses deflection changed abruptly upward and continued as straight line. It thus appears that an elastic relationship exists between load and deflection beyond the yield point in prestressed concrete slabs. The comparison of theoretical and actual loads at equal deflections is shown in Figure XIII. It is noted that the actual loads are higher than the theoretical loads for the same deflections in the region below the yield point. This was expected since the "beam method" considers only part of the slab in load calculations. The theoretical analysis in appendix "A" discusses this "beam method" in handling prestressed slabs. It is noted in Figure XIII that the two curves cross at a load slightly higher than the yield point load. The two curves diverge with the theoretical load becoming increasingly greater as compared to the actual load for equal deflections.

Table 1 tabulates the stresses at the strain gage locations for various loads. Compressive stresses were present in the top fibers in the lateral direction throughout the slab. The lateral stresses along the longitudinal center line decreased in magnitude as the distance from the loading point was increased. The stress magnitude also decreased as the distance increased laterally from points on the longitudinal center line. The compressive stresses indicate a positive moment was present in the top fibers of all panels in the lateral direction. The

moment was maximum at the loading point. This was further confirmed by a plot of the curvature of the lateral center strip under load as shown in Figure XIV. This curvature was concave indicating positive moment was present in the top fibers of the slab in the lateral direction.

Tension stresses were present in the top fibers in the longitudinal direction throughout the slab. These stresses were of much smaller magnitude than the lateral stresses. The stress values along the longitudinal center line decreased as the distance from the loading point increased. The magnitude of the stresses also decreased as the distance increased laterally from points on the longitudinal center line. The tension stresses indicate a negative moment was present in the top fibers of all the panels in the longitudinal direction. This was further confirmed by a plot of the curvature of the center longitudinal strip under load as shown in Figure XV. This curvature was convex indicating a negative moment was present in the top fibers of the slab in the longitudinal direction.

It should be noted that the theoretical load calculations assumed that the center longitudinal strip was acting as a simply supported beam. This was not the case according to the actual load deflection curve. The longitudinal strip acted as two cantilevers anchored at the free edge of the slab. The shape of the load deflection curve was convex from the edge support to the center load. Since the center load was directly over a joint, it is believed that this joint acted as a hinge which was responsible for the cantilever type action. Handling this strip as two cantilevers in the theoretical calculations would have

decreased the theoretical load P somewhat.

It was noted that when the load approached the yield point for the first time, a minor amount of flaking occurred at the panel joints. It was first thought that general failure was beginning to occur along these joints. However, since the stresses at right angles to the joints were tension stresses, this could not be general failure as only compressive stresses would cause crushing in the top fibers in this direction. It was concluded that the flaking was caused by local stress concentrations due to a small amount of non-uniformity in the joints.

Edge Load and Quarter Point Load

It was desired to have a check on the basic stress and deflection pattern established by the center loading. For this reason the loading device was transferred along the longitudinal center line to the edge and quarter points. Stresses at the strain gage locations for various edge loads are tabulated in Table 2. The maximum edge load of 1526 pounds caused no permanent deflection. The stresses and deflections formed the same basic pattern as with the center loading. Figure XVI shows the deflection curve of the longitudinal center strip under an edge load of 1526 pounds.

A single 1526 pound load was placed at the quarter point. The stresses at the strain gage locations are tabulated in Table 2. The load caused no permanent deflection in the slab. The stress and deflection patterns were similar to the patterns obtained from the center loading. Figure XVII shows the deflections of the center longitudinal strip due to the 1526 pound quarter point load.

The highest compressive stresses recorded approached 2000 psi. These stresses corresponded to a center load of 3488 pounds and a net center deflection of 2.69 inches. The highest tension stresses were less than half the compressive stresses. Failure could then be expected to occur due to the concrete crushing in the top fibers along the longitudinal axis. It should be again noted that the stresses recorded do not include dead load stresses or stresses due to the prestressing.

Three of the slabs were cracked during shipping to the laboratory from the casting yard. These cracks were on the top side and didn't appear to affect the experiment as the top fibers were in compression in the direction of the cracks.

CONCLUSIONS

The following conclusions have been drawn as a result of the testing and study described in this investigation.

1. The assembled panels act structurally as a slab and not as individual panels.

2. The slab appears to behave elastically beyond the yield point.

3. Due to the high magnitude of loads applied in this test without general slab failure, it appears that the full scale panel thickness may be reduced.

4. The flexibility possesses by this type of pavement will allow it to resist deflections far greater than the deflections that are common to rigid or flexible pavements.

5. From the structural standpoint these panel sections hold promise for high type highway or airport pavement.

6. The stress distribution indicates that the slab section will greatly reduce stress intensity in base course and sub-base by distributing the wheel load over a large area.

7. The use of a central casting yard will allow production of panels to be independent of weather. More rigid control of concrete mix will also result.

8. The smooth continuous paving surface will give greater riding comfort.

9. The elimination or reduction of temperature and moisture cracks will greatly reduce maintenance costs.

10. This type of construction will have the same effect as lengthening the construction season since the casting of the panel sections may be carried on during winter months.

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1. Journal of American Concrete Institute, "Prestressed Pavement; a World View of It," vol. 30, p. 829-838, Detroit, Michigan, February, 1959.

2. M. M. Lemcoe and C. H. Mohla, Prestressed Overlay Slab for San Antonio Airport," Journal of American Concrete Institute, vol. 31, p. 25-35, Detroit, Michigan, July, 1959.

3. American City, "Industry Experiments with Prestressed Concrete Pavements," Buttenheim Publishing Company, p. 11, New York, New York, September, 1957.

4. Engineering News Record, "Germans Prestress a Concrete Runway for Jets," McGraw-Hill Publishing Company, vol. 168, p. 92, New York, New York, April 26, 1962.

5. G. L. Rogers, "Validity of Certain Assumptions in Mechanics of Prestresses Concrete," Journal of the American Concrete Institute, vol. 49, p. 317-320, Detroit, Michigan, December, 1953.

6. T. Y. Lin, "Design of Prestressed Concrete Structures," John Wiley & Sons, Inc., p. 342, New York, New York, November, 1959.

7. E. P. Popov, "Mechanics of Materials," Prentice-Hall, Inc., p. 435, Englewood Cliffs, New Jersey, January, 1958.

8. S. Timoshenko and S. Woinowshy-Krieger, "Theory of Plates and Shells," McGraw-Hill Book Company, Inc., p. 214-218, New York, New York, 1959.

APPENDIX A

Design Data:

The concrete mix consisted of 704 pounds of cement, 1317 pounds of sand, 1833 pounds of aggregate, and 32 gallons of water per cubic yard. This mix yielded a 28 day cylinder strength of 6000 psi. The resulting modulus of elasticity was then:

$$E = 18,000,000 + 500(6000) = 4.8 \times 10^6 \text{ psi}$$

The $\frac{1}{4}$ inch, seven strand steel prestressing cable was initially tensioned to 175,000 psi; but losses reduced design tension to 140,000 psi.

Design Procedure:

A. Post tensioning along slab:

Total area of cross-sections = $12 \times 12 \times 2 = 288$ in.^{2}

Using 12 - $\frac{1}{4}$ inch strands

Area of steel $A_s = 12 \times 0.0356$ $= 0.4272$ in.^{2}

Total final force = $0.4272 \times 140,000$ = 59808 lbs.

Net area = $288 - 12(0.31)$ = 284.28 in.^{2}
using 12 conduits 5/8 inch in diameter

Stress in concrete (based on net area)

$$= \frac{59808}{284.28} = 210 \text{ psi.}$$

B. Pretensioning across the slab:

Using 2 - $\frac{1}{4}$ inch strands

Gross area of cross-section = 24×2 = 48 in.^{2}

Net area of cross-section = $48 - 24 \times 5/8$ = 33 in.^{2}

Total final force in the two strands
= $0.0356 \times 2 \times 140,000$ = 9968 lbs.

Stress in concrete (based on net area)

$$= \frac{9968}{33} \qquad = -302 \text{ psi.}$$

Moment on cross-section due to $\frac{1}{2}$ inch eccentricity

$$= 9968 \times 0.50 \qquad = 4984 \text{ in-lbs.}$$

Stresses due to moment

At top fibers $= \dfrac{4984 \times 1 \times 12}{24 \times 2 \times 2 \times 2}$ $= +\ 312$ psi.

At bottom fibers $= -\ 312$ psi.

Weight of panel $= 50$ lbs/ft.

The panels are supported 6 inches in from the ends.

Maximum moment due to dead load

$$= \frac{Wl^2}{8} = \frac{50 \times 121 \times 12}{8} \qquad = 9070 \text{ in-lbs.}$$

Stress due to dead load

At top fibers $= \dfrac{9070 \times 1 \times 12}{24 \times 8}$ $= -\ 566$ psi.

At bottom fibers $= 566$ psi.

Resultant stress

At top fibers $= -\ 556$ psi.

At bottom fibers $= -\ 58$ psi.

Theoretical Analysis:

Theoretical analysis has been done on a similar slab[5] loaded under somewhat different conditions in the past. In this test, however, the load was not taken beyond the yield point. In the course of loading this slab configuration, the yield point was greatly exceeded. For most

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[5] G. L. Rogers, "Validity of Certain Assumptions in Mechanics of Prestressed Concrete," Journal of the American Concrete Institute, vol. 49, p. 317-320, Detroit, Michigan, December, 1953.

materials, this would constitute failure. Tension failures in prestressed concrete does not necessarily constitute a complete failure. It was assumed, for ease of calculation, that the slab would behave elastically in the region beyond the yield point. Test results will indicate if this is a proper assumption.

For a prestressed slab, it is possible to treat strips of the slab as beams and compute the theoretical deflection or loads[6]. Two strips, two foot wide, were considered as acting as beams. These strips were in the longitudinal and lateral directions, and crossed at the center of the slab as shown in Figure XVIII. As previously stated, deflection reading were taken initially and during each load. These deflections are used to calculate the theoretical load.

Referring to Figure XVIII, consider beam GF simply supported with a concentrated load P' at the center. It thus follows[7] that

$$\Delta_{GF} = \frac{P' l^3}{48\ EI} \qquad \text{So,} \qquad P' = \frac{48\ EI}{l^3}\ (\Delta_{GF})$$

where: $E = 4.8\ (10^6)$ psi (previously calculated)

$$l^3 = 12^3\ (12^3) \text{ in.}$$

$$I = \frac{bh^3}{12} = \frac{24(2)^3}{12} = 16 \text{ in.}^4$$

Since these values remain constant for different P's

$$K_{GF} = \frac{48(418)\ 10^6(16)}{12^3(12^3)} = 1234.57$$

$$P' = K_{GF}\ \Delta_{GF} = 1234.57\ \Delta_{GF} \text{ lbs.}$$

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[6]T. Y. Lin, "Design of Prestressed Concrete Structures," John Wiley & Sons, Inc., p. 342, New York, New York, November, 1959.

[7]E. P. Popov, "Mechanics of Materials," Prentice-Hall, Inc., p. 435, Englewood Cliffs, New Jersey, January, 1958.

The deflection Δ_{GF} is the net deflection between the edges and the center of beam GF.

Referring to Figure XVIII, beam AL is simply supported with a centered concentrated load P".

$$P'' = \Delta_{AL} \frac{48\ EI}{l^3}$$

$$K_{AL} = \frac{48\ (4.8)\ 10^6\ (16)}{11^3\ (12^3)} = 1602.80$$

$$P'' = K_{AL}\ \Delta_{AL} = 1602.80\ \Delta_{AL}$$

with Δ_{AL} being the net deflection of beam AL. The net deflections for the various center loads are tabulated in Table 1, and Figure XIII correlates theoretical and actual loads.

No elastic support was derived from the 12' x 2" x ¼" steel bearing plates shown in Figure IV.

γ = flexural rigidity[8] of the bearing plate. As γ approaches ∞ the supporting beam becomes absolutely rigid. As γ approaches 0 the beam gives no support.

$$I = \frac{bh^3}{12} = \frac{1\ (2)^3}{4\ (12)} = .167$$

μ = Poisson's ratio = .25

a = 12 ft.

$$\gamma = \frac{12\ (1 - \mu^2)\ I}{h^3 a}$$

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[8]S. Timoshenko and S. Woinowshy - Krieger, "Theory of Plates and Shells," McGraw-Hill Book Company, Inc., p. 214-218, New York, New York, 1959.

$$\gamma = \frac{12\ (.9375)\ .167}{8\ (144)} = .00163$$

The beam action resulting from the bearing plates is not of major significance.

APPENDIX B

Table 1. Stresses at Strain Gage Locations for Center Loads

	Set A loads (pounds)				Set B loads (pounds)						
Gage Number	872	1744	2180	2485	872	1744	2180	2485	2616	3052	3488
1	+ 91	+ 19	- 91	+ 58	-168	-431	- 475	- 630	- 730	- 926	-1132
2	+ 14	+ 10	+ 35	- 24	+ 78	+ 10	+ 86.4	+ 125	+ 72	+ 144	+ 240
3	+ 34	+ 58	- 48	-1460	-202	+182	- 77	- 614	- 686	-1480	-3494
4	----	----	----	----	- 58	- 58	- 10	+ 10	----	----	----
5	- 19	+ 29	+ 53	+ 67	-278	-341	- 426	- 538	- 566	- 801	-1122
6	- 5	- 43	+ 86	-1406	+ 86	- 58	- 10	- 14	- 14	+ 106	+ 298
7	- 10	+ 10	+ 19	+ 24	-355	-715	- 855	-1060	-1058	-1038	-1009
8	- 96	-130	-115	- 326	+ 62	+ 72	+ 192	+ 374	+ 408	+ 576	+1019
9	+ 14	+ 38	+ 58	-1298	-369	-619	- 686	- 700	- 600	- 394	- 14
10	- 10	+ 43	+ 19	-1575	- 34	- 48	- 5	- 14	- 53	- 10	+ 53
11	- 81	-211	-202	- 389	-202	-393	- 418	- 312	- 288	- 230	+ 101
12	+ 24	+ 19	+ 58	+ 96	- 91	-182	- 144	- 250	- 308	- 331	- 355
13	-144	-298	-326	- 480	-460	-1210	-1259	-1685	-1747	-1910	-1830
14	+ 10	0	- 10	- 10	+ 62	+ 96	+ 187	+ 480	+ 619	+ 864	+1119
15	-106	-163	-173	- 134	-288	-585	- 644	-1010	-1095	-1382	-1820
16	- 43	- 96	-130	- 202	+ 62	+ 30	+ 86	+ 86	+ 91	+1297	+ 226
17	-173	-322	-350	- 532	-230	-308	- 408	- 580	+ 581	- 764	-1009
18	+ 48	- 53	+ 86	+ 139	+ 62	+240	+ 82	+ 24	+ 53	+ 53	+ 67
19	- 58	-192	-163	- 302	-264	-543	- 620	- 935	- 993	-1262	-1550
20	+ 38	+ 24	+ 58	+ 58	+ 86	+ 48	+ 120	+ 86	+ 134	+ 115	+ 96

\+ tension stresses
\- compression stresses

Table 1. (Continued)

Gage Number	Set C loads (pounds) 2616	3052	3488
1	- 33.6	+ 10	- 5
2	+ 5	+ 62	+ 96
3	-1420	-1810	-1810
4	+ 202	+ 230	+ 192
5	- 485	- 672	- 379
6	- 14	+ 129	+ 422
7	- 29	- 5	+ 5
8	0	- 91	+ 585
9	- 96	- 202	+ 245
10	- 384	- 538	- 240
11	- 48	- 53	- 72
12	- 58	- 72	- 77
13	- 480	- 686	- 931
14	+ 62	+ 864	+ 101
15	-1609	-2030	-2420
16	+ 77	+ 110	+ 120
17	- 10	+ 10	+ 499
18	+ 173	+ 182	+ 178
19	+ 34	+ 53	+ 561
20	+ 48	+ 53	+ 96

+ tension stresses
- compression stresses

Table 2. Stresses at Strain Gage Locations for the Edge Loads and Quarter Point Loads

Gage Number	Stress due to edge loads: Set B loads (pounds) 872	1090	1308	1526	Set C loads (pounds) 872	1090	1308	1526	Stresses due to quarter point loads: Set B loads (lbs) 1526	Set C loads (lbs) 1526
1	-288	- 96	-480	-575	- 96	- 106	- 154	- 168	- 192	- 82
2	+ 29	+ 5	- 14	- 10	+ 19	+ 10	+ 10	+ 10	+ 96	- 178
3	-278	-312	-312	-192	-192	- 302	- 67	- 48	- 344	-1297
4	---	---	---	---	+ 34	+ 14	+ 43	+ 53	---	+ 178
5	-264	-355	-427	-490	- 82	- 125	- 154	- 197	- 298	- 173
6	+ 43	+ 10	+ 5	+ 14	+ 43	+ 34	+ 38	+ 43	+ 19	+ 43
7	-312	-413	-505	-562	+ 14	- 29	- 29	- 29	- 442	+ 48
8	+115	+115	+144	+158	+ 29	- 19	+ 24	+ 24	+ 173	+ 682
9	-278	-374	-451	-509	+115	+ 130	+ 182	+ 235	- 442	- 235
10	+106	+ 91	+101	+120	- 53	- 96	- 106	- 144	+ 149	+ 259
11	-173	-259	-312	-360	+154	+ 144	+ 158	+ 197	- 365	+ 67
12	+ 67	+ 43	+ 48	+ 48	---	---	---	---	+ 72	---
13	-389	-552	-663	-806	- 96	- 149	- 202	- 273	- 835	- 178
14	+120	+106	+120	+130	+ 48	+ 43	+ 39	- 48	+ 240	+ 53
15	-226	-350	-432	-519	-815	-1138	-1472	-1910	- 562	- 556
16	+ 34	- 5	- 10	- 5	+ 10	+ 24	+ 48	+ 10	+ 77	+ 62
17	-158	-226	-269	-312	0	- 48	- 48	- 48	- 302	+ 28
18	+ 10	- 34	- 38	- 14	+ 29	- 10	0	0	+ 53	+ 110
19	-250	-355	-441	-528	0	- 48	- 43	- 34	-1075	+ 336
20	+ 24	- 14	+ 10	- 19	+ 19	- 29	- 14	- 14	+ 264	+ 53

+ tension stresses

- compression stresses

Table 3. Comparison of Theoretical and Actual Loads at Equal Deflections

Net deflections under load (inches)		Theoretical load (pounds)			Actual load (pounds)
beam GF	beam AL	P'	P"	P = P" + P'	
---	.125	---	---	---	872
.135	.531	167	852	1019	1744
.172	.625	212	1002	1214	2180
.281	1.250	347	2004	2351	2485
.313	1.594	386	2555	2941	2616
.344	1.719	424	2755	3179	3052
.531	2.688	656	4308	4963	3488

APPENDIX C

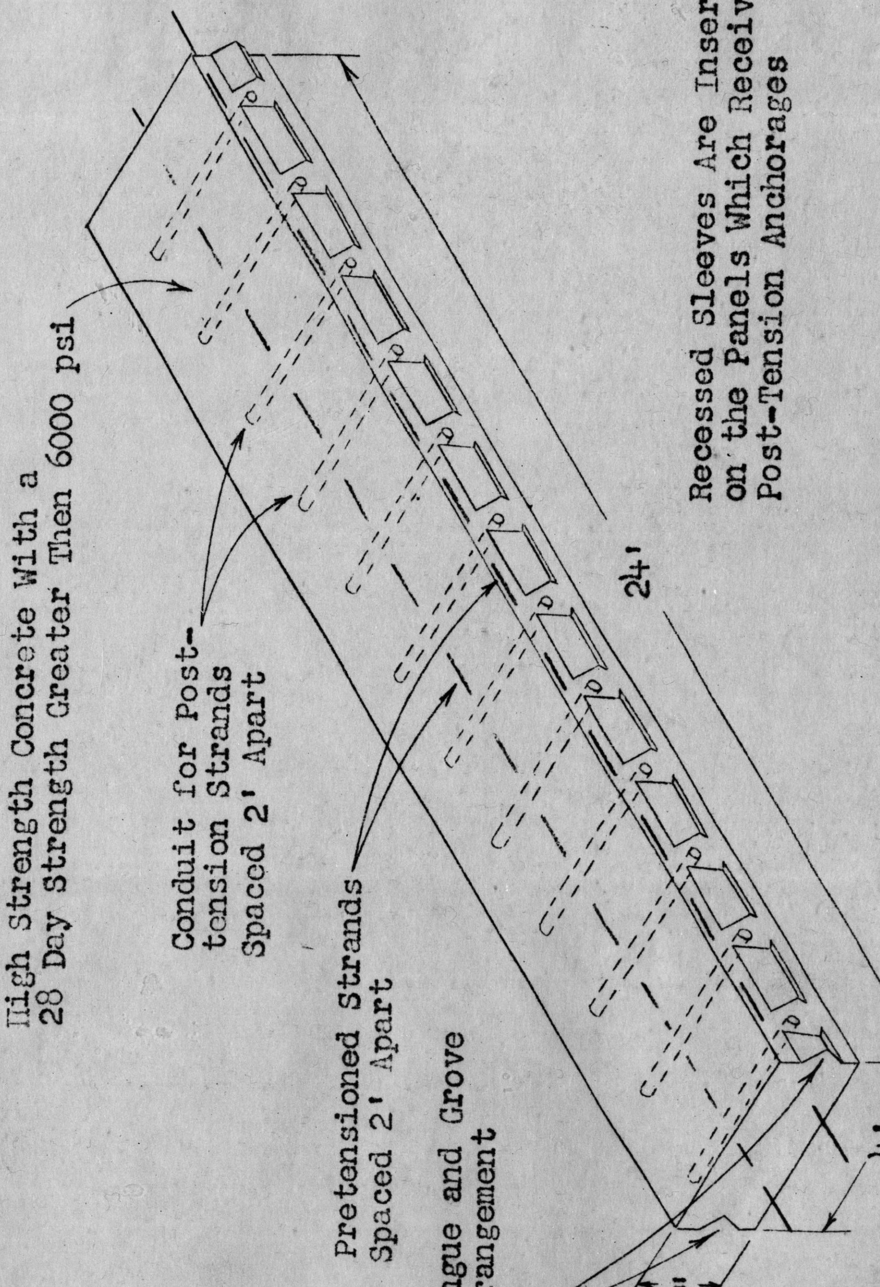


Figure I. Full Scale Panel

Figure II. Pouring Test Panels.

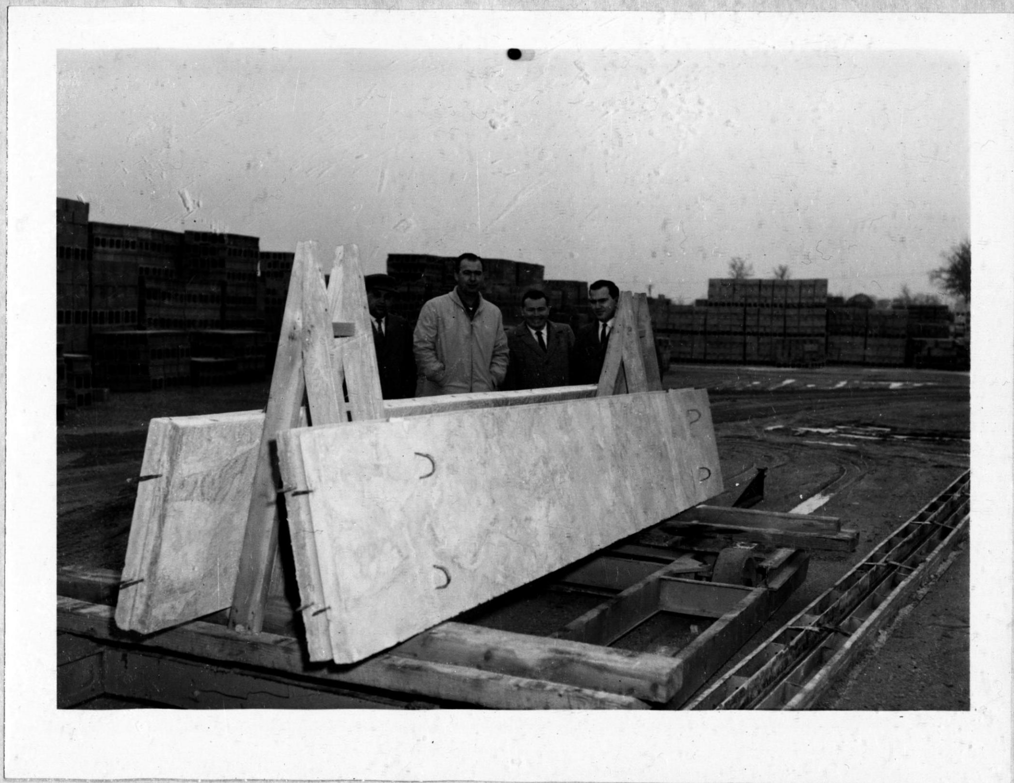


Figure III. Four Test Panels After Curing.

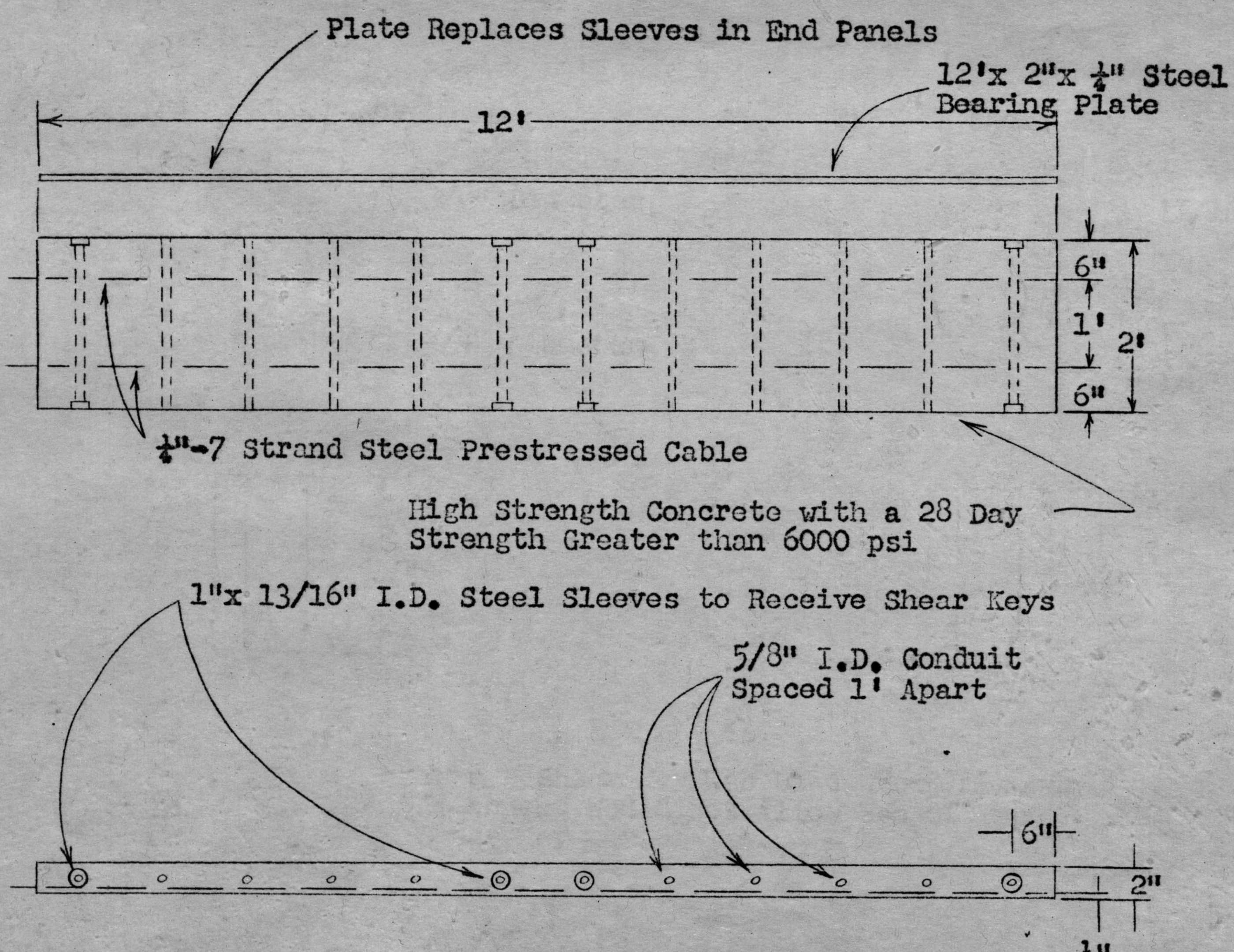
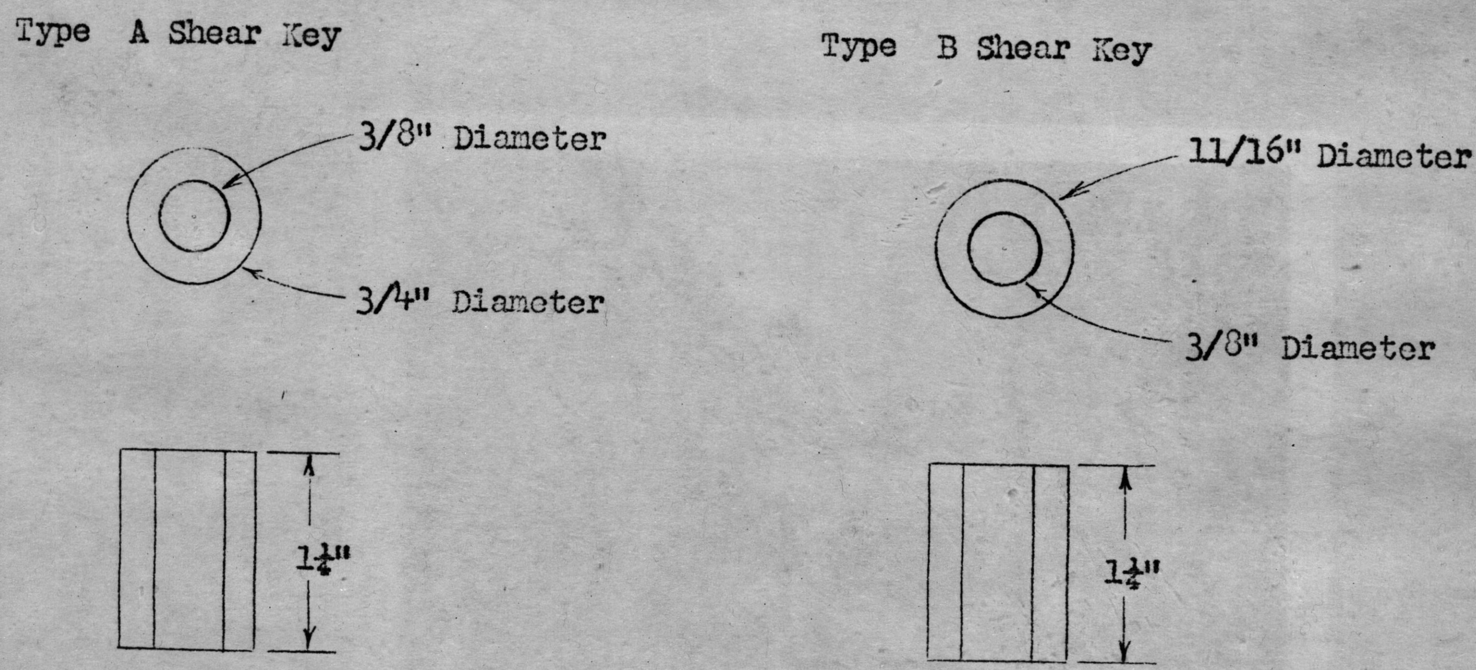


Figure IV. Half Scale Test Panel



Type B Shear Key Was Used When Improper Alinement Would Not Allow Use of Type A Shear Key

Figure V. Steel Alignment Shear Key

Figure VI. Post-Tensioning a Longitudinal Strand.

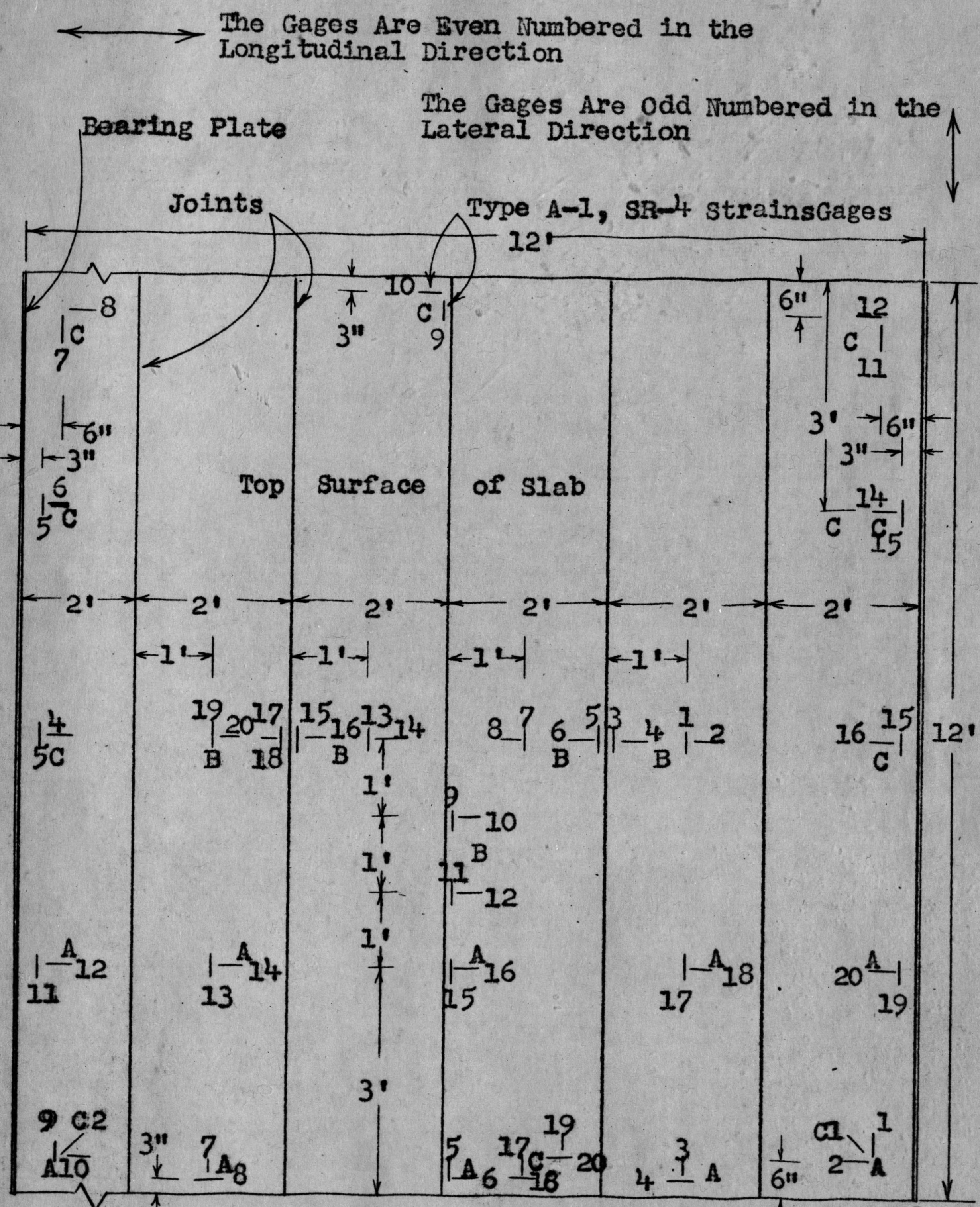


Figure VII. Strain Gage Distribution

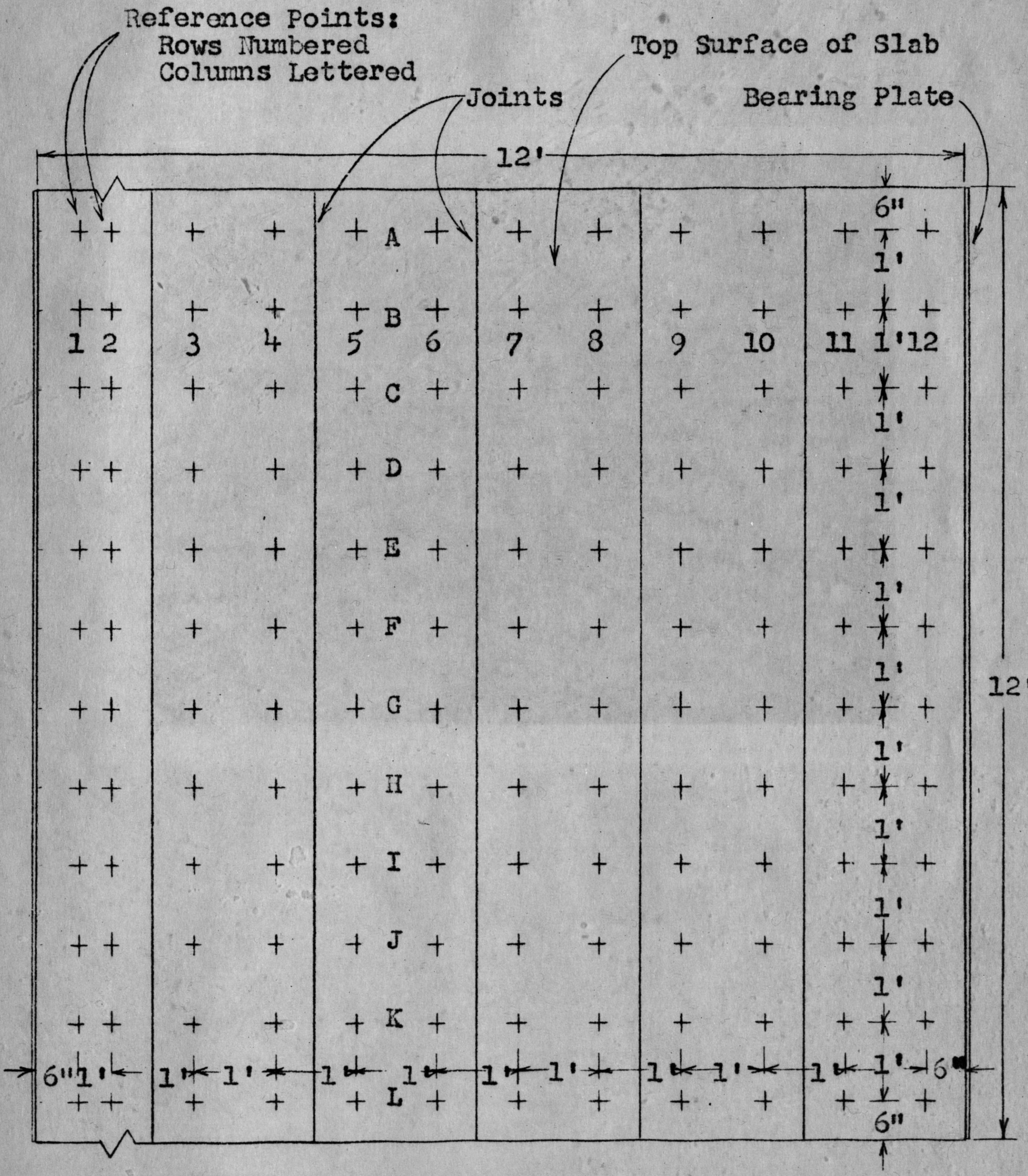


Figure VIII. Reference Points for Deflection Measurements

Figure IX. Loading Frame.

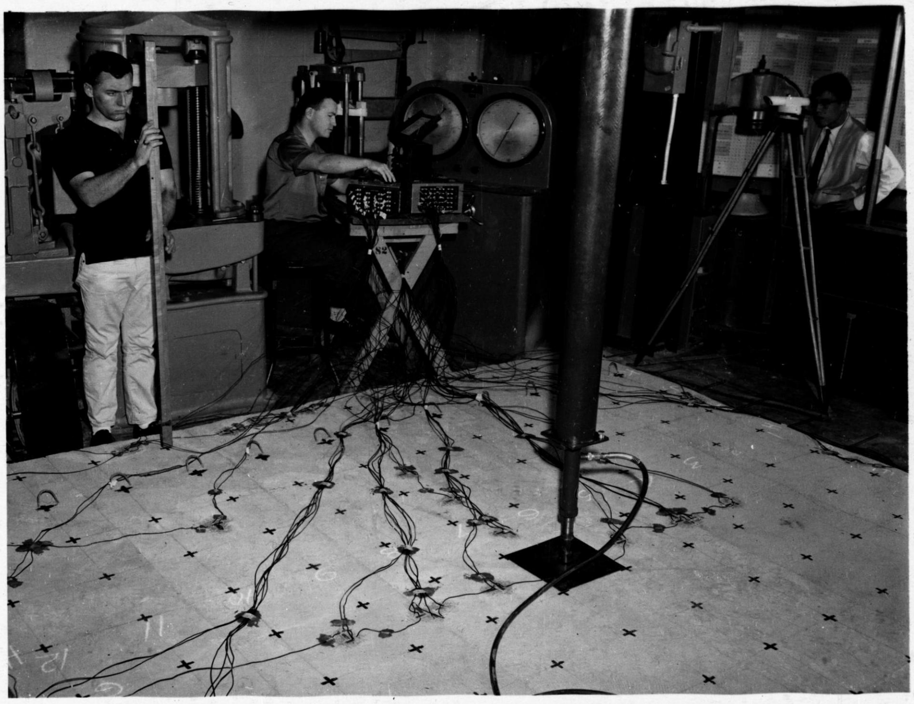


Figure X. Data Being Collected and Recorded.

XI. Loading Console.

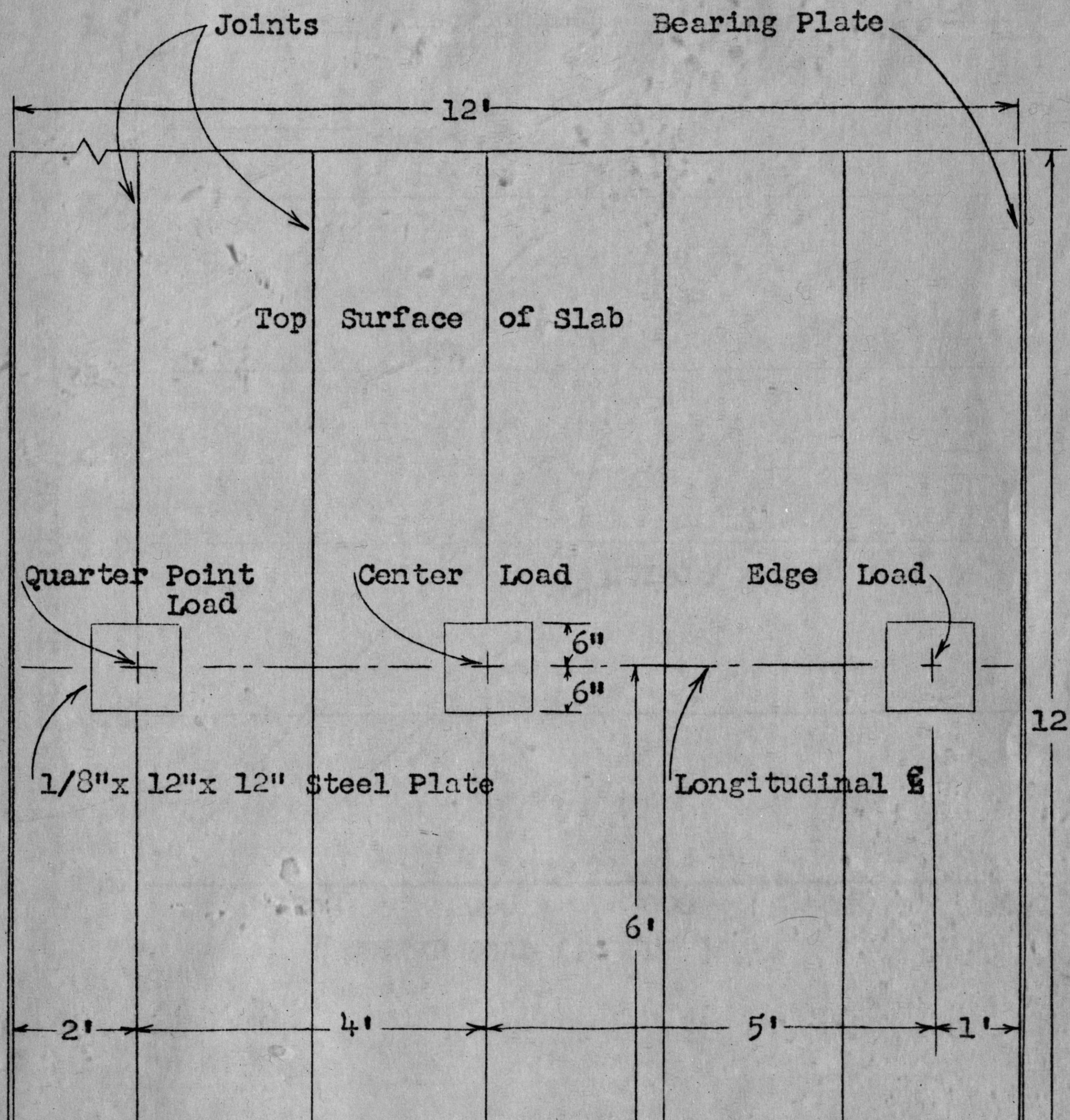


Figure XII. Loading Positions on the Slab

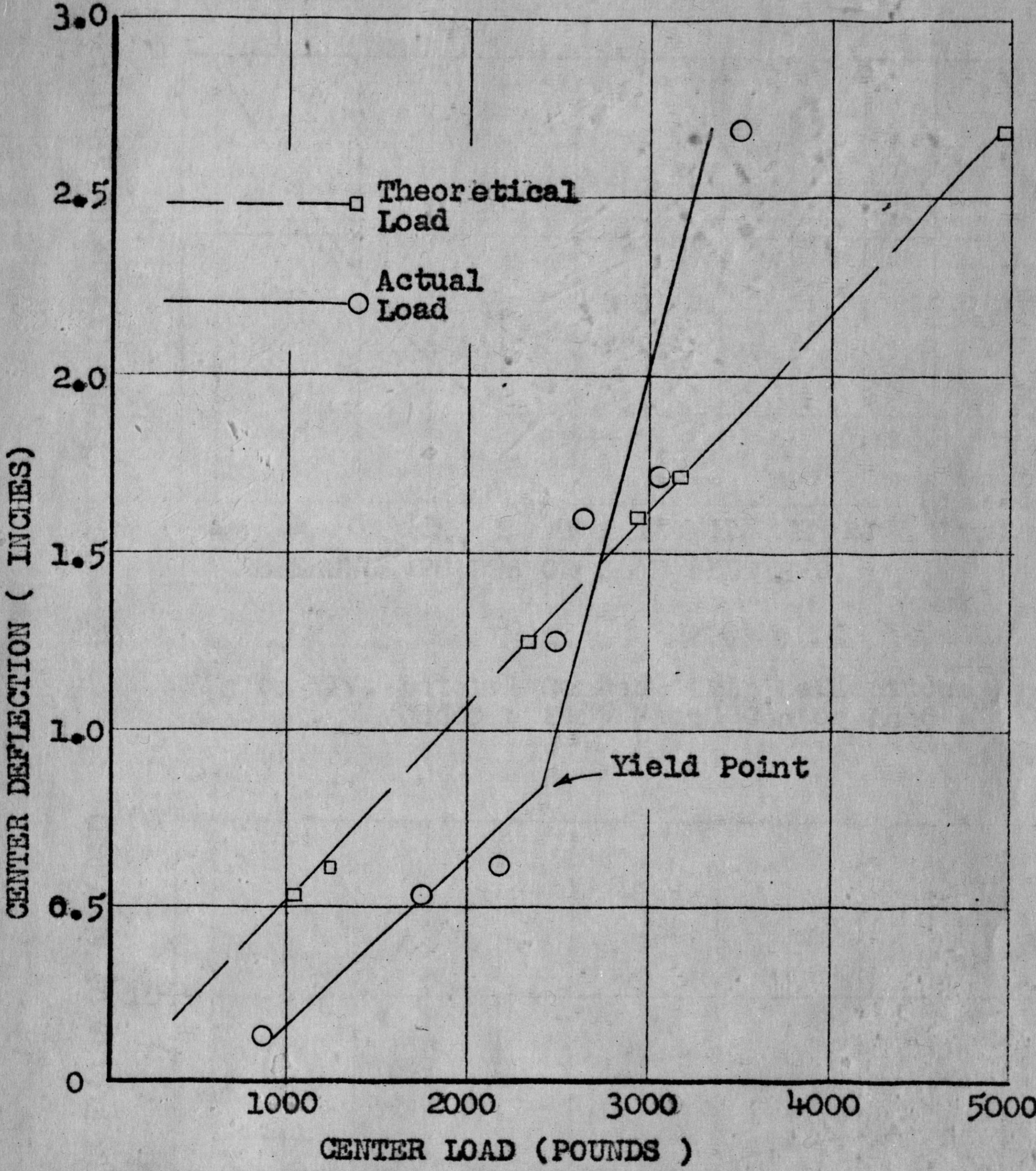


Figure XIII. Comparison of Theoretical and Actual Loads at Equal Deflections

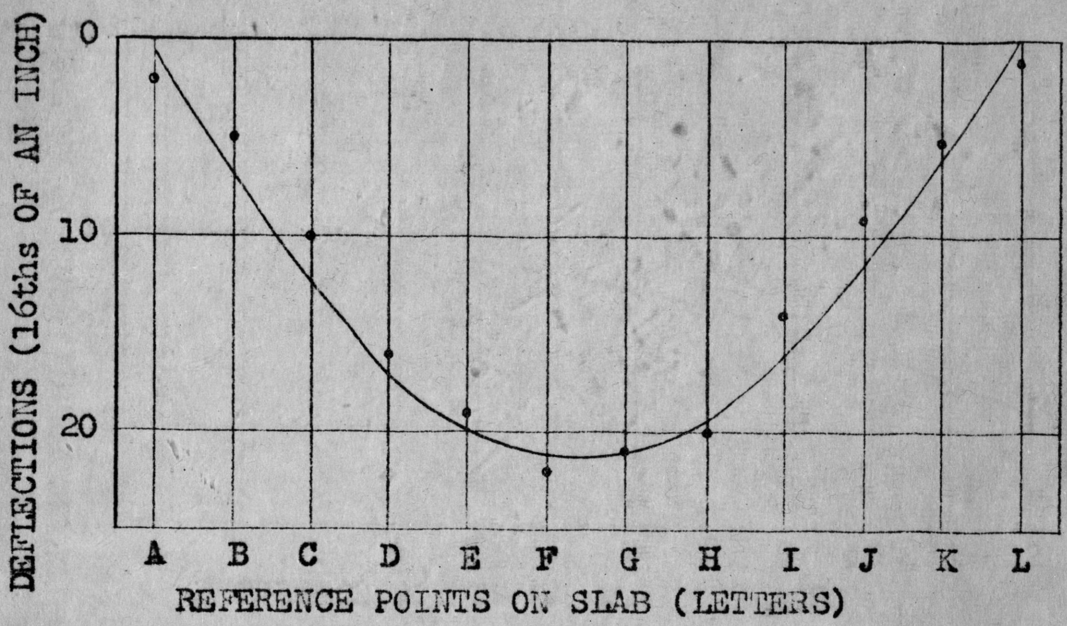


Figure XIV. Lateral Center Strip Deflections
Under a 2485 Pound Center Load

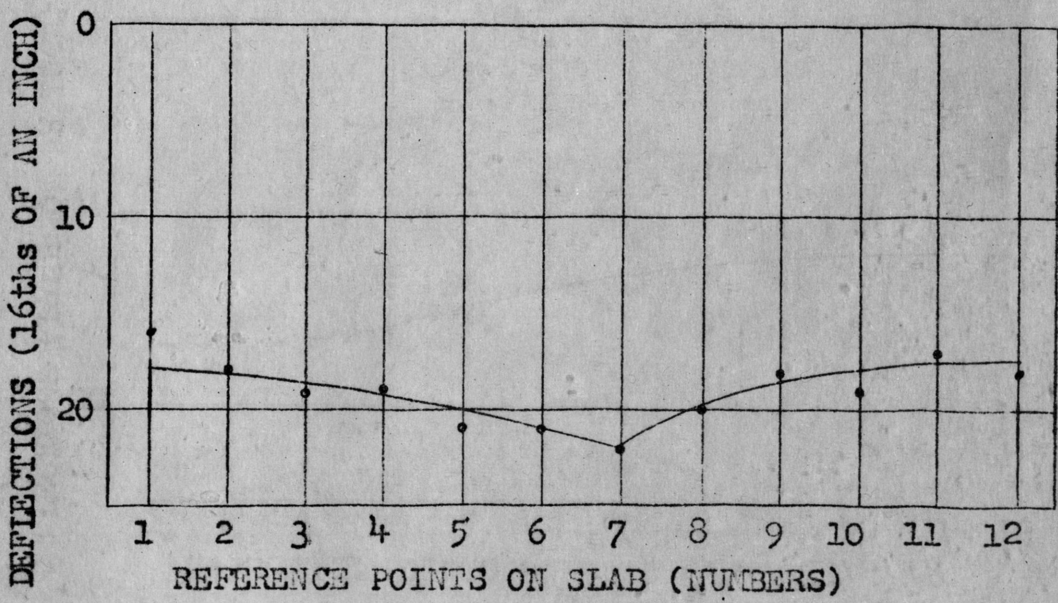


Figure XV. Longitudinal Center Strip Deflections
Under a 2485 Pound Center Load

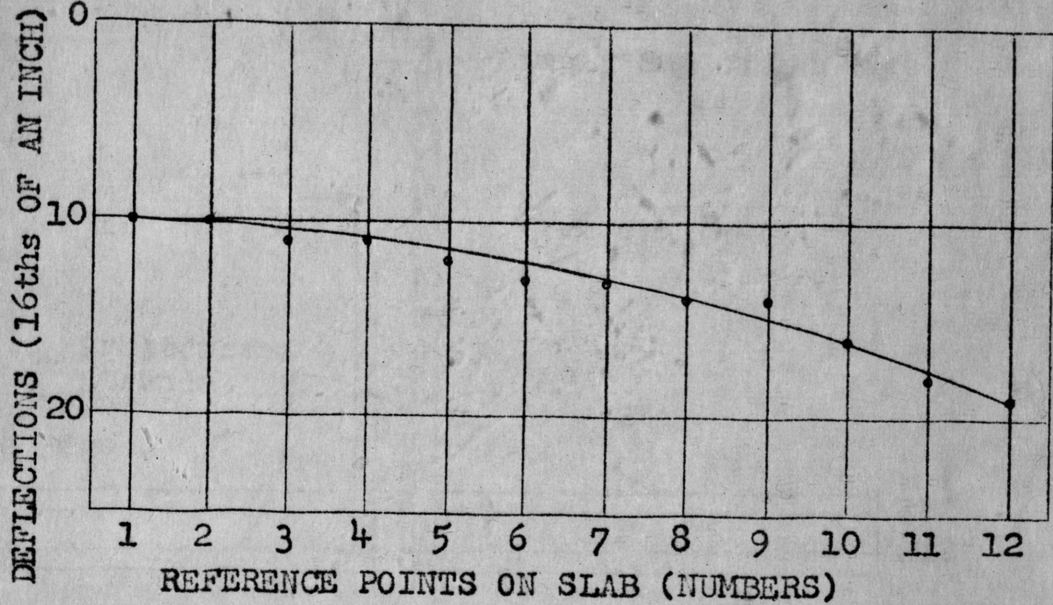


Figure XIV. Longitudinal Center Strip Deflections Under a 1526 Pound Edge Load

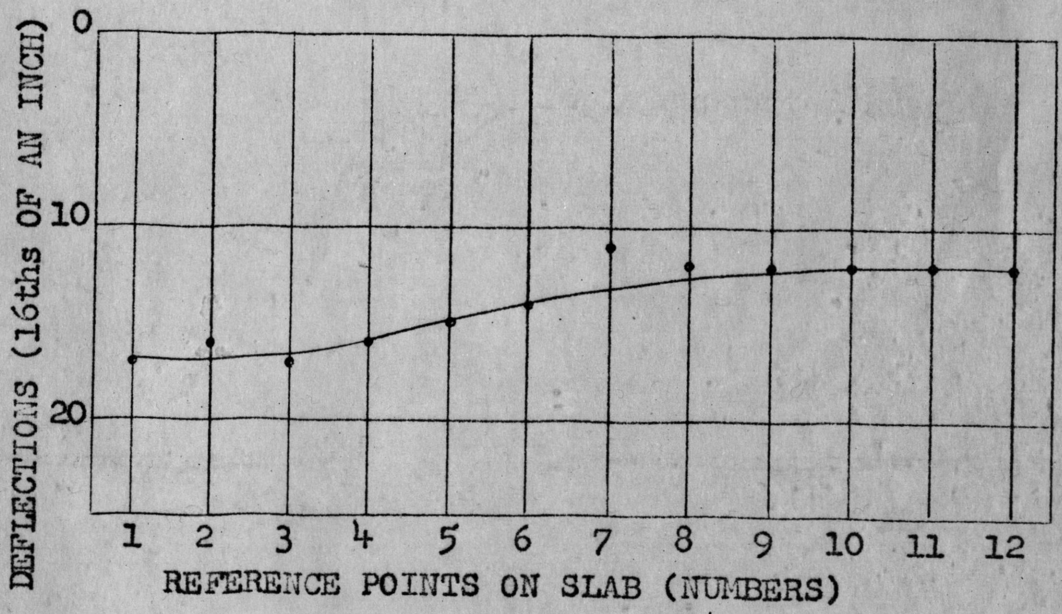


Figure XVII. Longitudinal Center Strip Deflections Under a 1526 Pound Quarter Point Load

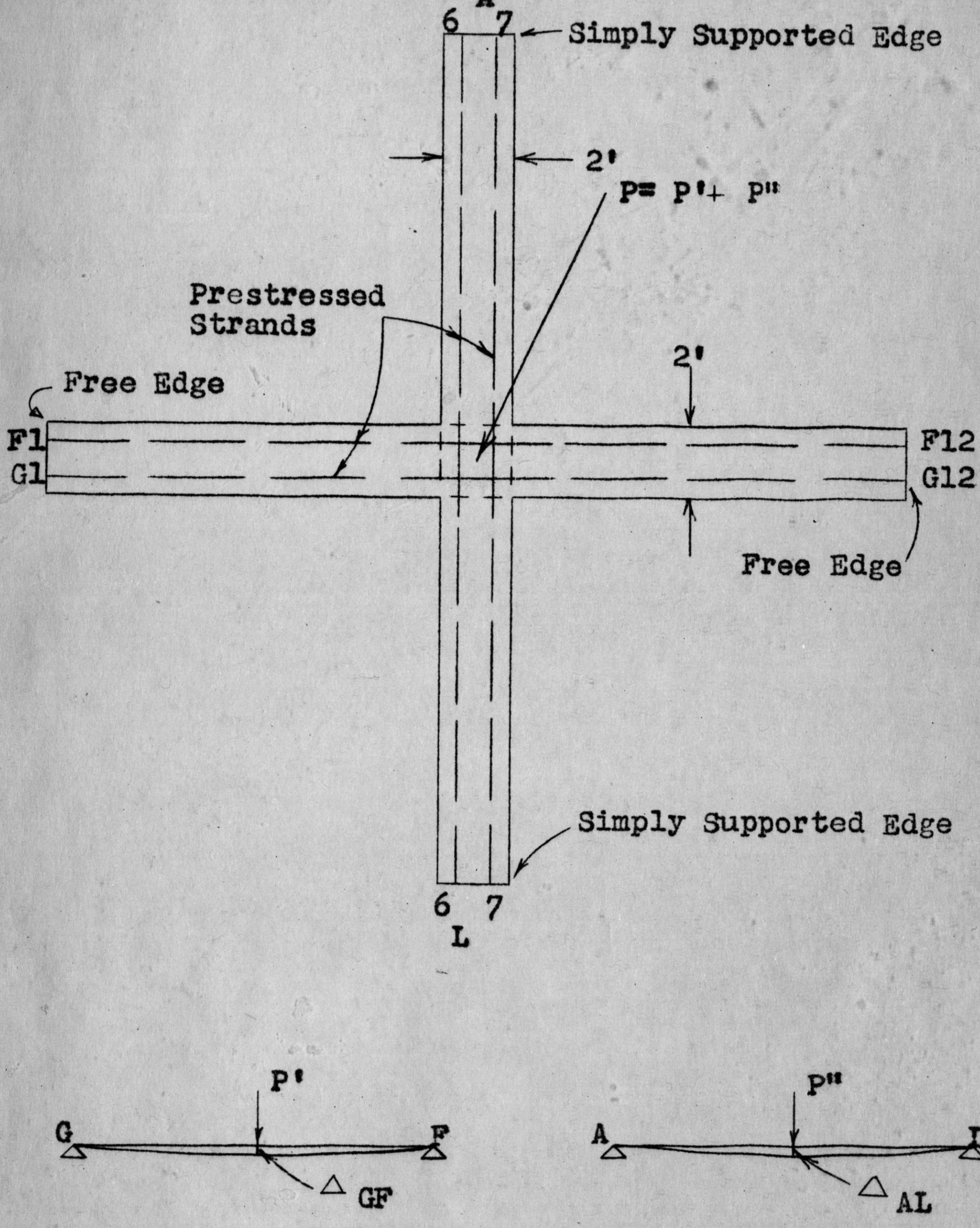


Figure XVIII. The Strips of Slab to be Treated as Beams for Theoretical Computations