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Static and fatigue tests of a 30 foot composite prestressed concrete beam. (Progress Report No. 23), Lehigh University, March 1961

K. W. Nasser

Prestressed Concrete Bridge Members

Progress Report 23

STATIC AND FATIGUE TESTS OF A 30 FOOT COMPOSITE PRESTRESSED CONCRETE BEAM

360
262 ok
List

by

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Pennsylvania Department of Highways

Bureau of Public Roads

Reinforced Concrete Research Council

John A. Roebling's Sons Corporation

American Marietta Company, Concrete Products Div.

Institute of Research, Lehigh University

The test member was manufactured by the Eastern Prestressed Corporation of Line Lexington, Pennsylvania, and was delivered at Fritz Engineering Laboratory for an acceptance test for the Pennsylvania Department of Highways. After the acceptance test was performed, Eastern Prestressed Corporation donated the beam to the Concrete Division for further testing at its discretion.

The author wishes to acknowledge his thanks and gratitude to F. J. Holmes who completed the manuscript; to

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INTRODUCTION

Composite construction of prestressed and ordinary cast-in-situ concrete is used extensively in highway bridges. According to current practice of some agencies, prestressed concrete beams are required to have shear keys in addition to steel shear connectors for complete monolithic interaction between the slab and the beam.

This report presents a study of the behavior of a prestressed composite bridge member under a series of tests that were conducted at Fritz Engineering Laboratory at Lehigh University. These tests were designed to check the beam behavior in flexure and shear under fatigue and static overloads and to compare the effectiveness of ordinary rough concrete surface with shear keys in composite beams. The static overload tests were conducted by applying the loads in different positions along the beam to simulate actual field conditions.

Since only one beam was tested, the results and conclusions must be considered as tentative until additional tests are completed and analyzed.

Hanson[1] has made an extensive study of this subject and reported that rough, bonded surface and stirrups are adequate for shear connections between the precast girder and cast-in-situ slab. Other studies reported by Dean and Ozell[2], as well as other authors[3, 4, 5], showed similar results and conclusions. However, it was recommended[1] that further studies were essential to evaluate the effects of concrete strength, stirrups, scale effect and repeated loading on the shear connection in composite construction.

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1. "Precast-Prestressed Concrete Bridges - 2. Horizontal Shear Connections", by N. W. Hanson, Journal of the PCA Research and Development Laboratories, Vol.2, No.2 (May, 1960)

2. "No Shear Keys are Needed here", by W. E. Dean and A. M. Ozell, Engineering News-Record, 156, 61-62 (June 7, 1960)

3. "Beam Test Shows Need for Web Steel", by W. E. Dean, Engineering News-Record, 157, 36-37 (December 20, 1956)

4. "Behavior of Composite Lintel Beams in Bending", by A. M. Ozell and J. W. Cochran, Journal of the Prestressed Concrete Institute, 1, No. 1, 38-48 (May, 1956)

5. "Some Recent Experience in Composite Precast and In-Situ Concrete Construction, with Particular Reference to Prestressing", by F. Samuely, Proceedings of the Institution of Civil Engineers (London), 1, Part III, 222-279 (August, 1952)

DESIGN AND FABRICATION OF THE TEST MEMBER

The beam was designed in accordance with the specifications of the Pennsylvania Department of Highways,[6] and its details and properties are shown in Fig. 1. The precast I-section was fabricated at Line Lexington, Pennsylvania, by Eastern Prestressed Concrete Corporation using a 5000 psi concrete mix. The prestressing steel consisted of straight 7/16 in. high-strength Roebling strands and the stirrups were made of No. 3 and No. 4 deformed structural grade steel bars. Shear keys 6-in. long and 1-in. deep spaced at 12-in. centers were provided all along the beam, as shown in Fig. 2.

The slab, which was poured at Fritz Laboratory using 3000 psi concrete, was reinforced with 6 x 6 - 2/2 wire mesh. One week before the slab was poured, the shear keys on one half section of the beam were filled with high early strength concrete. The surface of the filled-in concrete was finished in a manner similar to the original top surface of the beam as shown in Fig. 3, where 3/16-in. dia. ball bearings are shown for comparison purposes. The forms for the slab were

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6. "Prestressed Concrete Bridge Superstructures, Section 6.24", by Dept. of Highways, Pennsylvania (January 10, 1958)

supported on the beam to simulate field erection, but temporary supports resting on hydraulic jacks were provided for safety reasons. The jacks were lowered as the beam deflected under the slab dead load in such a way that no reaction was exerted by the temporary supports. Figs. 4 and 5 show a general view of the forms and safety supports for the construction of the slab.

The slab was covered with burlap and moist-cured for seven days. The forms were stripped off after 21 days. Table I shows in chronological order the different stages in the fabrication of the composite beam.

Table I Sequence of Manufacture and Testing

Date	Operation
24 Oct 1959	Concrete for prestressed I-Beam poured
2 Nov 1959	Prestress transferred to concrete
14 Nov 1959	Acceptance test on I-Beam only
8 Mar 1960	Shear keys on one half of beam filled in
15 Mar 1960	Cast-in-situ slab poured
22 Mar 1960	Curing of slab stopped
5 Apr 1960	Formwork to slab stripped
8 to 21 Apr 1960	Testing of Composite Beam

INSTRUMENTATION

Deflection

The deflection measurements were taken by using a conventional engineer's level, sighting on scales fastened to one side of the beam at the ends and midspan.

Longitudinal Concrete Strains

Longitudinal concrete strains were measured with a Whittemore Extensometer on one side of the beam at midspan and four other sections lettered A through E. The extensometer has a 10-in. gage length and measures strains accurately to 0.00001 inches per inch. Gage points consisted of small aluminum plates of dimensions 1/2 x 1/2 x 1/16 in. with small drilled holes to fit the points of the extensometer. Each plate was glued directly to the concrete surface with type A-6 adhesive manufactured by the Armstrong Cork Company. The locations of the extensometer points are shown in Fig. 6.

Differential Movement

Differential movement of the slab with respect to the beam was measured with Ames dial gages located as shown in Fig. 6. The gages had a least count of 0.001 in. except for two gages which were set at the quarter points of the beam and had a least count of 0.0001 in.

Fig. 7 shows strain measurements being taken with the Whittemore gage, and Fig. 8 shows the dial gages used for slip measurement between beam and slab.

PROGRAM OF TESTS

Tests were conducted in the following sequence and each will be described separately.

1. Static test of precast prestressed beam.
2. Fatigue test of the composite beam.
3. Static overload tests of the composite beam.

Table II gives a summary of all tests conducted on the beam with the various increments of loading.

Static Test of Precast Prestressed Beam

Objective

This test of the prestressed girder was a flexure acceptance test performed to satisfy the requirements of the Penna. Department of Highways. The purposes of the test are to check the strength of the member and also to check deflection and recovery on beams tested for plant approval or for suspected construction defects.

Test Set-Up and Procedure

The beam was supported on steel pedestals 30'-3" center to center on the testing bed of the 5,000,000 lb machine. A midspan load of 51.3 kips, which is the theoretical cracking load, was then applied and sustained for one hour. The Penna. Department of Highways in their Bridge Specifications of January 10, 1958 permitted center point or third point loading and required that the theoretical test loading produce a maximum tensile stress of 0.15 f_c' in the bottom fiber. Present specifications permit third point loading only.

Results

Deflection. At the removal of the load the immediate recovery of midspan deflection was 92.6%. A few small hair cracks in the immediate vicinity of the center of the tension flange, which originated under the 51.3 kip load, closed completely after removal of the load.

These results were satisfactory according to the requirements of the Pennsylvania Department of Highways.

The relation of deflection to time in this test is shown in Fig. 9.

Fatigue Test of the Composite Beam

Objective

The purpose of the test was to subject the beam to dynamic loading in order to check the performance of the two shear connections previously described. Slightly less than the design load was used to ensure that a fatigue failure of the strand did not occur. In this case the static overload tests were considered to be more important than the dynamic test.

Test Set-Up and Procedure

The beam was set on neoprene pads 6 x 18 x 3/4 in. which rested on steel supports 30'-3" center to center. See Figs. 10 and 11 for test set-up and support details.

The jack load was applied at midspan to the center of a spreader beam which rested on the slab at two points 4'-6" apart. An Amsler machine, which consisted of a jack and a pulsator, was used to apply the fatigue loading. The pulsator is essentially a hydraulic pump which causes variation of oil pressure within the jack, so that a load at a frequency of 250 or 500 cycles per minute can be applied to the test member. The same equipment was used to apply the static loads.

Natural Frequency

Before the fatigue test was started estimates of the natural frequency and the magnification factor for the test member were calculated. First the natural frequency was evaluated from the expression:

$$p = \frac{\pi^2}{L^2} \sqrt{\frac{EI}{m}}$$

where

p = natural frequency in radians per second

L = span of the beam in feet

E = modulus of elasticity of the beam

I = moment of inertia of the beam

m = mass per unit length of the beam = w/g

w = weight per unit length of the beam = 925 lbs/ft

g = acceleration due to gravity in ft/sec/sec

Numerically

$$p = \frac{(3.14)^2}{(30.25)^2}\sqrt{\frac{4300 \times 92,360 \times 32.2}{0.925 \times (12)^2}}$$

$$= 105.7 \text{ rad/sec} = 1010 \text{ rpm}$$

This showed that it was safe to run the test at 250 cycles per minute.

Watch resonance

Magnification Factor

For this rotational speed the magnification factor was estimated by using the approximate expression:

$$M.F. = \frac{W}{g} \times \frac{w^2}{c}$$

where

M.F. = magnification factor.

W = $W_1 + W_b/3$

W_1 = concentrated weight at the point of application of the dynamic load. In this case the spreader beam (negligible).

W_b = uniformly distributed weight over the span of the beam. (28 kips).

w = frequency of the loading in radians/sec.

g = acceleration due to gravity.

$$c = \text{spring constant of the beam,}$$

$$= \frac{\text{static load}}{\text{deflection due to static load.}}$$

Any system of consistent units may be used to evaluate the "M.F."

For this beam

$$\text{M.F.} = \left(\frac{28}{3}\right) \times \frac{1}{32.2} \times \left(\frac{250 \times 2\pi}{60}\right)^2 \times \frac{\frac{1}{60}}{(0.20/12)}$$

$$= 5.4\%$$

Maximum and minimum effective loads of 44.8 kips and 12.4 kips respectively were applied to the composite beam. These loads produced 97.0% and 26.8% of design moment in the section between the two loading points. The test member was subjected to the above loading for 1,072,000 cycles at a frequency of 250 rpm. It was assumed that a structure during its useful life may undergo one million cycles of design loading.[7]

Measurements for deflection and slip were taken at the end of every third of a million cycles.

Results

Deflection. Figure 12 shows the relation between load and midspan deflection before and after the fatigue test.

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7. "Endurance of a Full Scale Pretensioned Concrete Beam" by K. E. Knudsen and W. J. Eney. Fritz Laboratory Report 223.5.

The graph indicates that there was no significant change in the load deflection characteristics of the beam.

Slip. No slip dial showed any change in its reading. Hence there was no slip between the slab and the beam on either section of the member. Therefore the performance of the ordinary rough concrete surface was equivalent to that of the shear keys for this member under fatigue loading.

Static Overload Tests of the Composite Beam

Objective

The purpose of the static overload tests was to study the performance of the test member under increasing static loads, applied at the different load points described below, until failure was reached. This procedure of shifting the position of the static load was followed because it simulated the field conditions in a bridge with the wheel loads moving along the member.

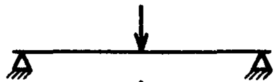
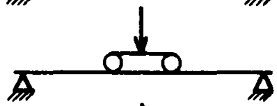
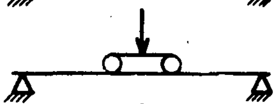
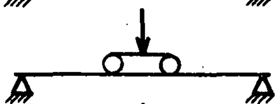
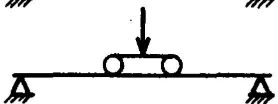
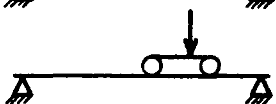
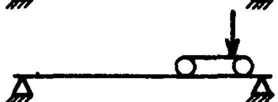
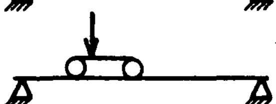
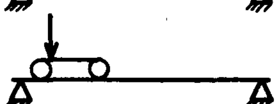
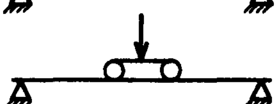
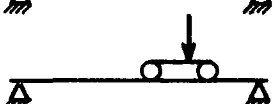
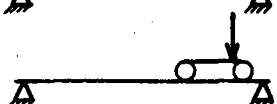
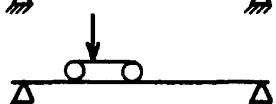
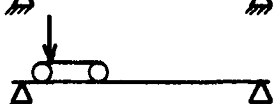
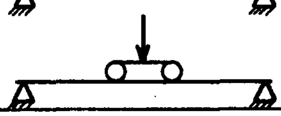
Test Set-Up and Procedure

The distance between the supports and the details of the neoprene pads were as explained for the fatigue test and as shown in Figs. 10 and 11. The load was applied at five different positions lettered A, B, C, D, and E as indicated in Fig. 6. Schematically the loadings and their increments are shown in Table II. Typical test set-ups are shown in Fig. 13.

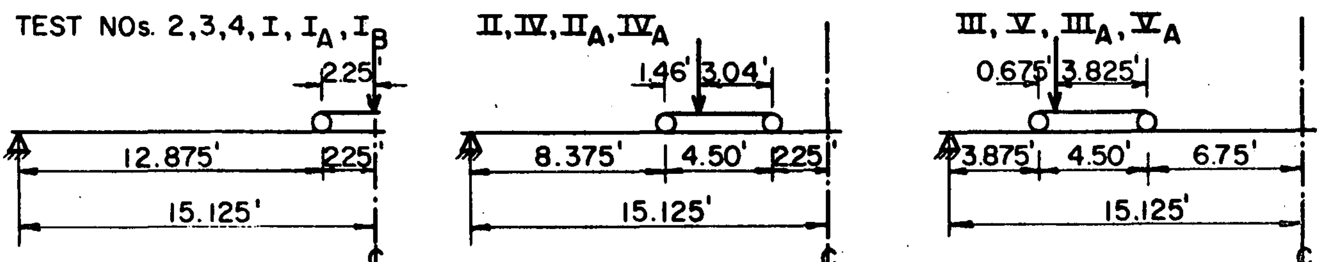
The load was applied through a spreader beam to the test member in such a way as to produce zero shear in the region lying beneath the spreader beam. These tests with different shear to moment span ratios were performed so as to obtain the maximum possible information for both the beam-slab interaction and the shear behavior of the beam and its cracking patterns. First the load increments were applied at midspan and next moved to the section with ordinary rough concrete surface and then to the section with shear keys.

Measurements of midspan deflection, strains, slip, and crack patterns were taken at each increment of the load.

Table II Sequence of Tests

DATE	TEST NO.	LOAD POSITION	TYPE OF TEST	LOAD INCREMENTS (KIPS)	REMARKS
1959 14 Nov.	1		Static	0 51.3	Prestressed Beam only Acceptance Test
1960 8 Apr.	2		Static	0 30 60	
8-11 Apr.	3		Dynamic	12.4 min., 44.8 max.	1,072,000 Cycles at 250 cpm
11 Apr.	4		Static	0 60	
12 Apr.	I		Static	0 60 95 115	
13 Apr.	II		Static	0 60 95 115	
14 Apr.	III		Static	0 60 95 115	
14 Apr.	IV		Static	0 60 95 115	All Tests with the Exception of the Acceptance Test were Made on the Composite Beam.
18 Apr.	V		Static	0 60 95 115 135	
18 Apr.	I_A		Static	0 95 135 150	
19 Apr.	II_A		Static	0 95 135 150	
19 Apr.	III_A		Static	0 95 135 150	
19 Apr.	IV_A		Static	0 95 135 150	
21 Apr.	V_A		Static	0 95 135 150	
25 Apr.	I_B		Static	0 135 150 186	Beam Failed at 186 k

LOAD POSITIONS



NOTE: Pure Bending is induced between the Load Points in all cases.

Results

The main results of these tests are shown in Figs. 14 through 26 which will be discussed separately.

Deflection. Fig. 14 shows the relation between load and midspan deflection. The zero-load points are plotted successively from the point of zero deflection at start of Test No. I, showing cumulative residual deflection after each test.

After the fatigue test and eleven static overload tests, with a maximum load of 150 kips, which produced 324% of design moment, the midspan residual deflection was 0.33 in. The recovery of deflection following the first and second series of tests was excellent, and the permanent set observed was 0.09 and 0.27 in. after Test No. V and V_A respectively.

Strains. The relation of strains to vertical distance at various sections of the beam are shown in Figs. 15-17.

Since there was essentially no difference in strains at comparative sections of the beam where shear keys and ordinary concrete surface were provided , average values were used to plot the curve; for example, the strains at section A, test II, were averaged with the strains at section E, test IV. Measured strains were linear over the major range of the overloads which indicated complete interaction between the slab and the beam at all sections at that range of loading.

Effective Width. Extensive measurements for the study of effective width in T-beams are shown in Figs. 15-17. Since a reasonably uniform distribution of strain was observed, it was concluded that the full slab width was effective.

Cracking Patterns. Three distinct cracking patterns were observed in the web of the member during the series of overload tests. In the first midspan loading, the first cracks were observed within the pure moment region and followed essentially a vertical path. With an increase in the load, other vertical cracks appeared close to, but on the outer or support side of the load points. These cracks from first appearance extended a few inches vertically, and upon further loading inclined toward the load point to follow a path approximately 45 degrees to the horizontal.

As the centerline load was increased to 106 kips a series of diagonal cracks suddenly formed in the web of the eastern section of the member - the side with ordinary regular concrete surface. This system of parallel cracks which appeared in the thin-web portion of the member, remote from the other cracks in the inner portion of the beam, extended out to within a few feet of the support. When the load had been increased to 115 kips, the diagonal cracks had extended to within three feet of the east support; no diagonal cracking had occurred in the west span of the member - the span with shear keys.

Loadings to 115 kips at load positions B and A successively, which increased the shear in the member, produced further vertical cracking in the outer portions of the beam and diagonal cracking to within one foot of the support.

Diagonal cracks first appeared in the west half span when the load was applied at position D. After the first set of tests (I to V) had been completed at 115 kips, a very symmetrical pattern of cracks had formed.

In the second set of tests (I_A to V_A) a development of the existing cracks occurred, together with the appearance of a considerable number of new ones. When the loadings were applied in the outer positions, some of the existing inclined cracks which had followed a path towards the central portion of the beam changed direction and developed in the direction of the new load position, sometimes cutting diagonally across existing inclined cracks.

The final loading was applied at midspan only, and further developments of the existing crack patterns occurred, with the vertical cracks in the central portion of the beam extending to within two inches of the top surface of the slab. The diagonal cracking had extended to a point a few inches directly above the east support and to within a foot of the west support. Considerable widening of the inclined and vertical cracks in the central portion of the beam occurred, accompanied by a central deflection of several inches. Although the diagonal cracks in the outer portion of the span extended through the web of the member, they did not open appreciably. It was apparent that the quantities of web reinforcement in the outer regions of the beam were more than adequate to carry the forces introduced when the concrete cracked.

Fig. 18 shows the extent of the cracking at the completion of Test No. I. The final appearance of the cracking patterns, after the two sets of tests and prior to the test to destruction, is shown in Fig. 19.

Ultimate Load. During the final static load to destruction no significant new cracks were observed to form, but the existing cracks developed and widened extensively. Yielding in the steel was evident at the 150 kip load but actual failure took place at a load of 186 kips by initial crushing in the top fibers of the slab at the west load point followed by a shattering of the slab in the pure moment region along the horizontal wire mesh reinforcement. At the instant of failure a central deflection of 9 in. had occurred. Figure 22 shows the beam with a load of 186 kips and a midspan deflection of 8-1/2 in. just before failure. Figures 23 and 24 show views of the final failure.

The theoretical estimated ultimate load was 153 kips, determined using the Tentative Recommendations of Prestressed Concrete of the joint ACI-ASCE Committee, and using the manufacturer's guaranteed value of 250,000 psi for the ultimate strength of the strand and the cylinder test value of 3650 psi for the ultimate strength of the slab concrete. Actual failure took place at a load of 186 kips which is an increase of 21%.

Shear Connection. The slip along the joint between the slab and the prestressed portion of the beam is shown in Fig. 26. Essentially there seemed to be no difference in performance between the section with shear keys and that with ordinary rough concrete surface. Near ultimate load a horizontal crack was observed forming at the joint in the section where shear keys were provided as shown in Fig. 25. Also, the slip was continuous and well pronounced in that section as seen in Fig. 26.

The behavior of the ordinary rough concrete surface and the shear keys was to be expected since the maximum shearing stress along the joint did not exceed 260 psi, and ample shear connectors were provided. These were adequate to transmit the horizontal shear under all loads.

CONCLUSIONS

1. The composite beam endured fatigue loadings for 1,072,000 cycles at a maximum of 97.0% and a minimum of 26.8% of design load without any slip between the slab and prestressed portion. Thus there was complete interaction at all sections of the beam where shear keys and ordinary concrete surface were used.

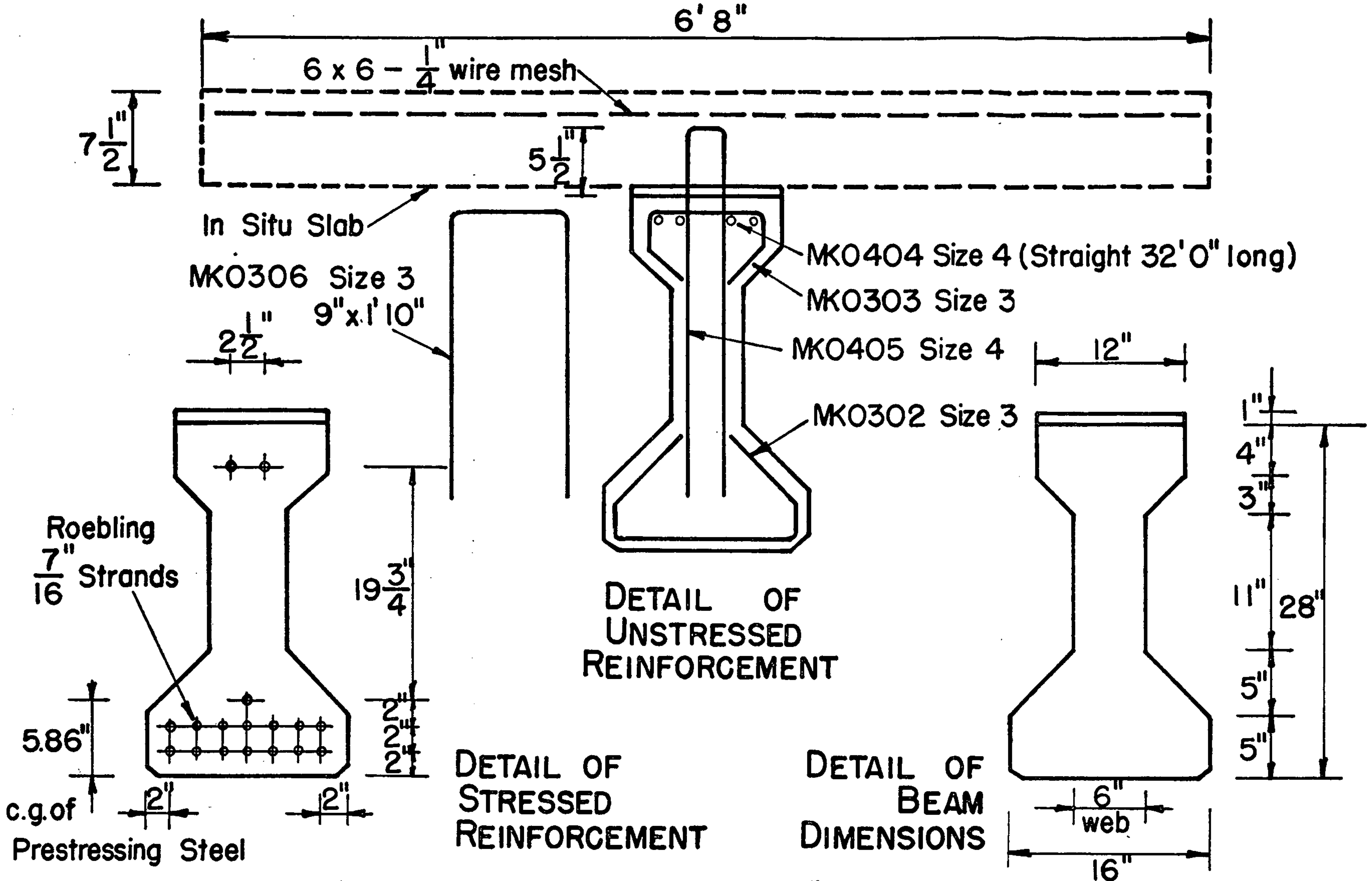
2. While diagonal cracking occurred initially on the side of the beam without shear keys, by the completion of the first set of tests (I to V) a very symmetrical crack pattern had formed. The performances of both the shear keys and the ordinary concrete surface for this test member were essentially the same under fatigue and static overloads.

3. The shear connectors used were adequate for transmission of shear under all loads without the assistance of shear keys.

4. The recovery of deflection of the beam was good for all tests preceding the ultimate load.

5. The ultimate load was 21% more than that estimated using the ACI-ASCE recommendations.

6. In similar T-beams, with a width to span ratio of 0.22, the full width of the slab is effective in resisting the longitudinal moments.



PROPERTIES OF BEAM

		Prestressed Beam only	Composite Beam
Second Moment of Area about horizontal axis through centroid	I_{xx}	22,780 in.^{4}	92,360 in.^{4}
Distance from Centroid to bottom	c^b	12·60 in.	25·30 in.
Distance from Centroid to top	c^t	15·40 in.	11·20 in.
Section Modulus I_{xx}/c^b	S^b	1820 in.^{3}	3670 in.^{3}
Section Modulus I_{xx}/c^t	S^t	1430 in.^{3}	10,700 in.^{3}
Dead Weight of Beam	w	288 lb./ft.	925 lb./ft.
Ultimate Compressive Strength of Concrete	f'_c	7150 p.s.i.	Slab only 3650 p.s.i.
Young's Modulus of Concrete	E_c	4,300,000	3,300,000
$\frac{E_c\text{ (Slab)}}{E_c\text{ (Beam)}} = n = 0{\cdot}77$	Effective Width of Slab = 61.5 in.		
Theoretical Ultimate Moment of Resistance for Composite Beam = 13,300,000 in.lb. = 1110 ft kips			

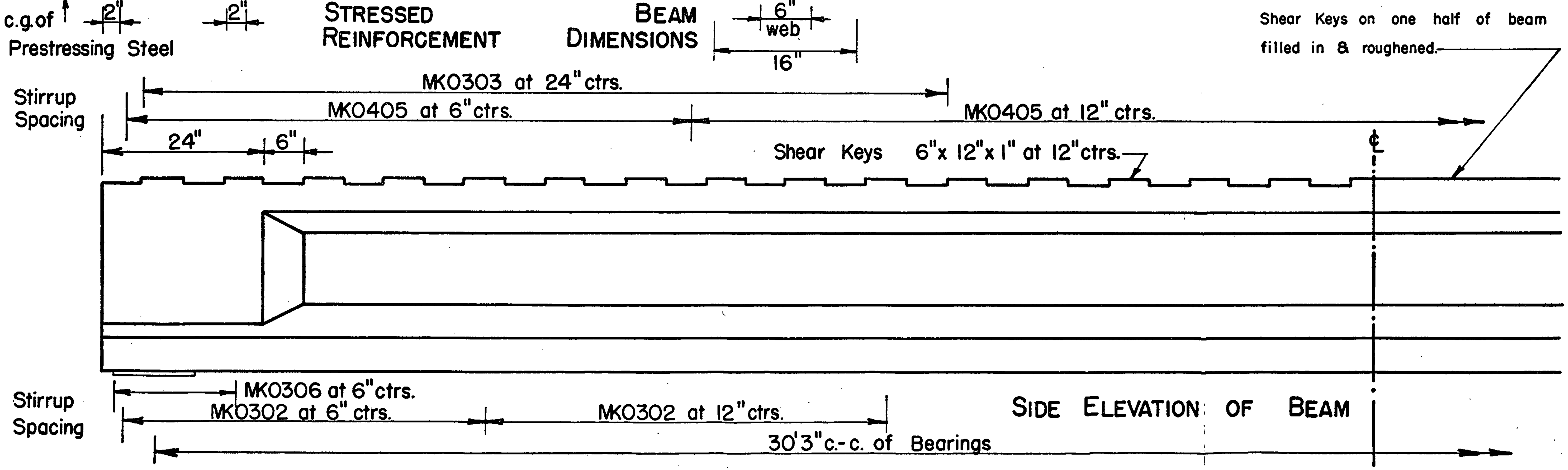


Fig. 1 - Elevation, Section & Details of Test Member

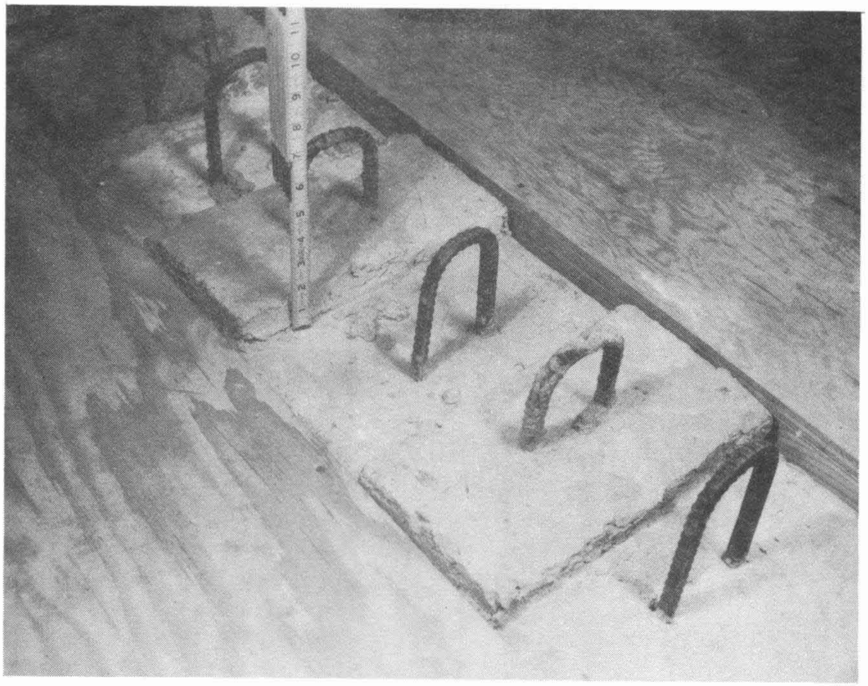


Fig. 2 Close-Up View of Shear Keys

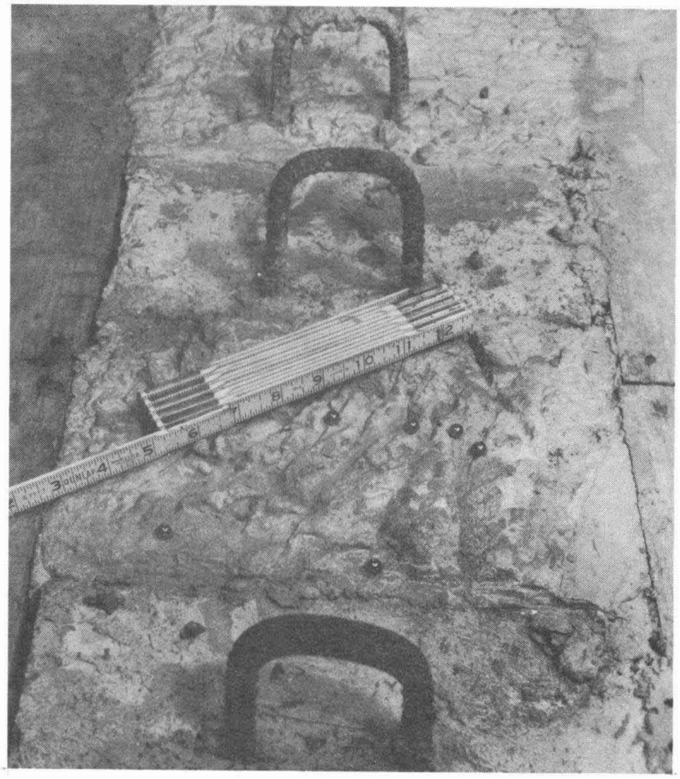


Fig. 3 Close-Up View of Filled-in Shear Keys

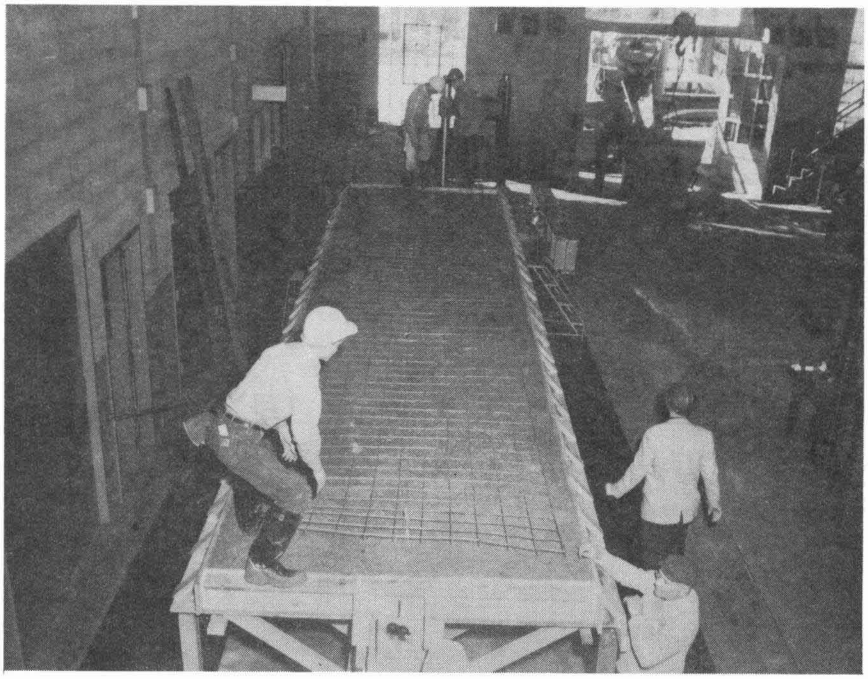


Fig. 4 Fabrication of the Cast-in-Situ Slab

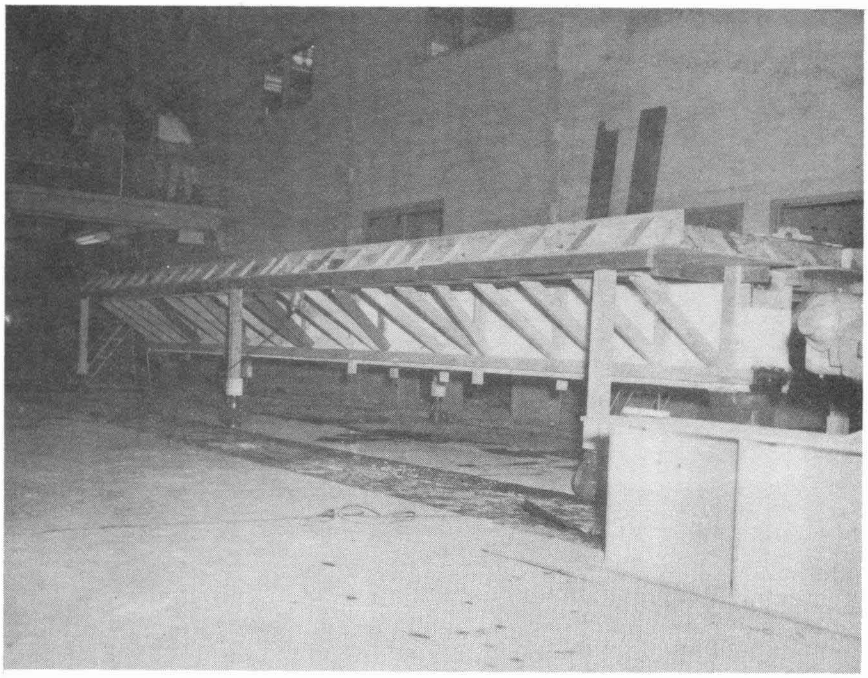


Fig. 5 General View of Forms and Safety Supports for the Slabs

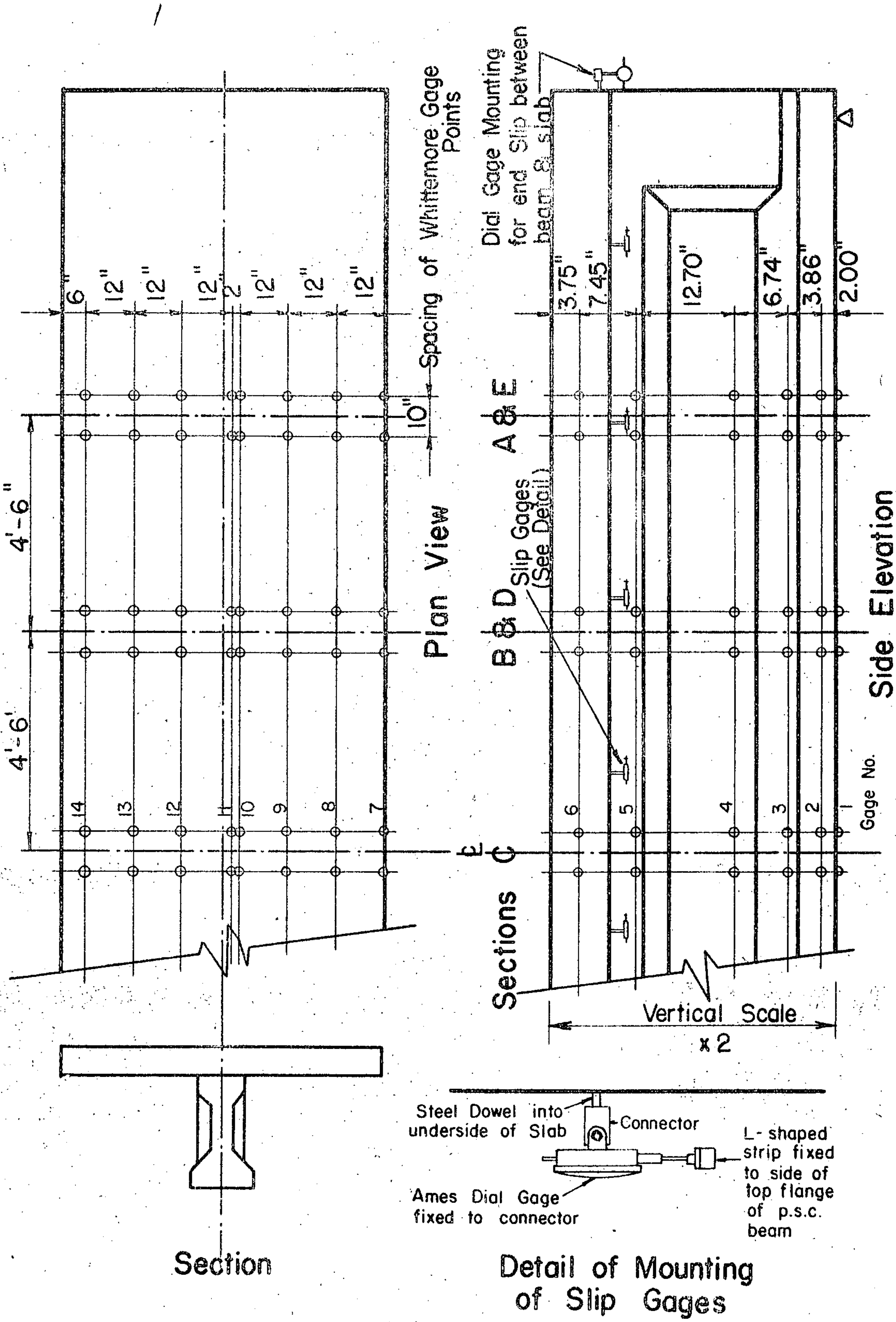


Fig. 6 Instrumentation for the Tests

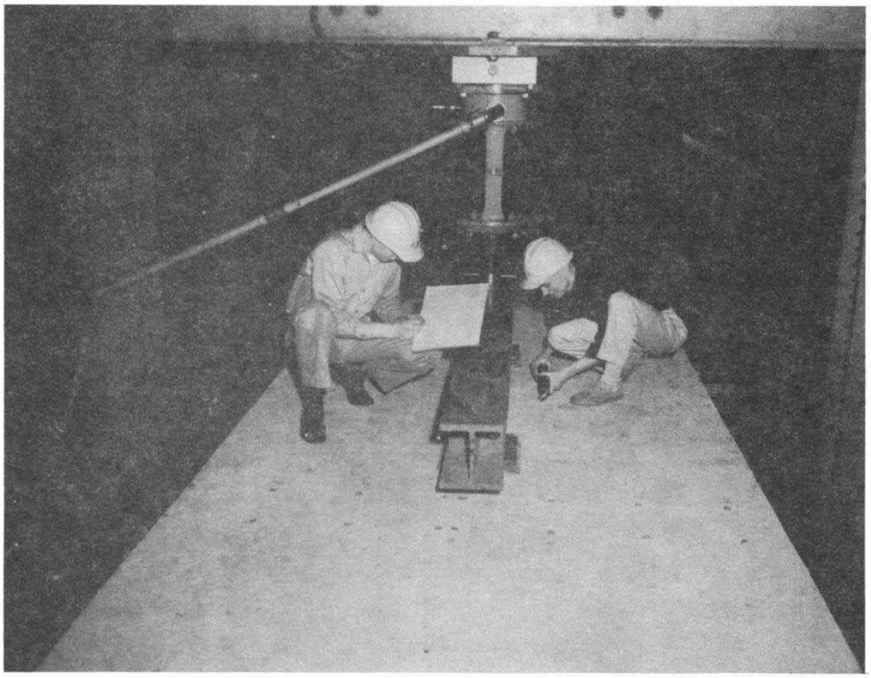


Fig. 7 Concrete Strain Measured by Whittemore Extensometer

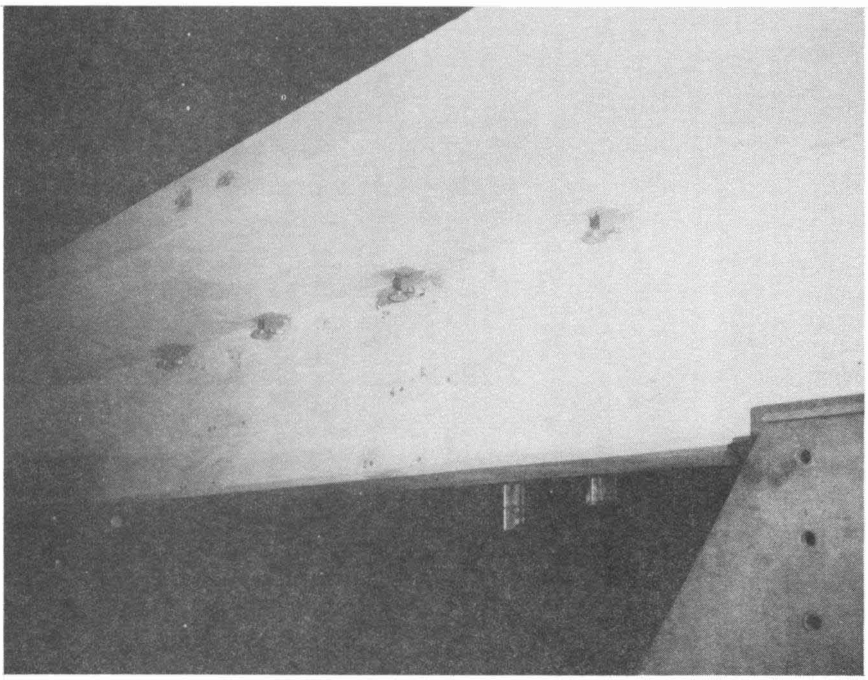


Fig. 8 Ames Dials for Measuring Slip

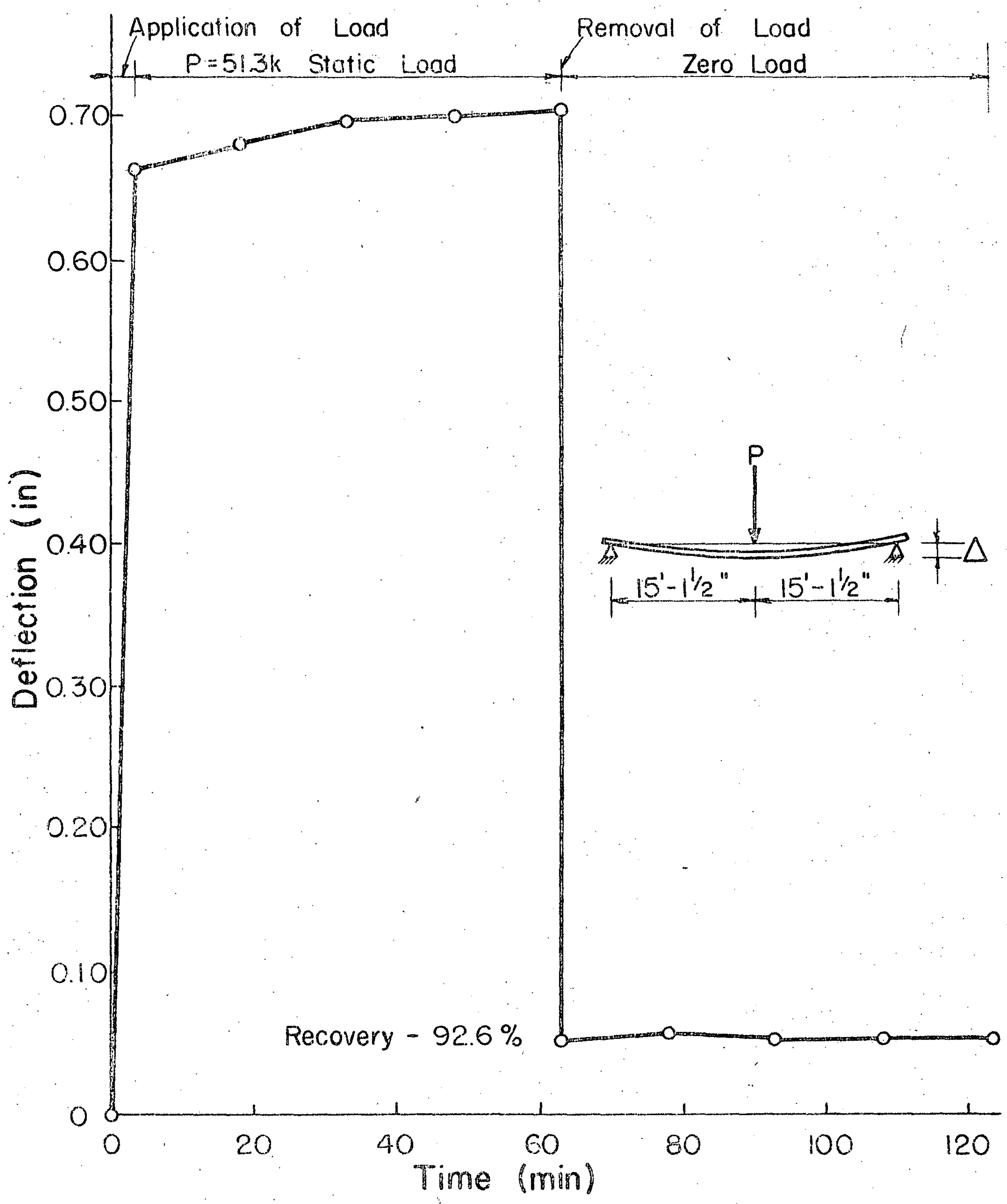


Fig. 9 Relation of Deflection to Time for the Precast Prestressed Beam only

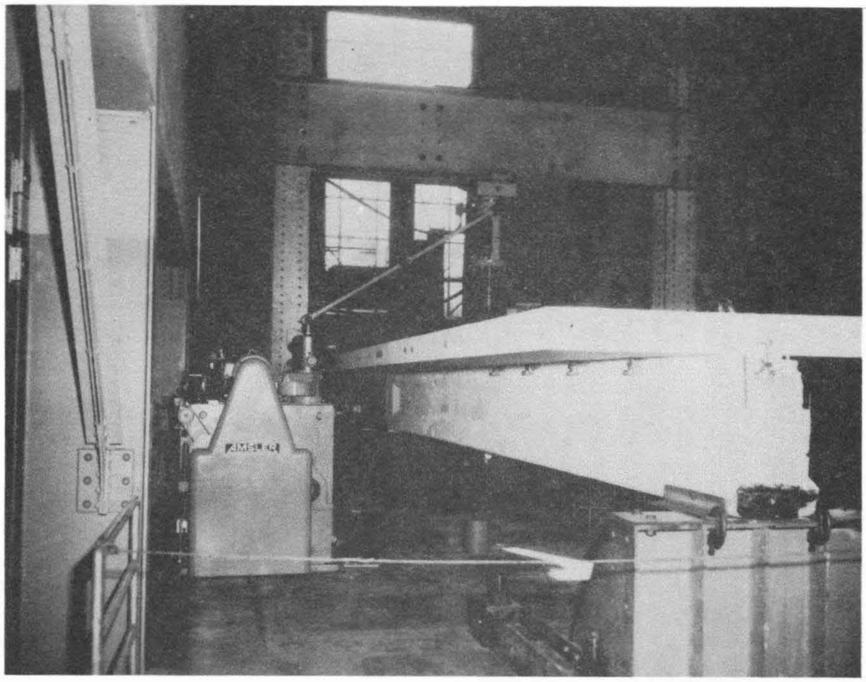


Fig. 10 Set-Up for Fatigue Test

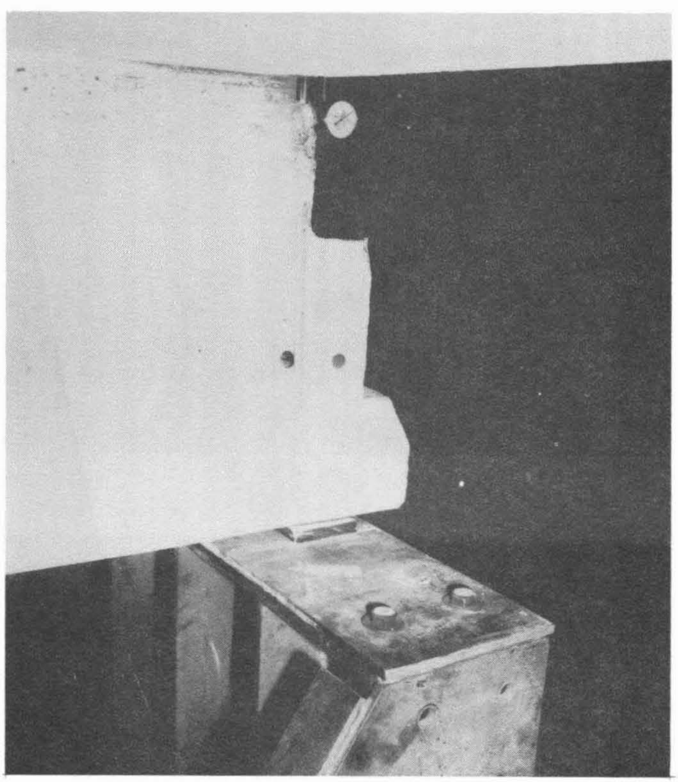


Fig. 11 Details of Support

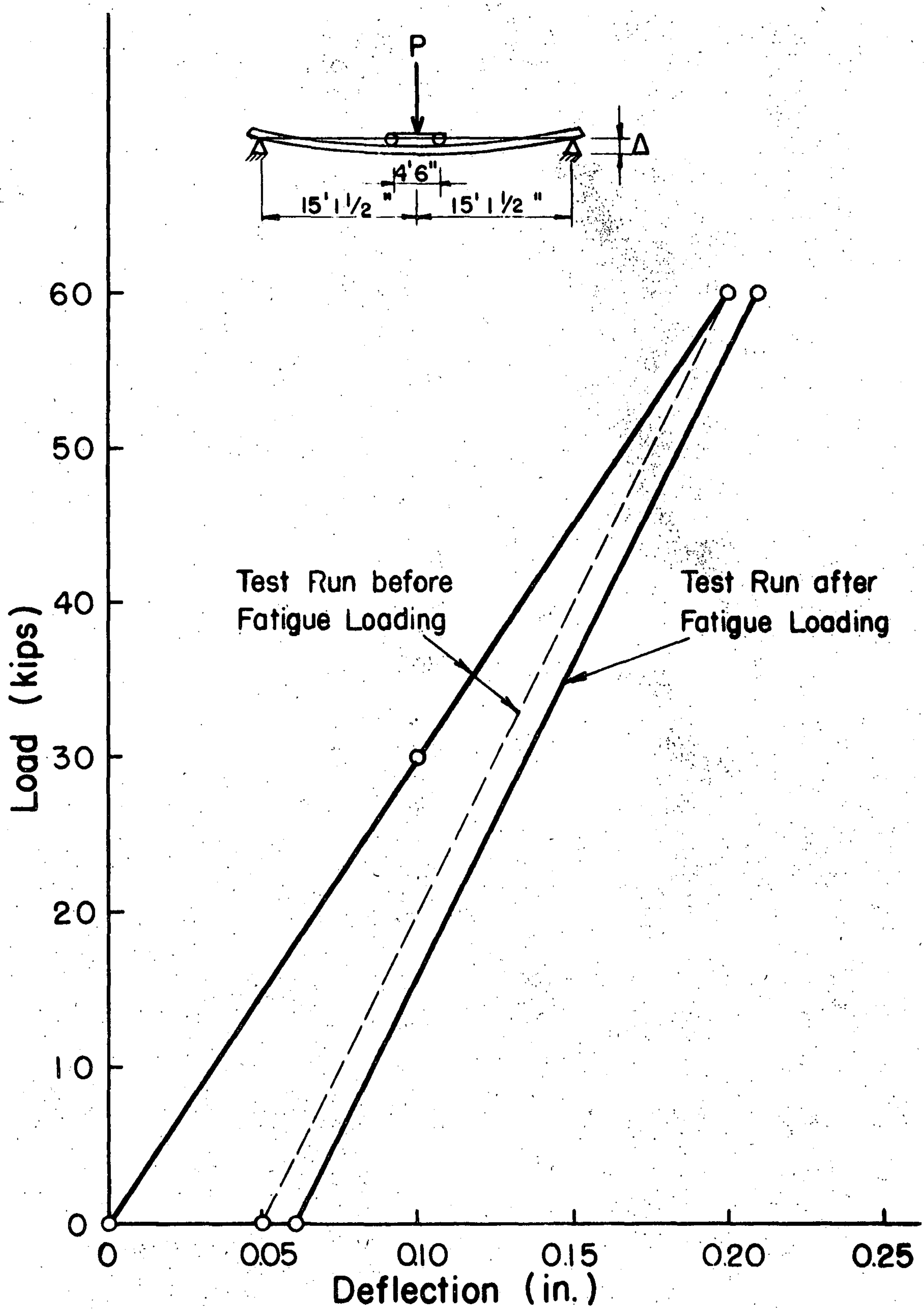
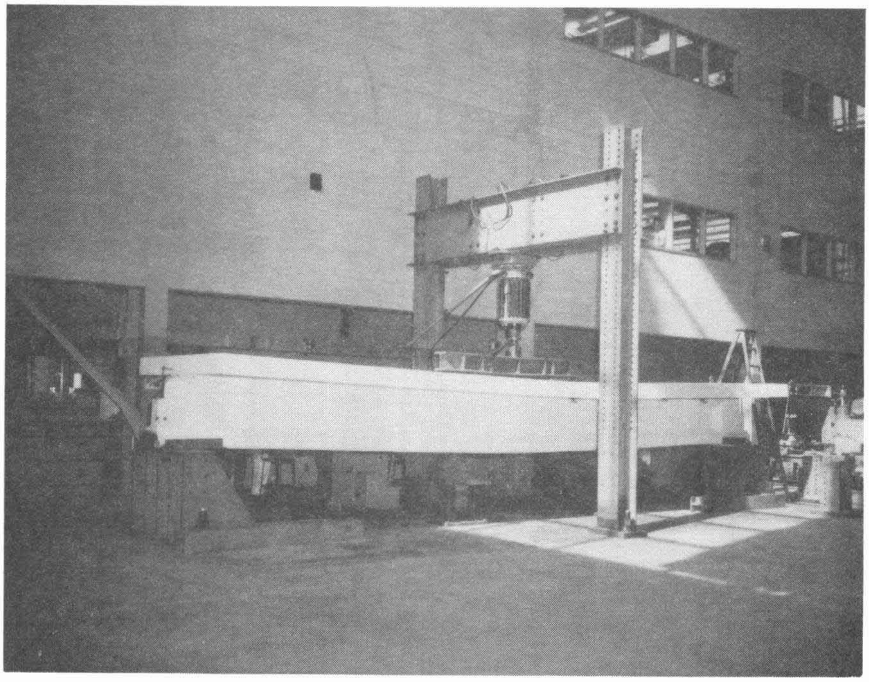


Fig. 12 Relation between Load and Midspan Deflection before and after 1,072,000 Cycles of Fatigue Loading



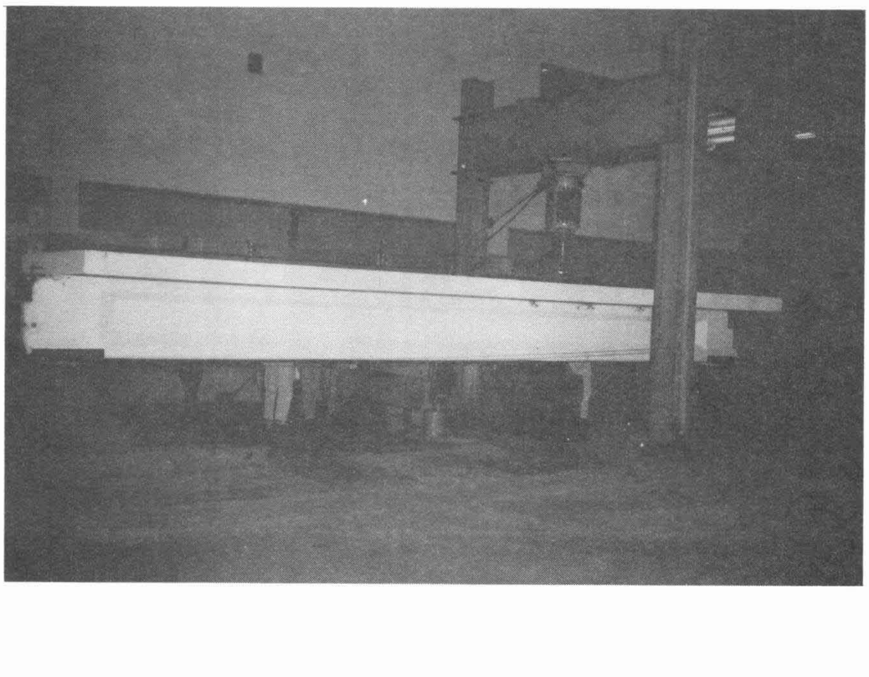


Fig. 13 Typical Test Set-Ups for Static Overloads

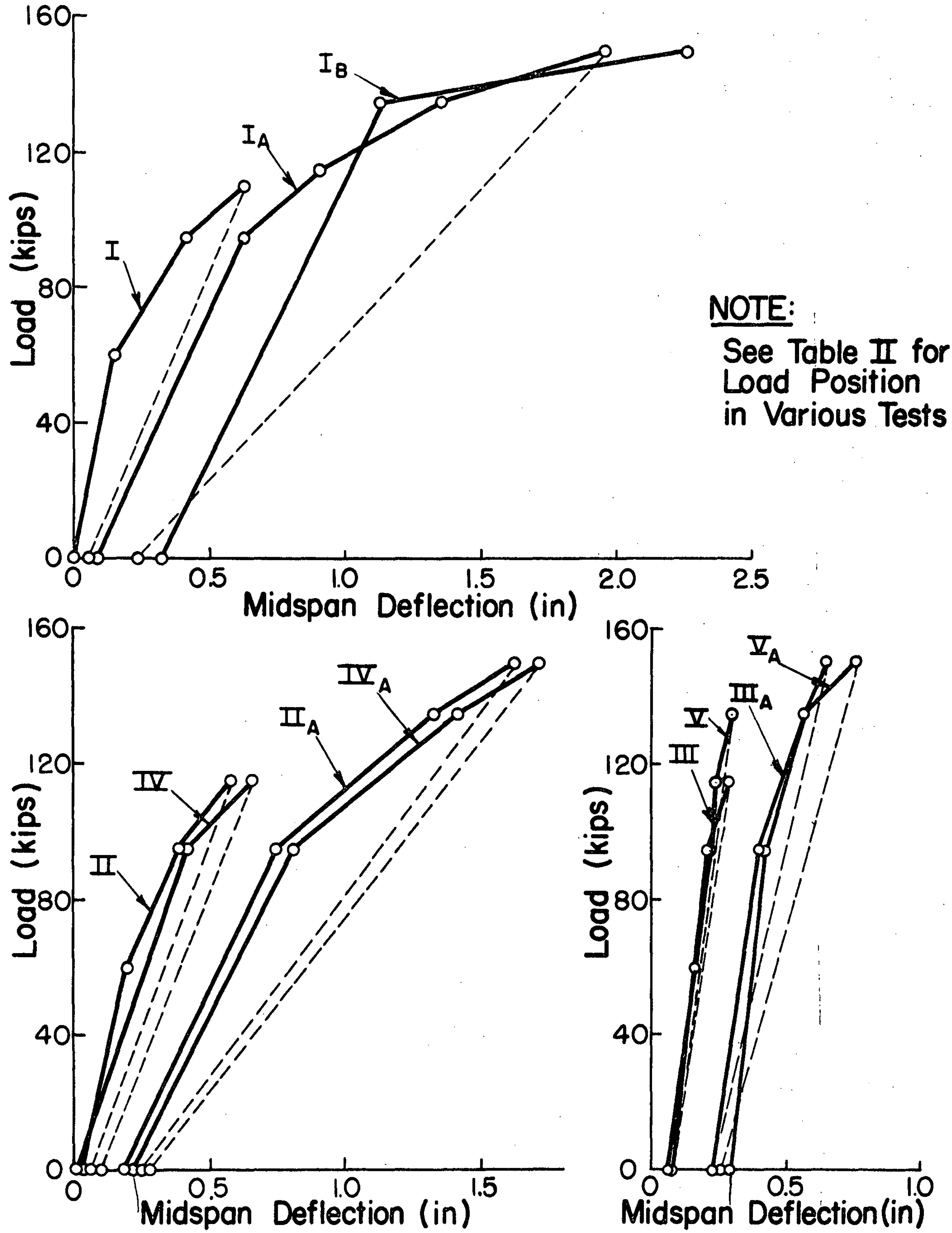


Fig. 14 Relation between Load and Midspan Deflection

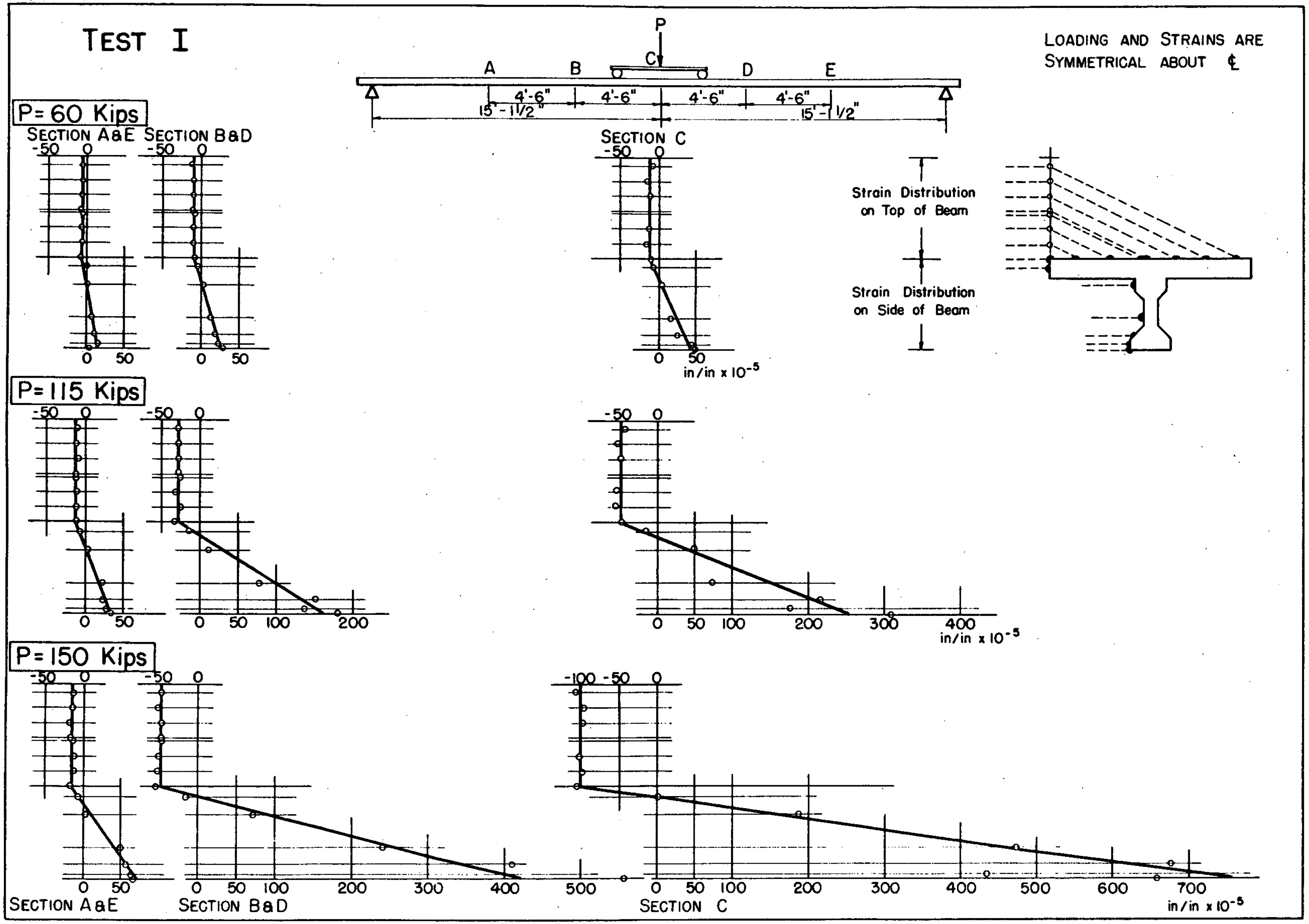


Fig. 15 Relation between Applied Loads and Induced Strains on Side and Top of Beam - Test I

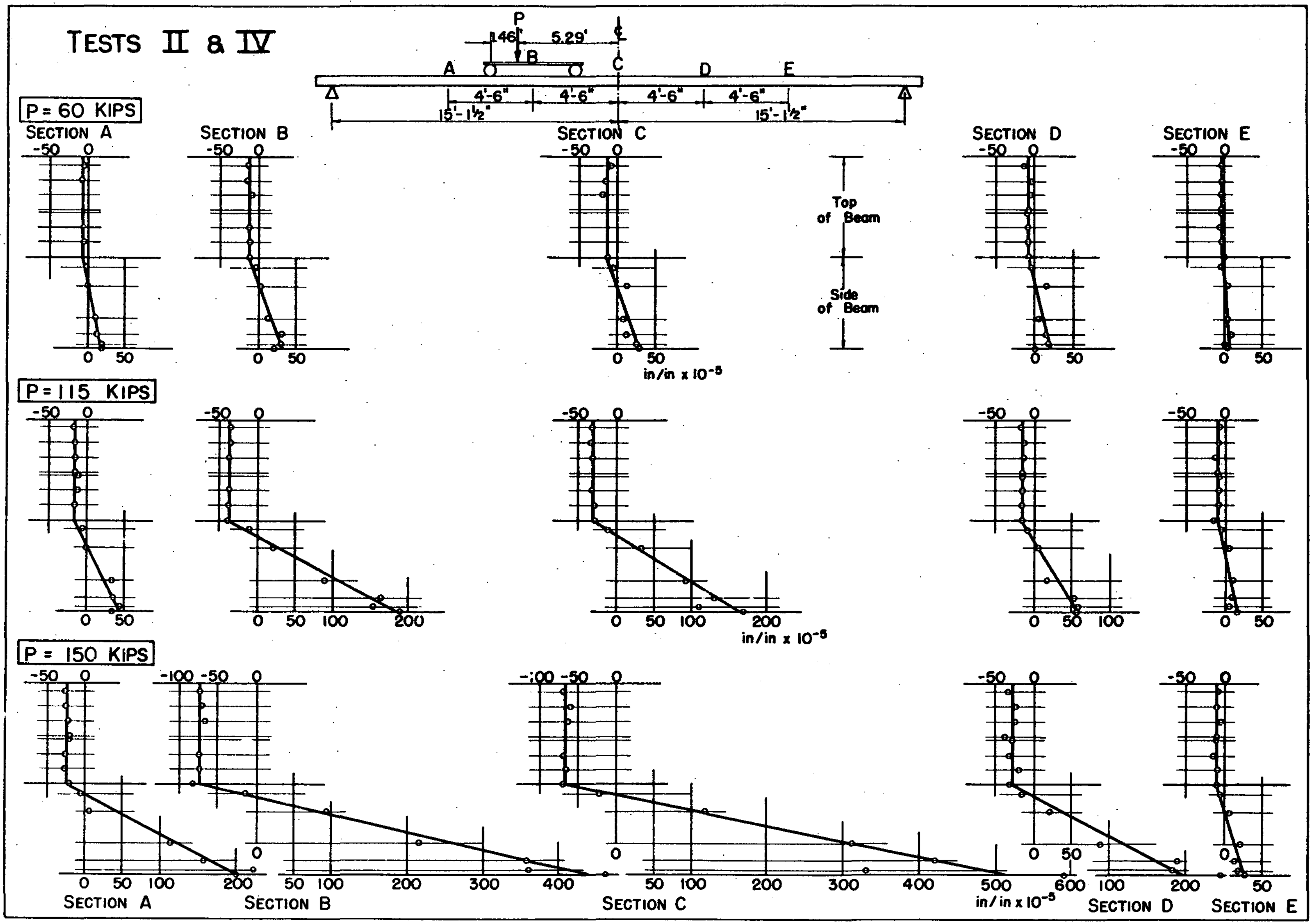


Fig. 16 Relation between Applied Loads and Induced Strains on Side and Top of Beam - Tests II and IV

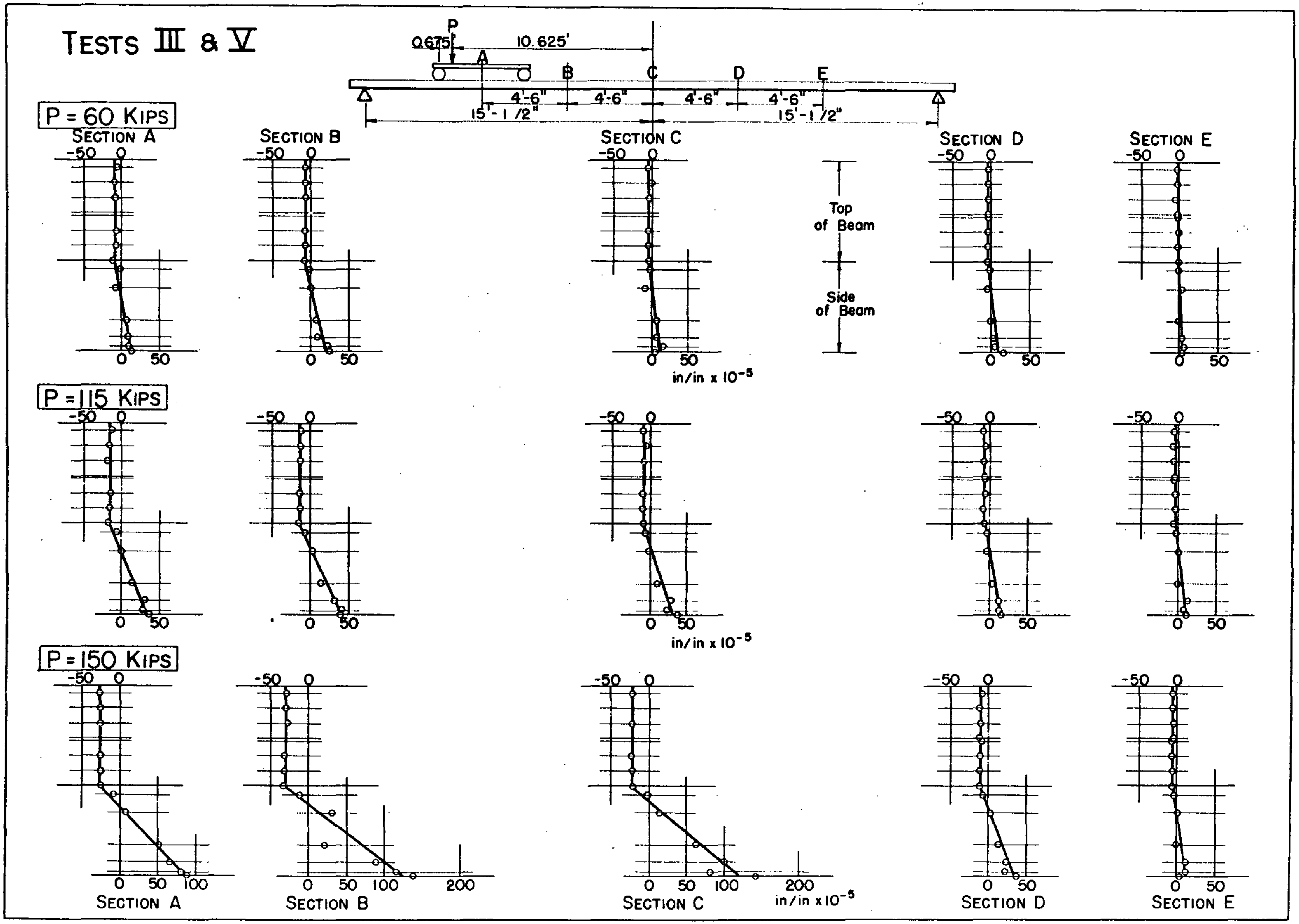


Fig. 17 Relation between Applied Loads and Induced Strains on Side and Top of Beam - Tests III and V

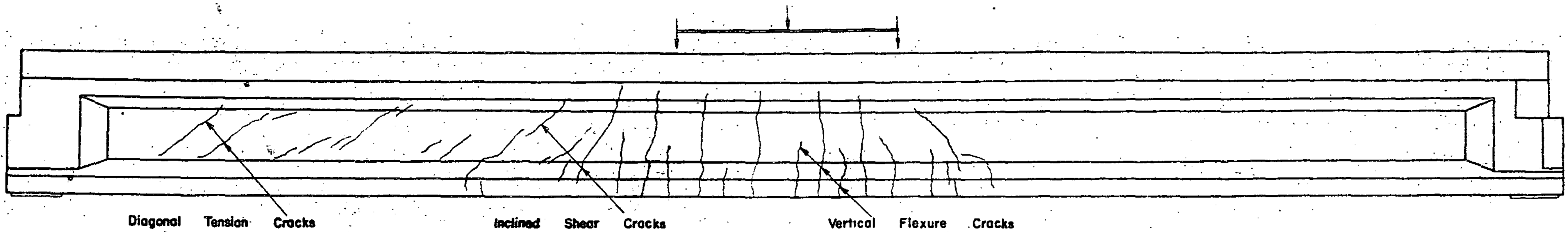


Fig. 18 Cracking Pattern at the Completion of Test No. I

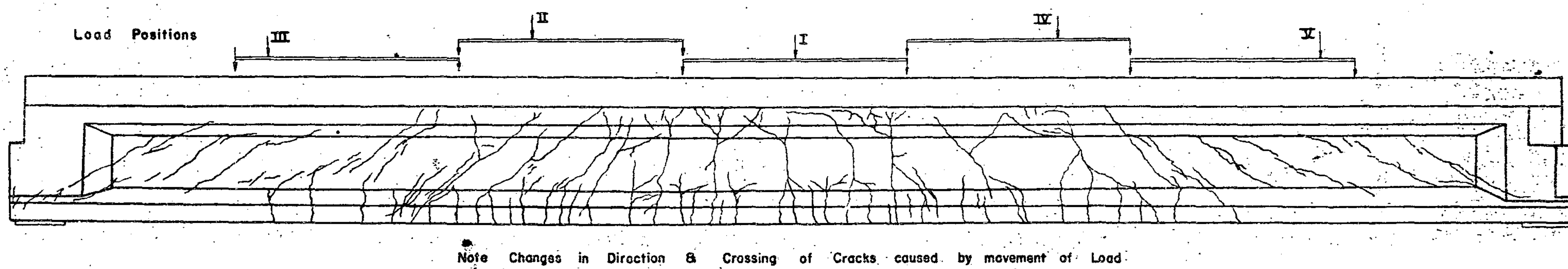


Fig. 19 Cracking Pattern at the Completion of Test No. V_A and Prior to Test to Destruction

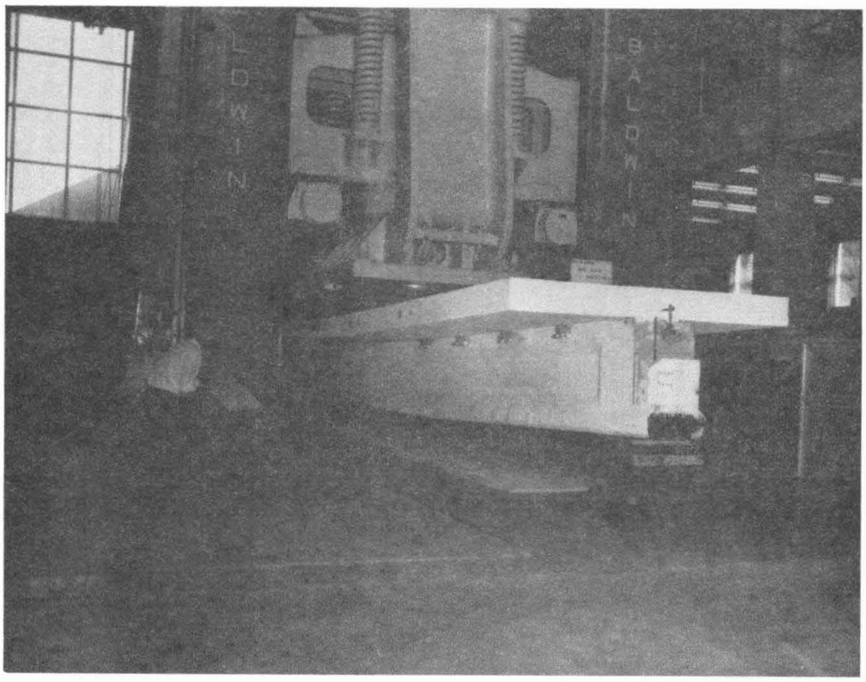


Fig. 22 View of the Beam with 186 kip Load and Midspan Deflection of 8.5 in.

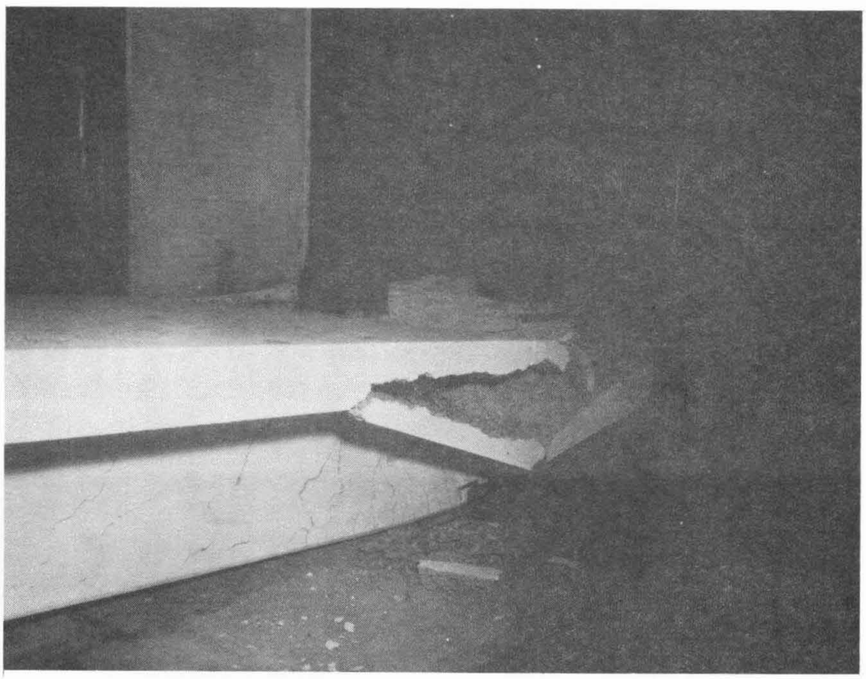


Fig. 23 View of the Beam after Failure

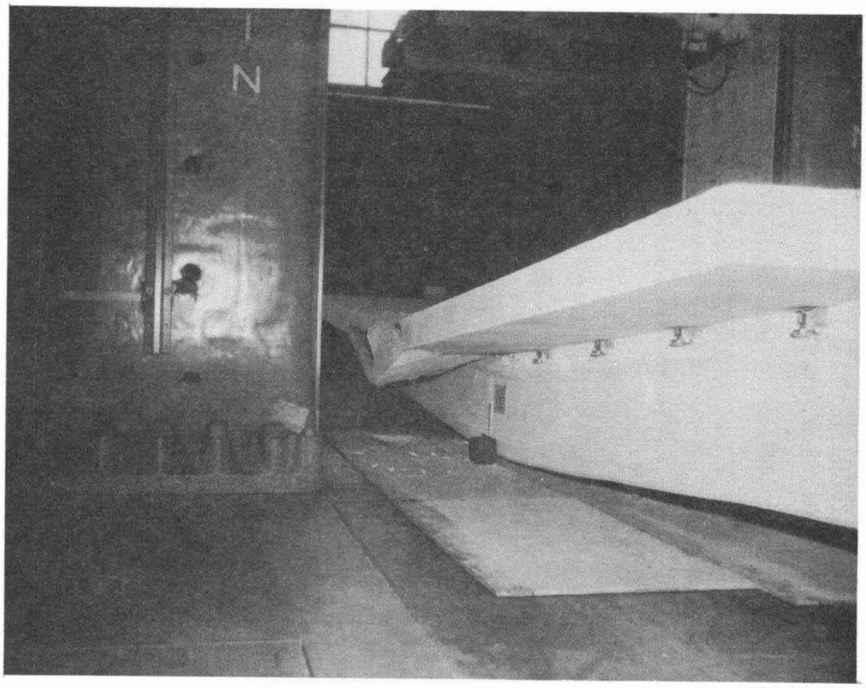


Fig. 24 View of the Beam after Failure

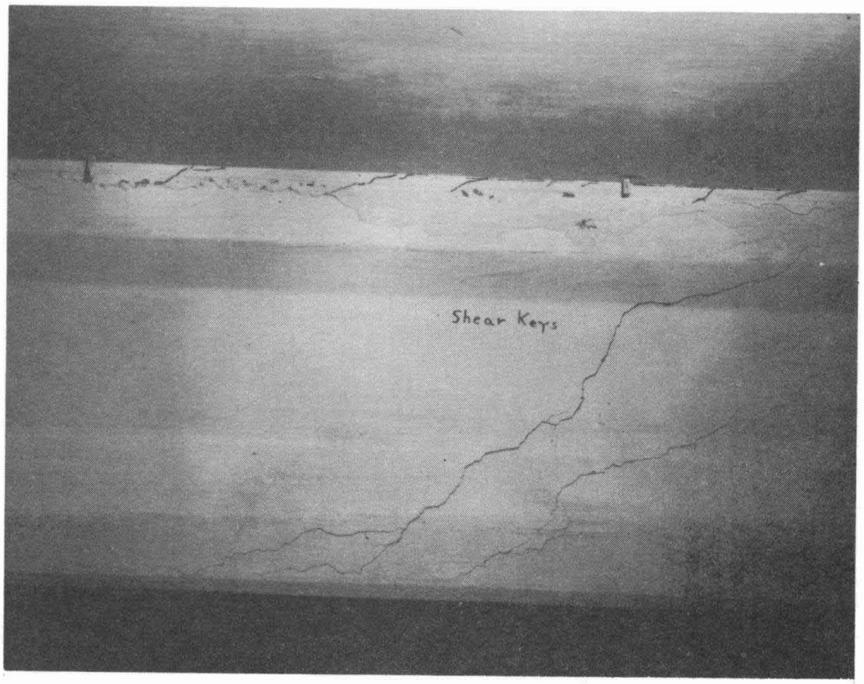
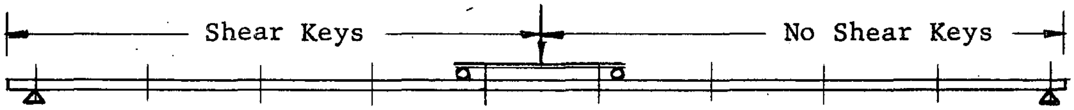


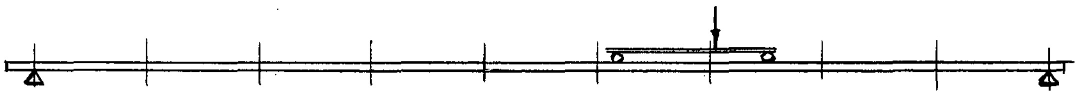
Fig. 25 View of the Horizontal Crack at the Joint of Slab and Prestressed Beam in the Region where Shear Keys were used

Load kips	SLIP - in. x 10^{-3}									

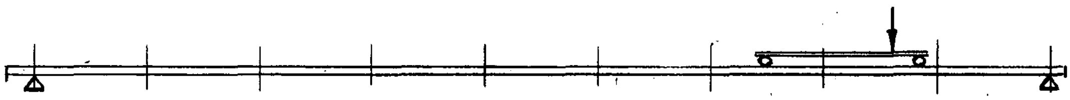
Shear Keys — No Shear Keys



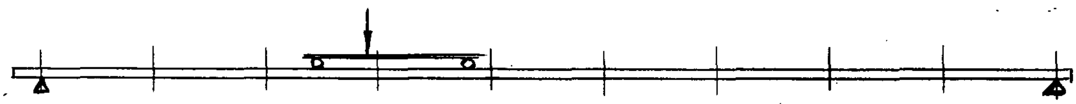
Load kips										
0	0	0	0	0	0	0	0	0	0	0
95	0	+1	0	0	0	0	0	0	0	0
115	0	+1	+1	0	0	0	0	0	-1	0
135	0	+1	+1	+1	0	-2	0	-1	-1	0
150	0	+1	+1	+1	+4	-5	0	-1	-1	0
0	0	0	0	0	+2	-2	0	0	0	0



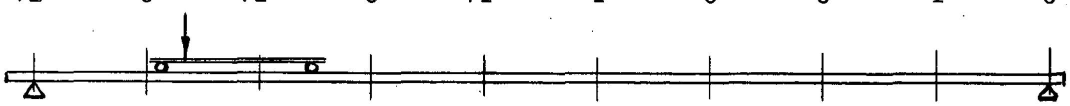
Load kips										
0	0	0	0	0	+2	-2	0	0	0	0
95	0	+1	0	+1	+2	-1	0	-1	-1	-1
135	0	+1	0	+1	+2	-1	0	-1	-1	-1
150	0	+1	+1	+1	+3	-1	-1	-1	-2	-1
0	0	0	0	0	+2	0	-1	0	0	0



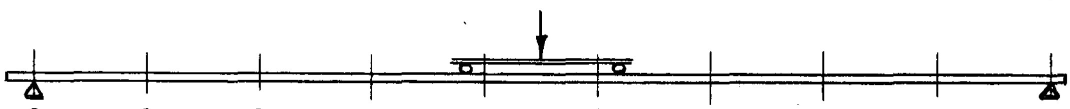
Load kips										
0	0	0	0	0	+2	0	-1	0	0	0
95	0	0	0	0	+2	0	-1	0	-1	0
135	0	0	0	0	+2	0	-1	0	-2	0
150	0	0	0	0	+2	0	-1	0	-3	0
0	0	0	0	0	+2	0	-1	0	0	0



Load kips										
0	0	0	0	0	+2	0	0	0	0	0
95	+2	+1	+2	0	0	-1	0	-1	-1	0
135	+2	+2	+2	0	-1	-1	0	-1	-2	-1
150	+2	+2	+2	-1	-2	-1	0	0	-2	-1
0	+2	0	+1	0	+1	-1	0	0	-1	0



Load kips										
0	+2	0	+1	0	+1	-1	0	0	-1	0
95	+2	+2	0	0	+1	-1	0	0	-1	0
135	+2	+3	0	-1	0	-1	0	0	-1	0
150	+3	+3	0	-1	0	-1	0	0	-1	0
0	+2	+1	0	0	+1	-1	0	0	-1	0



Load kips										
0	+2	+1	0	0	+1	-1	0	0	-1	0
186	+3	+1	+2	+5	+21	-4	0	-1	-1	0

Slip of Slab relative to Beam is POSITIVE ← → Slip of Slab relative to Beam is NEGATIVE

Fig. 26 Relation of Load to Slip along the Joint between the Slab and Prestressed Portion of the Beam