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1. SYNOPSIS

For long span flexural members, deflection limitations often control in design. A method called Preflex has been developed in Belgium and applied successfully in Europe to reduce the deflections of long span shallow depth composite steel and concrete beams.

A beam of 60-foot span and 24-inch depth comprised of an 18 WF 114 rolled steel beam encased in concrete and designed on the Preflex principles was fabricated in the United States and tested. This report describes the test of this beam and also the test of an identical beam which was not preflexed.

While the tests demonstrated the potential advantages of preflexing, they also emphasized that care must be taken in selection of materials and in the fabrication procedures to insure the full advantage of the preflexing technique.

2. INTRODUCTION

2.1. BACKGROUND

Within the past several years high strength structural steel shapes conforming to the ASTM A441 designation have become available to the construction field. Possessing a yield strength fifty percent higher than ordinary structural steel, these materials have particular application as long span flexural members. However, even though the material is applicable strengthwise, the stiffness of the material is no greater than structural steel, and hence deflection limitations may be exceeded in long span structures.

To gain the full benefit of the strength characteristics and at same time keep deformations within acceptable limits, a long span beam of this material must possess greater stiffness. The simplest solution is to use a beam of greater depth. However this would reduce clearance. Another controlling factor is now introduced, that being depth limitation. In many instances, the most desirable beam is that which gives the lowest depth to span ratio possible.

A type of composite beam which employs the Preflex technique was introduced in Belgium by Mr. M. A. Lipski about ten years ago, and has been used successfully in Europe since then. The aim of the Preflex method is to entirely encase the steel beam with concrete which is effective in adding to the stiffness of the beam. This implies that even concrete in the tension area is effective and hence does not crack. To accomplish this, it is necessary to impose precompressive stress in the concrete so that when the beam is loaded in service, this compressive stress is relieved but cracking does not occur.

In order to introduce the compressive stress in the tension area concrete, the bare steel beam is deflected by means of jacks until stresses approaching the yield point are reached in the tension flange. this operation is termed preflexing.

With the beam in this deflected position, a slab of concrete (called the Stage 1 concrete) is poured around the tension flange. When this concrete has hardened sufficiently to achieve the desired strength, the preflexing forces are released and the beam tends to return to its original profile. However, due to the presence of the Stage 1 concrete, and its composite action with the steel, the beam now has a higher stiffness and full return is not possible. Thus a higher load is now required to produce a given deflection than before with the bare steel only.

The beam is now ready to receive the Stage 2 concrete which encases the steel compression flange and the web. In this manner, a girder of low depth to span ratio and possessing high stiffness properties is obtained. In addition, all the steel has concrete cover which is not cracked under working loads.

As in prestressed concrete, creep and shrinkage effects would all but eliminate the effect of preflexing unless high strength steel and concrete materials were used.

For further information on the principles of preflexing and design procedures see references 1 through 6.

To introduce this method of construction to the United States, the consulting firm of Schupack and Zollman of Stamford, Connecticut and Newtown Square, Pennsylvania was retained to design a preflexed beam which satisfies all the requirements of American practice, using materials available in the United States, and which could be applied as a component of a useful structure.

A standard Bureau of Public Roads 28-foot roadway highway bridge 60 feet in length subjected to the AASHO H20-S16 loading was designed. Eight preflexed beams were the main flexural members. The design criteria were essentially the same as those used in Europe. Some alternations were necessary since American materials were used (See Appendix VI for design notes).

It was desired to demonstrate the capabilities of the preflexed beams as designed for this bridge by means of a series of tests. These tests were conducted at Fritz Engineering Laboratory, Lehigh University.

2.2. PURPOSES AND SCOPE

Two beams with 60-foot span, 24-inch depth, and 48-inch wide top flange were fabricated and tested. The steel elements were 18 WF 114 rolled sections. Of the two beams, only one was subjected to the preflexing operation. The preflexed beam was designated as Px-1 and the non-preflexed beam was termed Re-1. The test beams were identical in all other respects. The purpose of the two tests was to isolate and evaluate the effect of preflexing.

From these tests it was desired to prove the following points:

a. The deflection of a preflexed beam is substantially less than the deflection of a non-preflexed beam of the same dimensions and subjected to the same loads.

b. The elastic range of a preflexed beam is increased over that of an identical non-preflexed beam.

c. Cracks will not appear in the preflexed beam until a much higher load has been applied than would be the case with a non-preflexed beam.

It was also desired to obtain the proper n values (modular ratios) for the calculation of stresses and deflections in a preflexed beam.

2.3. TEST PROGRAM

The testing programs for the two beams were identical. The beams were loaded by means of two hydraulic jacks at the quarter points to produce the design moment for the preflexed beam at midspan. After unloading, two more cycles of the design load were applied. This sequence was repeated for 1.25, 1.5, and 2 times design load. With these cycles completed, the beams were then loaded to ultimate capacity.

The load at design moment consisted of two portions - a load of 6.5 kips to simulate dead load of the bridge carried by the beam and a load of 24.5 kips to simulate live load - applied at each jack. This loading produced the design moment of 465 ft.-kips at the midspan.

Tests on steel coupons and concrete cylinders accompanied the full scale beam tests. The purpose of these investigations was to determine the material properties of the beam components. In addition, preliminary static and fatigue tests were conducted to determine the suitability of stud shear connectors for transferring the shear from steel to concrete in preflexed beams. The material tests and other preliminary investigations are described in Appendices II through VI.

3. PREPARATION OF TEST BEAMS

3.1. PRELIMINARY

The 18 WF 114 steel beams for Re-1 and Px-1 were brought into the laboratory by truck. They were unloaded and placed on the floor to await preparation according to Figure 1.

Welding of end stiffeners, bearing plates, and lifting lugs was the first item accomplished. Electric arc hand welding using low hydrogen electrodes was the process employed.

Shear connectors were installed on the outer faces of both the tension and compression flanges. The layout pattern shown in Figure 1 was identical for both beams. All shear connectors were the bent stud type, one-half inch in diameter. Due to concrete cover requirements, those studs welded to the tension flange were only 1-3/4" in height, while those on the compression flange were 2-3/4" high, which is normal for this type of stud. Preliminary tests to evaluate the behavior of these studs are reported in Appendix IV.

The welding process for the shear connectors was also of the electric arc type. The studs are placed in a hand applicator and set in position. Each stud is then welded to the plate simply by pulling the trigger on the applicator. Figure 2 shows a stud being applied. The current intensity and current duration are predetermined and need not be adjusted for each weld once a satisfactory setting has been obtained.

After all welding operations on the bare steel beams were completed, the beams were placed in position for final preparation before pouring concrete. It is important to note that, in this position, the beams were inverted with respect to their final test position.

In Belgian practice, beams are usually preflexed in pairs, one beam right side up, the other beam inverted. The inverted beam is then turned over using large special wheels near the quarter points. Beams Re-1 and Px-1 were fabricated in the inverted position to help insure proper placing of the Stage 1 concrete and for general laboratory convenience.

Measurements were then taken of the cross section of both beams at five locations along their lengths. It was found that the deviation from the handbook values for the section properties was no greater than two percent in any case. Therefore, in the subsequent calculations, where these values are required, the handbook values are used.

Even though the section properties varied only slightly, the cross section of Px-1 was warped somewhat at midspan. In addition, the web of Px-1 was not located at the middle of the flanges, but was 1/4" off center; that is, the distance from the outer tip of the flange to the center of the web measured 5-3/4" on one side, 6-1/4" on the other. This may have had a significant effect on the performance of this beam which will be discussed later.

Both beams were supported at the ends of a 60-foot span. Beam Re-1 rested on simple bearing blocks as shown in Figure 3. The support arrangement for Px-1 was somewhat more complex because of the requirement that it be preflexed. At the quarter points, mechanical jacks were placed beneath the beams so as to exert an upward force on the beam. By using wide bearing blocks, it was possible to place three jacks at each quarter point. The use of three jacks at both points enabled the beam to be preflexed without twisting of the beam. This method also made the operation of restroking the jacks more easily accomplished. Throughout most of the preliminary work, prior to preflexing, Px-1 rested on these jacks.

At the ends of the 60-ft. span, Beam Px-1 was fastened to the floor through dynamometers. A detail of this arrangement is shown in Figure 4. The dynamometers, which had been previously calibrated, were the measuring elements for the preflexing loads. When it was desired to take zero readings, the dead load of the steel beam was relieved from the quarter point jacks and carried at the ends by these dynamometers. Under these conditions it was necessary to prevent longitudinal movement of the beam by using a horizontal strut to a fixed column.

Electric strain gages were applied at five stations along each beam according to Figure 5. The gages were the Baldwin SR-4 Type A-1 which measure the strain over a one inch gage length. A typically gaged section is shown in Figure 6. To prevent damage from the moisture in the concrete, the gages were coated with a waterproofing compound.

The steel reinforcement for the tension flange concrete (Stage 1) was then installed on both beams. This reinforcement pattern is shown in Figure 1.

3.2 PREFLEXING Px-1

At this point, Px-1 was ready for the preflexing operation. Before preflexing, the compression and tension flanges were braced against lateral movement at the ends, quarter points, and centerline. The jacks, located at the quarter points, applied the upward forces while the dynamometers at each end held the beam down and measured the load. The operation required three cycles of loading to relieve the residual stresses present in the beam due to straightening and cambering at the mill. The preflexing is discussed more fully in Section 5.

3.3 STAGE 1 CONCRETE

With the preflexing of Px-1 completed, both beams were ready to receive the Stage 1 concrete which encased the tension flanges as shown in Figure 1. The formwork was constructed so that the weight of the concrete was carried directly to the floor as much as possible. Figure 7 shows the formwork for Re-1 in place and Px-1 partially preflexed.

The Stage 1 concrete for both beams was poured the same day. A mix designed for 5000 psi compressive strength at 28 days was used. Details of all concrete mix designs are given in Appendix III. Separate batches of three cubic yards were used in each beam. The dry materials were added to a six cubic yard transit mix truck and transported to the laboratory. At the laboratory, water was added until a concrete having a slump of 2-1/2" was obtained.

The pouring operation is shown in Figure 8. A one-half cubic yard bucket was hung from a crane and transferred the concrete from the truck to the beam. Vibrators applied from both sides forced the concrete to flow fully around the web and under the steel flange. Final finishing was performed with a steel trowel.

For the first three days of curing, the wooden forms covered all concrete surfaces except the top on which damp burlap was spread. On the thrid day of curing the forms were stripped. The wet burlap remained on the top surface of the concrete and it hung down to cover the sides. Wet burlap was wrapped around the entire beam section on the fifth day of curing. This was done to improve the curing conditions.

On the twelfth day Whittemore strain gage points were then installed on the Stage 1 concrete at each station according to Figure 5. These points allow strain to be measured over a gage length of 10 inches.

The wrapped around curing arrangement continued until the beams were rolled over in preparation for the Stage 2 concrete.

3.4 STAGE 2 CONCRETE Re-1

Fourteen days after pouring the Stage 1 concrete, Re-1 was rolled over so that the Stage 2 concrete could be poured. The rolling process utilized wooden wheels at the ends. Figure 9 shows the beam about to be rolled. The curing of the Stage 1 concrete on Re-1 was stopped the next day.

Details of the Stage 2 reinforcement are shown in Figure 1. The forms were constructed so that the entire Stage 2 concrete weight was carried by the beam. Twenty days after the Stage 1 concrete was poured, the Stage 2 concrete was poured on Re-1. A single batch of six cubic yards was poured similarly to that for Stage 1. The forms were removed six days after pouring. Curing of the Stage 2 concrete consisted of wet burlap on the top surface while the forms were on, and wet burlap on the top surface and sides of the slab after the removal of the forms and until the end of curing, twenty-three days later. Whittemore plugs were installed on the Stage 2 concrete according to Figure 5. The test on Re-1 was started thirty-four days after pouring the Stage 2 concrete.

3.5 STAGE 2 CONCRETE Px-1

Twenty-eight days after the Stage 1 concrete was poured on Px-1 the curing was stopped and the preflexing load was released. It was not possible to turn the beam over on the day of detensioning, so approximately one-third of the original preflexing force was again jacked into the beam. This was done to minimize creep of the Stage 1 concrete.

Px-1 was turned over three days after the first release of preflex loads. The turning procedure was not the same as that for Re-1. Instead of using wooden wheels at the ends, the beam was supported at five points along its length by wooden timbers. Eccentric lifting forces located near the third points were applied so that the beam rolled as it was lifted. Px-1 was then placed on pedestals at the ends in order to receive the reinforcement and formwork for the Stage 2 concrete.

Details of the Stage 2 reinforcement and formwork were the same as for Re-1. The Stage 2 concrete was poured thirty-eight days after Stage 1 had been poured. Separate batches of three cubic yards each were poured similarly to that for Stage 1. For the first twenty-two days of curing the Stage 2 concrete, the forms remained on and wet burlap was spread over the top surface. After this time and until the test began, seven days later, wet burlap remained on the top surface. Whittemore plugs were installed on the Stage 2 concrete prior to the test. Following is a table giving the sequence of important events in the pouring and curing of concrete for both Re-1 and Px-1.

TABLE 1 TIME SEQUENCE OF EVENTS IN THE POURING AND CURING OF CONCRETE

Day	Re-1	Px-1
1	Stage 1 concrete poured	Stage 1 concrete poured
4	Formwork removed	Formwork removed
6	Wet burlap wrapped around	Wet burlap wrapped around
12	Whittemore plugs installed	Whittemore plugs installed
15	Beam rolled	
16	Curing of Stage 1 concrete stopped	
21	Stage 2 concrete poured	
27	Formwork removed	
29		Curing of Stage 1 concrete stopped First release of preflexing load
32		Preflexing load released Beam turned
39		Stage 2 concrete poured
50	Curing of Stage 2 concrete stopped	
55	Testing began	
61		Formwork removed
68		Curing of Stage 3 concrete stopped Testing began

3.6 DEFLECTION AND STRAIN READINGS

To obtain information on the behavior of both beams before the testing program began, readings of the vertical deflections and of the steel and concrete strains were taken at appropriate times during the preparation of beams Re-1 and Px-1.

4. BEHAVIOR OF BEAM Re-1 PRIOR TO TEST

4.1 DEFLECTIONS

The initial camber of Re-1 was measured as it rested on its end supports in the inverted position. Subtracting the dead weight deflection, the camber as measured at midspan was 3.38 inches. The east and west quarter point offsets from the chord were measured as 2.37 and 2.46 inches respectively.

After pouring the Stage 1 concrete and rolling the beam into position for the pouring of the Stage 2 concrete, the beam maintained an upward camber of 2.31 inches at midspan. The dead weight of the Stage 2 concrete caused a deflection of 1.96 inches, so that after pouring the Stage 2 concrete, an upward camber of 0.35 inches was present at midspan.

4.2 STRAINS

Steel strains at various steps in the preparation of Re-1 are shown in Figure 10 for Station 1. A strain of 345 micro-inches is indicated in the outer fiber of the tension flange prior to testing. Using a modulus of elasticity of 28.5 x 10^6 psi for the steel, this strain corresponds to a stress of about 10 ksi. At Stations 2 and 2' the stress was found to be 9 ksi. The corresponding stress at Station 3' was found to be 6.5 ksi. The scatter of the gage readings at Station 3 was so great that no reasonable conclusions can be drawn.

Concrete strains for the Stage 1 concrete were not available since tensile cracks formed when the beam was rolled. The cracks in all cases passed directly through the Whittemore gage points making strain readings impossible.

Gages were not installed on the Stage 2 concrete in time to measure the shrinkage effect prior to the test. However, if it is assumed that the steel strains in the vicinity of the top flange are the same as the concrete, then Figure 10 shows a strain of 100 micro-inches per inch due to shrinkage of the Stage 2 concrete. This strain is the difference between the strain readings taken after pouring the Stage 2 concrete and prior to the test.

4.3 ANALYSIS OF MEASURED STRESSES AND DEFLECTIONS

In analyzing the observed behavior of the beam, considerations must be taken which would be very difficult to predict beforehand. Specifically, the proportion of the Stage 1 concrete weight carried by the forms is not easily determined. Even though it was attempted to support the beam so that the entire concrete weight was transferred directly to the floor, the weight of the concrete resting directly on the steel flange was no doubt carried largely by the steel beam.

Another significant factor was the formation of tensile cracks in the concrete sometime during the rolling of the beam. Calculations show that the tensile stress induced in the outer fiber due to rolling would be over 600 psi. A stress of this magnitude would be sufficient to and did cause cracking.

The behavior of the beam after cracking is extremely difficult to predict accurately for two reasons. First of all, it is not known precisely what stress will cause the cracks to form. Secondly, the redistribution of stress at cracking involves quite a detailed analysis.

For the purposes of checking the magnitudes of the readings taken, a theoretical analysis was performed based on the following assumptions:

1. When the Stage 1 concrete was poured, half the weight was carried by the beam, and half the weight was transferred directly to the floor by the forms.
2. Tensile cracks formed in the Stage 1 concrete half-way through the rolling operation. The concrete stress was zero at cracking.
3. Strains obey elastic beam theory. Material properties are taken from cylinder and coupon tests explained in Appendices II and III.
4. Shrinkage effects are neglected.

If a comparison of calculated values with observed results is to be made, consideration must be taken of the fact that the exact location of the strain gages with respect to the cracks is not known. Therefore only a range of strain can be defined, i.e. the strain gage reading should lie between the value computed for the cracked section and that value calculated for an uncracked section. The theoretical analysis is given in Appendix I.

The following table gives a comparison of the measured steel stresses as indicated in Figure 10, with the calculated values.

TABLE 2 STEEL STRESSES IN Re-1 PRIOR TO TESTING

Loading Condition	σ_1 (ksi)			σ_1 (ksi)		
	Measured	Calculated		Measured	Calculated	
		Cracked	Uncracked		Cracked	Uncracked
Dead Weight of WF	- 2.97*		- 2.97	+ 2.97*		+ 2.97
After Addition of Stage 1	- 4.56		- 4.98	+ 3.98		+ 4.98
After Rolling Beam	+ 3.28	+ 4.24	+ 0.36	- 8.27	- 6.47	- 5.72
After Addition of Stage 2	+ 11.7	+12.91	+ 4.25	- 16.7	- 15.14	- 13.51
Prior to Test	+ 9.7	+12.91	+ 4.25	- 19.9	- 15.14	- 13.51

*These stress values due to the dead weight of the steel beam were actually calculated.

The following table gives a comparison of the measured midspan deflections with the calculated values.

TABLE 3 MIDSPAN DEFLECTIONS OF Re-1 PRIOR TO TESTING

Loading Condition	Midspan Deflection (inches)		
	Measured	Calculated	
		Cracked	Uncracked
Dead Weight of WF	3.94 $\downarrow$		3.99 $\downarrow$
After rolling beam	2.31 $\uparrow$	2.22 $\uparrow$	2.74 $\uparrow$
After addition of Stage 2	0.35 $\uparrow$	0.25 $\uparrow$	1.42 $\uparrow$

4.4 CONDITION OF Re-1 JUST BEFORE TESTING

To summarize the condition of Re-1 just before testing, the following points are restated.

1. The maximum stress in the tension flange was 10 ksi at midspan.
2. The Stage 2 concrete had a shrinkage strain of approximately 100 microinches per inch.
3. The camber of the beam was 0.35 inches measured at midspan.
4. The Stage 1 concrete was cracked throughout the length of the beam.

5. BEHAVIOR OF BEAM Px-1 PRIOR TO TEST

5.1 PREFLEXING

The preflexing forces were applied upwards at each quarter point of Px-1 by means of mechanical jacks. The end reactions were provided by dynamometers which also measured the load applied by the jacks.

On the first application of load excessive yielding began at diagonally opposite flange tips at Station 1 before a stress of 30,000 psi was reached. (30,000 psi is only 65% of the coupon yield strength.) This type of yielding pattern indicated that high residual stresses had been introduced by cold-bending the beam to remove a lateral sweep which was present after cooling. The initial lateral sweep may have been due to the eccentricity of the web described earlier.

Having observed this yielding, the preflexing was begun again and more yielding was noted when the full preflex load was finally applied.

Upon release and subsequent reloadings, the load-strain relationship was linear indicating that the effect of the residual stresses had been eliminated in this region.

5.1.1 Deflections

The initial camber of the inverted beam was measured to be 6.62 inches as it rested on its end supports. Subtracting the dead weight deflection, the camber was computed as 6.1 inches.

Figure 11 shows the load vs. midspan deflection for the entire preflexing operation. The circled numbers refer to the sequence of loading. The zero for the deflection values refers to the beam resting on end supports.

A total deflection of 13.7 inches is shown for the full preflexing load of 51.7 kips applied at each jack. With the inverted beam in its final preflexed condition, the upward deflection from the horizontal was 7.6 inches. Prior to the application of the final preflexing load, a residual deflection of 4.2 inches was present. This indicated that of the original 6.1 inches camber only 1.9 inches remained. The unexpectedly large loss was due to the high residual stresses present in the beam when it was received.

5.1.2 Strains

The effect of the high residual stresses present in the beam caused uneven yielding to take place much earlier than anticipated. However, after successive cycles of preflexing to higher loads a greater range of elastic behavior was obtained. Finally, the last application of the preflexing loads produced elastic behavior in the strains for the full preflexing range.

An average elastic strain of 1400 microinches per inch was recorded at the outer tensile fibers at Stations 1, 2, and 2' with the final preflex load applied. This value of strain corresponds to a stress of 40,000 psi.

At Station 3 and 3' the average strain was 670 microinches per inch, corresponding to a stress of 19,100 psi.

Inelastic strains as high as 5100 microinches per inch were recorded in the midsection of the beam before achieving the elastic behavior for the full range of loads.

With the inverted beam in its final preflexed condition, the Stage 1 concrete was poured around the tension flange.

5.2 CURING PERIOD

The curing period for Px-1 was essentially the same as for Re-1 except for the release of preflexing loads after the curing of the Stage 1 concrete.

5.2.1 Deflections

After release of the preflexing loads, and rolling the beam into position for the pouring of the Stage 2 concrete, the beam had a sag of 1.64 inches measured at the centerline. The dead weight of the Stage 2 concrete caused a deflection of 1.02 inches, so that with the beam ready for testing, a sag of 2.66 inches was present at midspan.

In the design of Px-1, it was intended to have a slight upward camber when completed as was present in Re-1. However, the loss of 4.2 inches of the original bare steel beam camber as explained earlier caused the sag.

5.2.2 Strains

Steel and concrete strains at various steps of the curing period of Px-1 at Station 1 are shown in Figures 12 and 13. Since behavior at Station 2' proved significant during the test, Figures 14 and 15 are included showing the strain at this station.

For intervals of several days between readings, the electric strain gages on the steel could not be depended on to give an accurate measure of the strains. It was attempted to use reference gages in order to detect changes in the measuring system not caused by strains and some benefit was gained through the use of these reference gages. However, based on electric strain gage readings alone, an accurate determination of strains during the curing period could not be obtained.

The Whittemore gages on the Stage 1 concrete could be relied on for determination of long term changes in strain since all readings are referred to a standard. The standard is known not to change.

By considering the two measuring methods together, the steel strains during the curing period could be defined with some degree of confidence. However, these strains as shown in Figures 12 and 14 should not be considered to have the same accuracy as the strains measured during preflexing and the test.

The strains given just prior to testing have been converted to stresses using the appropriate value of the modulus of elasticity. Stresses just prior to testing at critical points in the steel and concrete are given in the following table.

TABLE 4 STRESSES IN Px-1 PRIOR TO TESTING

STATION	1	2	2'
Steel stress, tension flange (ksi)	25.0	24.6	22.7
Concrete stress, bottom fiber (ksi)	- 1.5	- 1.2	- 0.77

5.3 ANALYSIS OF MEASURED STRESSES AND DEFLECTIONS

As in the case of Re-1, the behavior of Px-1 during the curing period is very difficult to predict beforehand. Again the proportion of the Stage 1 concrete supported by the beam is not known. In addition to shrinkage, the effect of creep is present in Px-1.

A theoretical analysis of the pretest behavior has been performed. No considerations of creep and shrinkage were taken into account. A comparison of the theoretical with the observed behavior should give an estimate of the losses due to creep and shrinkage.

The following assumptions were made:

1. When the Stage 1 concrete was poured, half the weight was carried by the beam, and half the weight was transferred directly to the floor by the forms.
2. Strains obey elastic beam theory. Material properties are taken from cylinder and coupon tests explained in Appendices II and III

The theoretical analysis is given in Appendix I.

Significant stresses, both theoretical and measured are presented below. Values of stress for Station 2' are indicated since during the test this was found to be the most critical section.

TABLE 5 STRESSES IN Px-1 AT STATION 1 PRIOR TO TESTING

Loading Condition	σ_1 ksi (Steel)		σ_{c1} ksi (Concrete)	
	Measured	Calculated	Measured	Calculated
Dead Weight of WF	- 3.00	- 3.97	--	--
After Preflexing	39.6	39.0	--	--
After Second Release	18.8	19.7	- 3.56	- 3.71
After Rolling	24.5	25.4	- 2.41	- 2.61
At Start of Test	25.0	28.9	- 1.50	- 1.94

TABLE 6 STRESSES IN Px-1 AT STATION 2' PRIOR TO TESTING

Loading Condition	σ_1 ksi (Steel)		σ_{c1} ksi (Concrete)	
	Measured	Calculated	Measured	Calculated
Dead Weight of WF	- 2.82	- 2.78	--	--
After Preflexing	39.8	39.4	--	--
After Second Release	17.2	20.3	- 3.07	- 3.71
After Rolling	21.8	25.9	- 1.73	- 2.61
At Start of Test	22.7	29.2	- 0.77	- 1.98

Calculation of Deflections at Station 1

Calculations of the midspan deflections during the curing period were performed. These were based on the same assumptions and data as given for the stress computations. The table below compares the theoretical values with those measured.

TABLE 7 MIDSPAN DEFLECTIONS OF Px-1 DURING CURING PERIOD

Loading Condition	Midspan Deflection (inches)	
	Measured	Calculated
Dead Weight of WF	2.30 ↓	2.30 ↓
After Preflexing	7.21 ↑	7.22 ↑
After Release	0.15 ↓	0.42 ↓
After Rolling	1.64 ↓	2.31 ↓
At Start of Test	2.53 ↓	3.48 ↓

Discussion of Losses

The losses due to creep and shrinkage may be divided into two periods (between the first release of preflex and the rolling) and (between the rolling and the start of the test). If the steel stress at the outer tensile fiber at Station 1 is taken as the basis for evaluation of losses, the following values are obtained.

TABLE 8 LOSSES IN Px-1 BASED ON STATION 1

Period	Initial Stress (ksi)	Loss (ksi)	% Loss
First Release - Rolling	20.6	1.75	8.5
Rolling - Test	27.5	2.61	9.5

If the calculated stress at the Start of the test is compared with the measured value a loss of 17% is indicated; that is, theoretical calculations based on a 17% loss due to creep and shrinkage would give nearly the actual stress condition.

Inspection of the data showed results consistent with that above except at Station 2'. Greater strains were recorded at this location than in the other sections when the beam was rolled. Although every effort was made to support the beam uniformly during this time the concrete was overstressed enough to cause a significant plastic deformation at this station. At the time of the test, a compressive stress of only 0.77 ksi remained in the outer fiber of the Stage 1 concrete. A stress of 1.5 to 2.0 ksi was expected.

5.4 CONDITION OF Px-1 JUST BEFORE TESTING

To summarize the condition of Px-1 just before testing, the following points are restated.

1. The maximum stress in the tension flange was 24,970 psi at midspan.
2. A loss of 17% of the preflexing stresses due to shrinkage and creep was recorded.
3. A significant loss was noted at Station 2' where the maximum compressive stress in the outer fiber of the concrete was only 0.77 ksi.
4. The beam had a sag of 2.7 inches at the centerline prior to the start of the test.

6. DESCRIPTION OF TESTS

6.1 TEST PROGRAM

The test program for Re-1 is shown in Figure 16. Intermediate load levels are indicated. Figure 17 shows the same for Px-1. Testing of each beam covered four days. Each evening the maximum load reached up to that time was held overnight. This led to sustained periods of application of design live load, 1.5 times design load, and 2 times design load.

Note that jack loads of 6.5 kips were used to simulate the superimposed dead load that the beams would have carried in a bridge. To these jack loads were added increments to give some factor of the design live load moment for Beam Px-1. That is, a jack load of 6.5 kips at each quarter point corresponds to zero live load moment, and an increment of 24.5 kips (31.0 kips Jack Load) corresponds to design moment. That is, 1.5 times design live load equaled $\left[6.5 + 1.5(24.5)\right]$ kips.

In the last phase of loading, i.e. when loads approaching ultimate were applied, increments of deflection rather than load were used. As the deflection was held constant, the load decreased slightly. The average of the loads before and after taking readings was used as the test load. The difference in the two loads amounted to no more than two kips in any case.

6.2 TEST SETUP

The overall test setup is shown in Figure 18. The arrangements for Re-1 and Px-1 were identical. The west end of the beam was supported by a pedestal allowing rotation only. At the east end, the support permitted longitudinal movement as well as rotation.

A test frame from which was suspended a single Amsler jack was placed at each quarter point. The jacks are hydraulically operated, each having a static loading capacity of 200^k. Pressure was applied to the hydraulic system by means of an Amsler pendulum dynamometer which also measured the applied loads. Since the rams of the two jacks have equal surface area, and the jacks are connected to the same hydraulic system, the applied load at the quarter points are necessarily equal.

The maximum stroke of the jacks is five inches. Therefore restroking was necessary at several intervals during the test. Restroking was done at times when the load was completely removed and toward the end of the test when the load was taken beyond 2 times design load.

During these latter stages of testing the restroking was performed after the beam was blocked to hold the maximum previous deflection. The blocking arrangement, shown in Figure 19, consisted of two channels at each load point bolted to the test frame. Between the channels and the beam were placed steel wedges to take up the clearance.

6.3 INSTRUMENTATION

Scales graduated to 1/100 of an inch were placed at the centerline, quarter points, and ends of the two beams. Readings on these scales were taken at each test load with a Wild level.

The purpose of the end scales was to correct for deflection at the supports. In the reduction of the data these changes proved to be small and were subsequently neglected. The quarter point scales served as an approximate check of the midspan readings.

As previously described, electric strain gages of the Baldwin SR-4 type had been placed on the steel at five stations on each beam. Whittemore plugs were placed on both the Stage 1 and Stage 2 concrete at the same five stations as the steel. The location of both the steel and concrete gages is shown in Figure 5.

Readings of the strain gages were taken at each test load. The Whittemore gages on the Stage 1 concrete of Re-1 were not recorded since extensive cracking of the concrete had occurred before any test load was applied.

Ames dial gages were placed at the ends of each beam to record any slip between the steel and concrete at these points.

The air temperature in the vicinity of the testing area was recorded at each test load.

7. TEST RESULTS: BEAM Re-1

7.1 DEFLECTIONS

The load vs. centerline deflection envelope for the entire test of Re-1 is shown in Figure 20.

To show the deflection under given percentages of live load, Figures 21, 22, 23, and 24 have been presented. These figures correspond to 1.0, 1.25, 1.5, and 2 times design load respectively. Each cycle of loading at a given load level is included. The dashed line indicates the theoretical load-deflection relationship. Residual deflections present after the first cycle are clearly evident. Residual deflections at the start of each load sequence were used as the new zero points for deflections. For later convenience of comparison, corresponding curves for Px-1 are shown on these figures.

7.2 STRAINS

To demonstrate the linear distribution of strains at a cross section Figure 25 is presented. This gives the distribution at Station 1 for the first five load increments. Also indicated are the concrete stresses corresponding to the given compressive test strains, and the tensile stresses in the steel assuming that the recorded strains are completely elastic. Similar plots could be drawn for the other Stations.

Figure 26 shows the strain distribution at the final test load for all five stations.

Figure 27 indicates the strain distribution at Station 1 with the design load applied. Two experimental lines are shown -- one corresponding to the change in strain from zero load to the first application of load, the other indicating the strain increment for succeeding applications. These are instantaneous strain increments only -- strains under constant loading have been subtracted. The dashed line is the theoretical line which will be explained in the following chapter.

Figures 28, 29, and 30 give the same information for loads of 1.25 times design, 1.5 times design, and 2.0 times design load.

Figures 31, 32, 33, and 34 apply to Station 3. With one exception these figures give the same information as explained in the preceding paragraphs for Station 1. The exception is that only one experimental line is shown. Subsequent loadings produced the same readings as obtained in the first loading.

7.3 END SLIPS

The maximum slips between the concrete and steel at each end are shown in the following table:

TABLE 9 CONCRETE SLIP OF Re-1

Gage Location	Slip (inches)
Top Slab West	0.0020
Top Slab East	0.0016

7.4 TEMPERATURE

The air temperature in the testing area recorded at each load ranged from 62° F. to 73° F.

7.5 OTHER OBSERVATIONS INCLUDING CRACKING NOTES

After rolling Re-1, tensile cracks in the Stage 1 concrete were noticed throughout the length of the beam. Since the cracks passed through Whittemore gage points in all cases, concrete strain observations were impossible. The spacing of the cracks was 7-1/2" or the same as the spacing of the transverse reinforcement.

The width of nine cracks, designated as A through J, which had formed in the Stage 1 concrete were measured during the course of the test with a microscope equipped with a graduated lens. Only on cracks in the pure bending region of the beam were these measurements taken. The increase in crack widths is indicated in the table below:

TABLE 10 CRACK WIDTHS IN Re-1

Load No.	Jack Load (kips)	Width of Cracks (in. x 10^3)								
		A	B	C	D	E	F	G	H	J
2	6.2	6								
3	15.3							12	11	11
11	31.0		10		20		30		13	14
23	37.2						40			
28	37.2					25				
48	43.5						55	16	15	16
49	50.0					40	65			
51	56.0						75			
53	6.2						50			
56	56.0					45	80			

With the load at 31.0 kips (Design Load) about ten cracks in the web concrete were observed to extend within one-half inch of the underside of the top concrete slab. The pattern is shown in Figure 35.

After reaching 37.2 kips, separation between the underside of the top steel flange and the concrete was observed at the end of the beam. This separation was not noted however on the top side. At the end of the test tensile cracks had progressed to 4-1/2 inches from the top of the concrete.

The final test loading was 70.5 kips applied at each jack (2.56 times Design Load). This condition is shown in Figure 36. The maximum deflection was 31 inches at the centerline. There was no crushing of the compression concrete.

8. THEORETICAL ANALYSIS OF TEST RESULTS - Re-1

To analyze the strains during the testing phase of the beam, the composite cross section with the Stage 1 concrete completely cracked through was used. It is assumed that the strains obey elastic beam theory. Material properties are taken from cylinder and coupon tests explained in Appendices II and III.

A sample calculation follows:

$I = 3934 \text{ in.}^4$

$S_1 = 281 \text{ in.}^3$ (Section modulus at outer fiber of steel flange which will be the tension flange in the final position.)

$S_{c2} = 523 \text{ in.}^3$ (Section modulus at outer fiber of Stage 2 concrete flange)

For Station 1 with design load applied:

$M = 5570 \text{ in.-kips}$

$$\epsilon_1 = \frac{M}{S_1 E} = \frac{5570}{281 \times 28.5 \times 10^3}$$

$$\epsilon_1 = 695 \times 10^{-6} \text{ in/in}$$

$$\epsilon_{c2} = \frac{-M}{S_{c2} E} = \frac{-5570}{523 \times 28.5 \times 10^3}$$

$$\epsilon_{c2} = -374 \times 10^{-6} \text{ in/in}$$

Using the values calculated for ϵ_1 and ϵ_{c2}, the theoretical line may be drawn in Figure 27. These strains also apply to Stations 2 and 2' since the moment is the same.

Likewise, for deflections, theoretical values may be computed based on the cracked section properties, neglecting the stiffening influence of the Stage 1 concrete.

For design load (superimposed dead plus live load applied by jacks):

$$\delta_{\mathbb{C}} = \frac{11}{96} \times \frac{ML^2}{EI}$$

$$\delta_{\mathbb{C}} = \frac{11}{96} \times \frac{5570 \times 60^2 \times 144}{28.5 \times 10^3 \times 3934}$$

$$\delta_{\mathbb{C}} = 2.96''$$

Deflections at other loads may be computed in a similar manner. The theoretical load vs. deflection line is shown in Figures 21, 22, 23, and 24.

Based on the yield stress of the steel and ultimate strength of the concrete, an ultimate moment calculation was performed to check the final test load and the final location of the neutral axis. The ultimate load was calculated to be 66.7 kips at each jack and the location of the neutral axis to be 4.5 inches from the top surface of the concrete.

9. DISCUSSION OF TEST RESULTS: BEAM Re-1

9.1 DEFLECTIONS

Figure 21 shows the amount of residual deflection present after the first loading to design load. Since the succeeding loadings followed essentially a linear path, it may be said that the first loading served to relieve the residual stresses present in the beam.

None of the test results agree very closely with the theoretical line. This is due to two influences; residual stresses in the steel and the stiffening effect of the Stage 1 concrete. On the first loading, residual stresses caused the beam to deflect more than predicted. The succeeding load cycles, with the residual stresses relieved and with the benefit of the Stage 1 concrete, showed a greater stiffness than predicted.

The effect of creep under sustained loading is indicated by the increase in deflection when 31.0 kips (design load) was held for sixteen hours.

Figure 22 shows essentially the same information as Figure 21 but in this case the maximum load was 37.2 kips (1.25 times design load). There was no period of constant loading; therefore no creep deflection is present. Since the increase over the previous load (31.0 kips) is only about 20 percent, the residual deflection is markedly less in this case.

Figure 23 shows the cycles of load at 43.5 kips (1.5 times design load). The residual deflection is of the same order as for 37.2 kips since the increment of load increase is the same. A creep deflection is noted at 43.5 kips; the load was held for nineteen hours.

Figure 24 shows the cycles of load at 56.0 kips (2 times design load). The residual deflection due to this load is relatively high since the increment of load increase was twice that for 37.2 and 43.5 kips.

Theoretically, a load of 64 kips was required to produce yielding stresses (about 44 ksi) in the tension flange. The applied load was 56 kips, therefore the bending stresses alone were not responsible for the residual deflection. The residual deflection must be attributed to the sum of the bending stresses and residual stresses having exceeded the yield stress.

9.2 <u>STRAINS</u>

Figure 25 is presented merely to demonstrate that the strains behaved as expected for the first loading cycle, i.e. there is a linear distribution of strains at each load increment and the neutral axis stayed at approximately the same location. The steel stress indicated would be true if Re-1 behaved perfectly elastic. However, as the later unloadings showed, a significant amount of these strains are plastic, therefore, the stresses shown are not valid.

Figure 26 gives the strain distribution at ultimate load. Stations 1, 2, and 2' which were in the pure bending region, show that the beam curvature was much greater in this section than at the end sections where Stations 3 and 3' were located. The position of the neutral axis is shown to be about five inches from the top of the slab which checks with the progress of tensile cracks and the ultimate moment calculation.

A comparison of test strains with the theoretical values at Station 1 for design load applied is given in Figure 27. As described previously for the deflections, the first loading strains exceed the predicted because of the presence of residual stresses. Upon loading the second time, after relief of the residual stresses, the recorded increment of strain is less than the theoretical. The presence of the Stage 1 concrete around the gages stiffens the beam in that area and does not allow as great a strain as would be at the cracked sections.

Figures 28, 29, and 30 give the same information as described above for 1.25, 1.5, and 2 times design load respectively.

Figures 31, 32, 33, and 34 apply to Station 3', the location halfway between the west load point and the reaction. In this case, the first and second loadings at each level were nearly identical, indicating that the stresses were not high enough to cause yielding when added to the residual stresses. Again, the theoretical line gives higher strains than the test line due to the presence of the Stage 1 concrete.

9.3 END SLIPS

The end slips recorded are very small when compared to the magnitude of slips obtained in other tests of composite beams. Since the bond did not appear to have broken on the top of the steel flange, it seems that these slips had little significance.

9.4 TEMPERATURE

The variation of temperature was not considered in any of the test readings.

9.5 CRACKING

The cracking was visible to the naked eye before any load was applied. If the usefulness of the beam were to be dependent on the presence of cracks, then in that sense the beam was never satisfactory.

10. TEST RESULTS: BEAM Px-1

10.1 DEFLECTIONS

The load vs. centerline deflection envelope for the entire test of Px-1 is shown in Figure 37. On Figures 21, 22, 23, and 24 the percentage of live load vs. centerline deflection is presented. For comparison purposes, the corresponding curves for Re-1 are shown.

10.2 STRAINS

To demonstrate that the distribution of strains at the cross section is linear, Figure 38 is presented. This figure refers to Station 1. Similar results were indicated at the other stations. The concrete stress corresponds to the compressive test strains, and the stress in the steel corresponds to the tensile test strains. These stresses are based on the assumption of elastic behavior.

Figure 39 shows the strain distribution at the final test load for all five test stations.

Figure 40 indicates the strain distribution at Station 1 with the design load applied. Unlike Re-1, only one experimental line is drawn. This is due to the absence of residual strains after the first loading of Px-1. Two theoretical lines are presented however. Since cracking occurred prior to the design load, only a theoretical range of strain can be defined, as explained before for Re-1. These theoretical lines will be discussed in the following chapter.

Figures 41, 42, and 43 indicate the strain distribution for loads of 1.25, 1.5, and 2.0 times design load.

Figures 44, 45, 46, and 47 apply to Station 3. Only one theoretical line is shown at this station since cracking was not observed at this location until much later.

10.3 END SLIPS

The maximum slips between the steel and concrete at each end of Px-1 are shown in the following table:

TABLE 11 CONCRETE SLIP IN Px-1

Gage Location	Slip (inches)
Top Slab West	0.0015
Top Slab East	0.0012
Bottom Slab West	0.0042
Bottom Slab East	0.0037

10.4 TEMPERATURE

The air temperature in the testing area recorded at each load ranged from 62° F. to 73° F.

10.5 OTHER OBSERVATIONS INCLUDING CRACKING NOTES

On the first cycle of loading tensile cracks appeared in the Stage 1 concrete at a load of 25 kips per jack. The cracks occurred first near Station 2' on the south side of the beams. As the load was increased to 32.5 kips, additional cracks appeared throughout the section between the load points. Upon release of the 32.5 kip load, it was observed that the cracks closed at 16.8 kips.

Four typical cracks in the pure bending region of the Stage 1 concrete were observed during the course of the test. These are designated as A, B, C, and D. A_s, D_s, C_s, D_s refer to measurements taken on these cracks near the south edge of the beam. Likewise, A_n, B_n, C_n, D_n refer to measurements on the north edge of the beam.

Measurements of crack widths for these four cracks are shown in Table 12.

TABLE 12 CRACK WIDTHS IN Px-1

Load No.	Load per jack (kips)	Width of Cracks (in x 10^3)							
		A_s	A_n	B_s	B_n	C_s	C_n	D_s	D_n
4	25	3	0	2	0	0	0	0	0
5	32.5	4	3	3	1	2	2	2	1
6	32.5	4	3	3	1	2	2	2	1
7	16.8	0	1	1	0	0	0	1	0
8	7.7	0	0	0	0	0	0	0	0
9	16.8	0	1	0	0	0	1	1	0
10	32.5	3	2	3	2	2	1	2	2
11	16.8	0	0	1	0	0	1	1	0
12	7.7	0	0	0	0	0	0	0	0
13	32.5	4	1	4	2	2	2	2	2
14	32.5	4	1	4	2	2	2	2	2
15	7.7	0	0	0	0	0	0	0	0
16	32.5	3	3	5	2	2	1	3	2
17	39.2	5	4	5	3	3	1	4	2
25	37.2	5	4	4	4	3	2	2	3
36	43.5	6	5	7	3	5	4	6	3
48	15.3	6	5	8	4	5	5	6	6
57	56	7	6	11	7	6	7	7	8
--	0	1	1	3	0	1	1	0	2

In addition, general remarks on the appearance of the cracking in Px-1 were recorded. They are as follows:

Load No.	Load per Jack (kips)	Remarks
4	25	Four cracks, 0.002", appeared between west quarter point and midspan - spaced about 2" - located on south face of bottom flange only.
-	29	Eight additional cracks, previous cracks widened to 0.003".
5	32.5	Ten additional cracks appeared on south face, all ranging between 0.001" and 0.003" in width. Spacing was approximately 15". Six cracks were completely through Stage 1 concrete and extended to base of web.
	25	Most cracks disappeared. Only about six were still 0.001" in width.
7	16.8	All cracks disappeared.
8	7.7	No cracks visible.
9	16.8	Only crack A_n reopened to 0.001".
10	32.5	Same cracks as in Load 5.
11	16.8	Cracks disappeared almost completely.
12	7.7	No cracks were visible.
13	32.5	Same cracks as in Load 10, plus about twelve additional which were 0.001" to 0.002" wide extending 6" into web concrete.
14	32.5	No change after seventeen hours at load
15	7.7	All cracks disappeared.
16	32.5	Same cracks as Load 14, some slightly larger.
17	39.2	More extensive cracking, web cracks extended 6" up into concrete.
36	43.5	About thirty cracks formed, some were spaced at 7-1/2".
57	56.0	One crack on the north side west of the centerline widened to 0.020".

Figures 48 and 49 show the cracking patterns in the Stage 1 concrete and the web at ultimate load.

With the load at 56 kips, separation of about 1/16 of an inch was observed between the underside of the top steel flange and the concrete adjacent to it. This behavior was noted in Re-1 also.

At the end of the test, the cracks in the concrete had extended to within 4-1/2" of the top of the slab. The final deflection was 30-1/4" at the centerline at a load of 68 kips. The test was stopped at this load, though there was no crushing of the compression concrete. Figure 50 shows the beam in this condition.

11. THEORETICAL ANALYSIS OF TEST RESULTS - Px-1

11.1 CONCRETE AND STEEL STRAINS

A theoretical analysis of the behavior of Px-1 was performed. This work gives the strains and deflections for each test load based on the following assumptions:

a. Sections of the beam, originally plane remain plane. This implies that there is complete interaction between the steel and concrete components.

b. The concrete and steel obey Hooke's law, i.e. stresses are proportional to strains.

Values of the modulus of elasticity for steel and concrete were taken from the average of test values given in Appendices II and III.

In the analysis it was necessary to consider the effect of cracking of the Stage 1 concrete. Cracking was observed at a jack load of 25 kips (76% of the design live load). To take this effect into account when predicting strains, only a range of strain for a particular gage can be defined, i.e. the strain gage indication should lie between the value computed for an uncracked section and that value for a section which cracked at 25 kips. A more accurate prediction is not possible, since the variation of strain between cracks is not known.

Therefore the lower bound strain value is computed for an uncracked section. For the upper bound value, redistribution of strains at cracking is considered. It was assumed that each fiber in the Stage 1 concrete was stressed to 500 psi when cracking occurred. From the assumed 500 psi stress value, a concentrated force at the centroid of the cracked portion was applied to the cracked cross section. This gave an increment of strain which was added to the total strain developed up to 25 kips per jack to give the strains after cracking. Strains for load increments beyond cracking are computed considering cracked section properties. Total strain beyond cracking is obtained by summing the three strain increments mentioned above.

Theoretical deflection values were computed by an elastic analysis by making the same assumptions as were made in the strain analysis.

Uncracked Section Properties:

$I = 6731 \text{ in.}^4$

$S_1 = 601 \text{ in.}^3$ (Section modulus at outer fiber of steel flange which will be the tension flange in the final position)

$S_{c2} = 623 \text{ in.}^3$ (Section modulus at outer fiber of Stage 2 concrete flange)

Cracked Section Properties:

$I = 3853 \text{ in.}^4$

$S_1 = 205 \text{ in.}^3$

$S_{c2} = 476 \text{ in.}^3$

Strain Calculation for Station 1 at 1.06 times design load:

$$M = 5840 \text{ kip-in.}$$

A. <u>Uncracked Section</u>

$$\epsilon_1 = \frac{M}{S_1 E} = \frac{5840}{601 \times 28.5 \times 10^3}$$

$$\epsilon_1 = \underline{341 \times 10^{-6} \text{ in./in.}}$$

$$\epsilon_{c2} = -\frac{M}{S_{c2}\ E} = -\frac{5840}{623 \times 28.5 \times 10^3}$$

$$\epsilon_{c2} = -\underline{329 \times 10^{-6} \text{ in./in.}}$$

Using these values calculated for ϵ_1 and ϵ_{c2}, the lower bound theoretical line may be drawn as shown in Figure 40.

B. <u>Cracked Section</u>

Strain up to 25 kips jack load (cracking occurs at this load).

$$M = \frac{25}{32.5} \times 5840 = 4490 \text{ kip-in.}$$

$$\epsilon_1 = \frac{M}{S_1\ E} = \frac{4490}{601 \times 28.5 \times 10^3} = 262 \times 10^{-6} \text{ in./in.}$$

$$\epsilon_{c2} = \frac{M}{S_{c2}\ E} = \frac{-4490}{623 \times 28.5 \times 10^3} = -253 \times 10^{-6} \text{ in./in.}$$

Additional Strain at Cracking

$$P = 500 \times 26 \times 6.5 = 84.5^k \quad \text{(Force to be applied to center of Stage 1 concrete)}$$

$$\Delta M = Pe = 84.5\,(15.9 - 3.25)$$

$$\Delta M = 1069''k \quad \text{(Moment acting on cracked section)}$$

$$\Delta\epsilon_1 = \frac{\Delta M}{S_1\,E} = \frac{1069}{205 \times 28.5 \times 10^3}$$

$$\Delta\epsilon_1 = 132 \times 10^{-6} \text{ in./in.}$$

$$\Delta\epsilon_{c2} = \frac{-\Delta M}{S_{c2}\,E} = \frac{-1069}{476 \times 28.5 \times 10^3}$$

$$\Delta\epsilon_{c2} = -79 \times 10^{-6} \text{ in./in.}$$

Strain from time of cracking to 32.5 kips at each jack

$$M = \frac{32.5 - 25}{32.5} \times 5840 = 1350k''$$

$$\Delta\epsilon_1 = \frac{1350}{205 \times 28.5 \times 10^3} = 166 \times 10^{-6} \text{ in./in.}$$

$$\Delta\epsilon_{c2} = \frac{-1350}{476 \times 28.5 \times 10^3} = -100 \times 10^{-6} \text{ in./in.}$$

Total Strains at 1.06 x Design Load

$$\epsilon_1 = (262 + 132 + 166) \times 10^{-6}$$

$$\epsilon_1 = \underline{560 \times 10^{-6} \text{ in./in.}}$$

$$\epsilon_{c2} = (-253 - 79 - 100) \times 10^{-6}$$

$$\epsilon_{c2} = \underline{-432 \times 10^{-6} \text{ in./in.}}$$

Using these values calculated for ϵ_1 and ϵ_{c2}, the upper bound theoretical line may be drawn as shown in Figure 40.

These strains also apply to Station 2 and 2' since the moment is the same. By performing similar calculations, the theoretical lines for the other loads may be obtained.

11.2 FIRST YIELD IN BOTTOM FLANGE

Since the preflexing cycles insured elastic behavior only up to 40,000 psi at Station 2', it would be expected that any test load causing a stress beyond this point would cause yielding. Station 2' will be investigated since the first cracks and thus the subsequent higher stresses formed at this station.

Stress at end of preflexing, 40,000 psi (Page 21)

Stress at start of test, 22,700 psi (Page 24)

Difference = 40,000 - 22,700 = 17,300 psi

or in terms of strain

$$= \frac{17300}{28.5 \times 10^6} = 607 \times 10^{-6} \text{ in./in.}$$

Inspection of the strain readings showed that yielding had commenced at the strain gage locations when the value of 607×10^{-6} in./in. was reached. This occurred with a load of 43.5 kips on each jack or 1.5 times design load.

However, since the strain gages were not necessarily located at the cracks, yielding probably commenced at the cracks at a lower load than the 43.5 kip load.

To find this load, the procedure used for the preceding calculations is used. This computation shows that a load of only 34.0 kips could have started yielding.

11.3 CRACKING OF STAGE 1 CONCRETE

It was found that at the start of the test a stress of 770 psi compression remained in the bottom fiber of the concrete at Station 2'. If a tensile stress of one-tenth f'_c is taken as the cracking stress, an imposed stress of approximately 1300 psi would have been necessary to cause cracking.

The computed cracking load would then be:

$$P = \frac{I \times \sigma_c \times n_1}{Lc}$$

$$P = \frac{6731 \times 1.3 \times 7.31}{180 \times 13.2}$$

$$P = 26.8 \text{ kips}$$

This agrees favorably with the observed cracking load of 25 kips.

In previous tests performed in Europe, it was found that cracking would initiate at the top face of the Stage 1 concrete. This was due to the shift in the neutral axis after the addition of the Stage 2 concrete, causing the cracking stress to be exceeded first at the top face.

This behavior did not occur in the <u>initial</u> cracking of Px-1 since the probable overstressing of the bottom fiber at Station 2' made that location more critical. However, further cracking at other locations did show evidence of cracks initiating at the top face.

12. DISCUSSION OF TEST RESULTS - BEAM Px-1

12.1 DEFLECTIONS

Figure 21 shows that the amount of residual deflection present after the first application of design load was about 0.25 inches. Comparison with the theoretical load deflection lines shows good agreement until cracking occurs. An increase in deflection of 0.10 inches over a period of 17 hours is noted at design load.

Figure 22 shows that comparison of the theoretical load deflection relationship is still fairly good up to 1.25 design load. With the load increased to 1.5 times design load, a significantly greater deflection than the theoretical is noted as indicated in Figure 23. This is due primarily to the extensive cracking which took place in the Stage 1 concrete. An increase of 0.3 inches over a period of 22 hours at 1.5 times design load is evident. This was due partly to creep of the Stage 2 concrete and partly to yielding of the steel which occurred at this load.

Beyond 1.5 times design load, much larger deflections take place for increased loadings as shown in Figure 24. Again this is due primarily to the yielding of the steel. Unlike previous load levels, further increases in deflection took place when the beam was reloaded to 2 times design load. In addition, Figure 24 shows a slight increase in deflection which occurred due to the beam having been held at design load for sixty-five hours.

12.2 STRAINS

Figure 38 shows that a linear distribution of strains was obtained along the cross section for the first loading sequence. Since elastic behavior for this region had been insured by the preflexing operation, these strains are elastic and may be converted directly to stresses. The stress at 31.0 kips is shown to be very close to the design stress of 37.0 ksi.

The strains at each cross section at ultimate load are shown in Figure 39. Much larger strains took place at the pure moment region than at the ends. As in Re-1, the neutral axis at ultimate load is located about five inches from the top of the slab in the center section.

With loads beyond design, Figures 40 through 42 show that the strains became closer to the upper bound value, that is the strain based on a section cracked at 25 kips. Since at 2 times design load, the strains are well beyond yield, no theoretical values are shown in Figure 43.

Fairly good agreement with the theoretical values for strain at Station 3 is shown on Figures 44 through 47. The theoretical values are based on the uncracked section which was truly the case at this station.

12.3 END SLIPS

As in Re-1, the end slips recorded were very small when compared to the magnitude obtained in other tests of composite beams. Again, they are considered to have little significance.

12.4 TEMPERATURE

The variation of temperature was not considered in any of the test readings.

12.5 CRACKING

The occurrence of cracking at 25 kips has already been discussed. The most significant thing to be noted concerning the width of the cracks is that the cracks are consistently wider on the south side of the beam. This is possibly due to overstressing the concrete on the one side during the turning of the beam.

13. SUMMARY AND CONCLUSION

Two 60-foot span composite steel and concrete beams, one preflexed (Px-1) and one not preflexed (Re-1) were fabricated and statically tested to evaluate the effect of preflexing on structural behavior. Two basic factors greatly influenced the test results; these were (1) the high residual stresses in the A441 steel beams as delivered to the laboratory, and (2) the methods used to turn the inverted beams right-side-up after pouring the Stage 1 concrete. The higher than expected residual stresses caused excessive loss of camber in preflexed beam Px-1, and premature inelastic behavior of both beams. The turning operation apparently overstressed the Stage 1 concrete for both Re-1 and Px-1, thus causing premature cracking and inelastic behavior of both beams. Nevertheless, valid and important results were obtained from the tests, and these are discussed below. Furthermore, proper control of materials and fabrication procedures could easily eliminate these two damaging factors now that their importance has been clearly emphasized. The test results may be summarized as follows:

13.1 STIFFENERS

Figure 37 shows that beam Px-1 is over 70 percent stiffer than Re-1 in the design live load range. The live load deflection of the preflexed beam Px-1 agreed quite closely with theoretical values based on the concrete modulus of elasticity as found from the cylinder tests.

Thus no verification could be found for the factor of 1.5 applied to the modular ratio for deflection in the beam design notes (Appendix VI).

Using the beam stiffness as found by the test in the live load deflection formula

$$\delta_{\mathrm{C_L}} = \frac{324}{E\ I} P_T \left(L^3 - 555\ L - 4780\right)^{*}$$

a value of 1.19 inches is obtained. This exceeds the AASHO allowable deflection of $\frac{L}{800}$ = 0.90 inches by 32%. Therefore, if the preflexed beam Px-1 were to be used as a bridge member, the allowable load would have to be reduced considerably to meet the ASSHO Specifications.

If application to buildings is considered, a deflection of $\frac{L}{360}$ or 2 inches is usually allowed. Considering a resisting moment of 5580k", Px-1 would deflect 1.60 inches at full live load. Hence Px-1 would be satisfactory for buildings, whereas the non-preflexed beam Re-1 would not as is evident from Figure 21.

13.2 ELASTIC RANGE

The elastic range of the preflexed beam is considerably greater than the non-preflexed beam. Figure 37 shows a residual deflection in Px-1 of about 0.2 inches after the beam had been loaded to 1.5 times design load and then released. This slight residual deflection was due to the cracking of the Stage 1 concrete, not to the inelastic behavior of the steel.

After being loaded to only design load, the residual deflection of Re-1 was already 0.8 inches. Since all the cracks had been formed in

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* page 39, Reference 6

the Stage 1 concrete prior to testing, this deflection must be attributed to yielding of the steel.

Therefore, the preflexing operation has the greatly beneficial effect of modifying the residual stresses in the steel so that the steel behaves elastically when in service.

13.3 STRESSES

Allowable design stresses in the concrete and steel were not exceeded at design load for Px-1. Figure 44 shows that the steel tensile stress was less than the 0.8 f_y (37 ksi) at the design load of 31.0 kips. The concrete compressive stress was considerably less than 0.4 f'_c (1600 psi).

13.4 CRACKING

Cracking occurred in the Stage 1 concrete of Px-1 at a load considerably less than the design load. This was due primarily to inelastic strains in the concrete at the release of preflex loads and in the turning operation. Stresses in excess of 3600 psi were obtained which is higher than the allowable of 0.6 x f'_c = 3000 psi given in the design notes. Had the Stage 1 concrete not been cast with the beam in the upside-down position, the stresses could have been kept within the allowable limits. Due to its own dead weight, cracking occurred in the Stage 1 concrete of Re-1 during the turning operation.

13.5 CREEP AND SHRINKAGE

Loss of preflexing stress due to creep and shrinkage was approximately 17% in beam Px-1.

13.6 ULTIMATE LOADS

While yielding of the tension flange of the steel beam occurred in Px-1 at 1.5 times design load, the ultimate capacity of the beam was nearly 2.5 times design load, giving a rather safe margin against failure. Though its yield load was much lower, the ultimate capacity of Re-1 was about equal to that of Px-1. This is as expected, since Preflex is designed to improve stiffness and increase working range, not increase ultimate strength.

13.7 CAMBER

A noticeable sag was present in Px-1 without any load applied. This was due to the loss of the original camber when the bare steel beam was preflexed. This loss of camber may be attributed to very high residual stresses caused by straightening the beam at the mill.

ACKNOWLEDGEMENTS

The investigations described in this report were sponsored by Alpha and Company of New York City. Schupack and Zollman, Consulting Engineers were responsible for the design of the 60-foot beams. All fabrication and testing was done at the Fritz Engineering Laboratory, Department of Civil Engineering, Lehigh University. L. S. Beedle is Director of the laboratory and W. J. Eney, who gave valuable assistance in guiding the program, is Head of the Department.

Mr. A. Dobruszkes of Preflex Brussels was retained by the sponsor to supervise the fabrication of the 60-foot preflexed beam, and Mr. Marcello Garavaglia represented Schupack and Zollman at the laboratory during the fabrication and testing.

A P P E N D I X I

C A L C U L A T I O N O F S T E E L S T R E S S E S

I N Re-1 A N D Px-1

I.1 STEEL STRESSES IN Re-1

Section Properties for Bare Steel Beam:

$I = 2034\ in^4$ (moment of inertia)

$S = 220\ in^3$ (section modulus)

Section Properties for Bare Steel Beam Plus Stage 1 Concrete:

$I = 3020\ in^4$

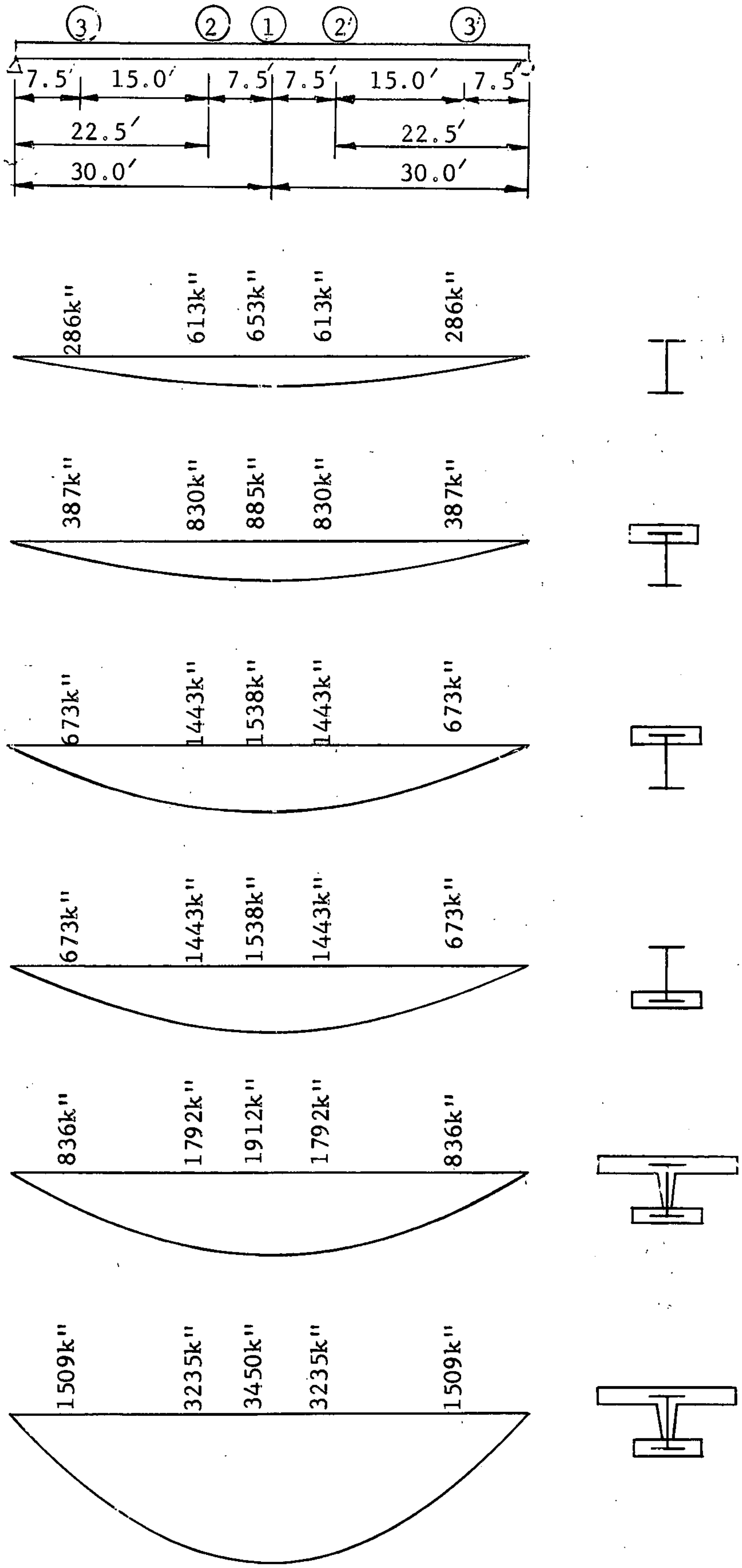
$S_1 = 493$ (Section modulus at outer fiber of steel flange which will be the tension flange in the final position)

$S_2 = 246$ (Section modulus at outer fiber of steel flange which will be the compression flange in the final position)

Dead Weights:

Stage 1 concrete	164 lb./ft.
Stage 2 concrete	354 lb./ft.
Steel beam + Stage 1 reinforcement	121 lb./ft.
Stage 2 formwork	36 lb./ft.

According to these weights, the applied moments at various stages of preparation have been computed.



Locations of Stations

Moments (kip-inches) due to:

Weight of steel beam and reinforcement

Weight of Stage 1 concrete

Total before rolling

Total after rolling

Weight of Stage 2 concrete

Total at start of test

Calculation of Stresses at Station 1:

From those moments, the stresses may be computed.

Stresses due to dead weight of steel beam and Stage 1 reinforcement:

$M = 653$ kip-in.

$$\sigma_2 = -\sigma_1 = \frac{653}{220} = 2.97 \text{ ksi}$$

Stresses due to dead weight of Stage 1 concrete (assuming half of weight is transferred directly to the floor through the formwork):

$M = 443$ kip-in.

$$\Delta\sigma_2 = -\Delta\sigma_1 = \frac{443}{220} = 2.01 \text{ ksi}$$

$$\sigma_2 = -\sigma_1 = 4.98 \text{ ksi} \quad \text{Total}$$

Additional stresses due to removal of forms (total weight of Stage 1 concrete is carried by the beam):

$M = 443$ kip.-in.

$$\Delta\sigma_1 = -\frac{443}{493} = -0.90 \text{ ksi}$$

$$\Delta\sigma_2 = \frac{443}{246} = 1.80 \text{ ksi} \quad \text{Total}$$

$$\sigma_1 = -4.98 - 0.90 = -5.88 \text{ ksi}$$

$$\sigma_2 = 4.98 + 1.80 = 6.78 \text{ ksi}$$

Stresses due to rolling of the beam (assuming that concrete cracked half-way through rolling operation):

For uncracked interval:

$$M = 1538 \text{ kip-in.}$$

$$\Delta\sigma_1 = \frac{1538}{493} = 3.12 \text{ ksi}$$

$$\Delta\sigma_2 = -\frac{1538}{246} = -6.25 \text{ ksi}$$

For cracked interval:

$$\Delta\sigma_1 = 3.12 \times \frac{493}{220} = 7.00 \text{ ksi}$$

$$\Delta\sigma_2 = 6.25 \times \frac{246}{220} = 7.00 \text{ ksi}$$

$$\sigma_1 = -5.88 + 3.12 + 7.00 = 4.24 \text{ ksi} \quad \text{Total}$$

$$\sigma_2 = 6.78 - 6.25 - 7.00 = -6.47 \text{ ksi} \quad \text{Total}$$

Stresses due to addition of Stage 2 reinforcement and formwork:

$$M = \frac{.051}{8} \times 60^2 \times 12 = 276 \text{ kip-in.}$$

$$\Delta\sigma_2 = -\Delta\sigma_1 = \frac{276}{220} = -1.21 \text{ ksi}$$

$$\sigma_1 = 4.24 + 1.21 = 5.45 \text{ ksi} \quad \text{Total}$$

$$\sigma_2 = -6.47 - 1.21 = -7.68 \text{ ksi} \quad \text{Total}$$

Stresses due to addition of Stage 2 concrete and removal of formwork:

$$M = \frac{0.303}{8} \times 60^2 \times 12 = 1640 \text{ kip-in.}$$

$$\Delta\sigma_2 = -\Delta\sigma_1 = -\frac{1640}{220} = -7.46 \text{ ksi}$$

$$\sigma_1 = 5.45 + 7.46 = 12.91 \text{ ksi} \quad \text{Total}$$

$$\sigma_2 = -7.68 - 7.46 = -15.14 \text{ ksi} \quad \text{Total}$$

If a comparison of calculated values with observed results is to be made, consideration must be taken of the fact that the exact location of the strain gages with respect to the cracks is not known. Therefore only a range of strain can be defined, i.e. the strain gage reading should lie between the value computed for the cracked section and that value calculated for an uncracked section.

For the latter case, the stresses are the same as that for the cracked section up to the point halfway through the rolling operation where:

$$\sigma_1 = -5.88 + 3.12 = -2.76 \text{ ksi}$$

$$\sigma_2 = 6.78 - 6.25 = 0.53 \text{ ksi}$$

The evolution of stresses will now be computed for an uncracked section through the addition of the Stage 2 concrete.

$$M = \frac{0.285 \times 60^2 \times 12}{8} = 1538 \text{ kip-in.}$$

$$\Delta\sigma_1 = \frac{1538}{493} = 3.12 \text{ ksi}$$

$$\Delta\sigma_2 = \frac{1538}{246} = -6.25 \text{ ksi}$$

$$\sigma_1 = -2.76 + 3.12 = 0.36 \text{ ksi}$$

$$\sigma_2 = 0.53 - 6.25 = -5.72 \text{ ksi}$$

Stresses due to addition of Stage 2 reinforcement and formwork:

$$M = 276 \text{ kip-in.}$$

$$\Delta\sigma_1 = \frac{276}{493} = 0.56 \text{ ksi}$$

$$\Delta\sigma_2 = \frac{276}{246} = -1.12 \text{ ksi}$$

$$\sigma_1 = 0.36 + 0.56 = 0.92 \text{ ksi}$$

$$\sigma_2 = -5.72 - 1.12 = -6.84 \text{ ksi}$$

Stresses due to addition of Stage 2 concrete and removal of formwork:

$$M = 1640 \text{ kip-in.}$$

$$\Delta\sigma_1 = \frac{1640}{493} = 3.33 \text{ ksi}$$

$$\Delta\sigma_2 = \frac{1640}{246} = -6.67 \text{ ksi}$$

$$\sigma_1 = 0.92 + 3.33 = 4.25 \text{ ksi}$$

$$\sigma_2 = -6.84 - 6.67 = -13.51 \text{ ksi}$$

Calculation of Deflections at Station 1

By going through the same sequence of computations as for the stresses the midspan deflections can be calculated.

I.1 STEEL STRESSES IN Re-1

Section Properties for Bare Steel Beam:

$I = 2034 \text{ in}^4$ (moment of inertia)

$S = 220 \text{ in}^3$ (section modulus)

Section Properties for Bare Steel Beam Plus Stage 1 Concrete:

$I = 3020 \text{ in}^4$

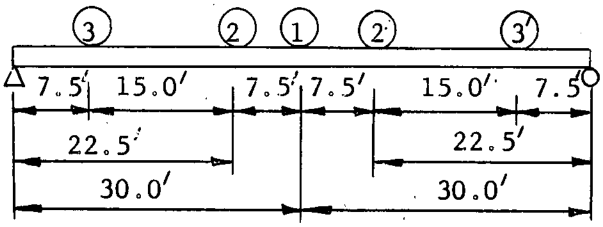
$S_1 = 493$ (Section modulus at outer fiber of steel flange which will be the tension flange in the final position)

$S_2 = 246$ (Section modulus at outer fiber of steel flange which will be the compression flange in the final position)

Dead Weights:

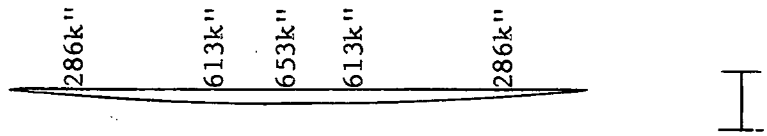
Stage 1 concrete	164 lb./ft.
Stage 2 concrete	354 lb./ft.
Steel beam + Stage 1 reinforcement	121 lb./ft.
Stage 2 formwork	36 lb./ft.

According to these weights, the applied moments at various stages of preparation have been computed.

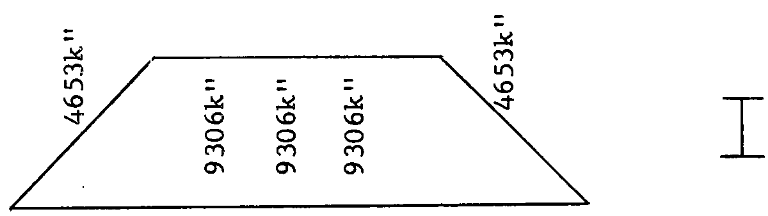


Locations of Stations

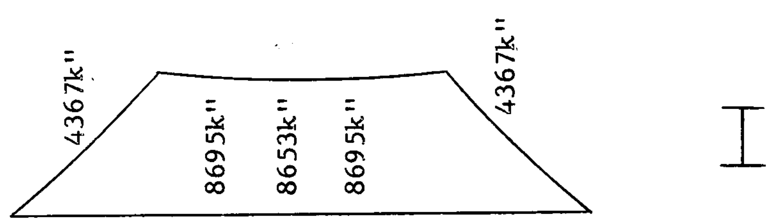
Moments (kip-inches)
due to:



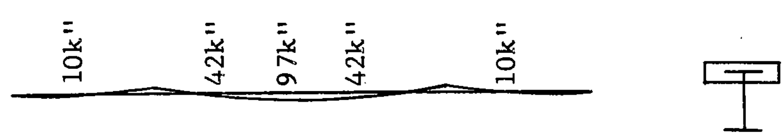
Weight of steel beam
and reinforcement



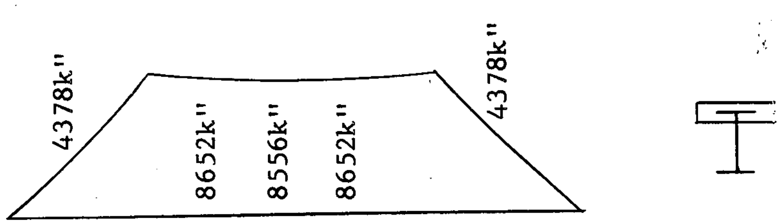
Preflexing forces applied
51.7k at each quarter point



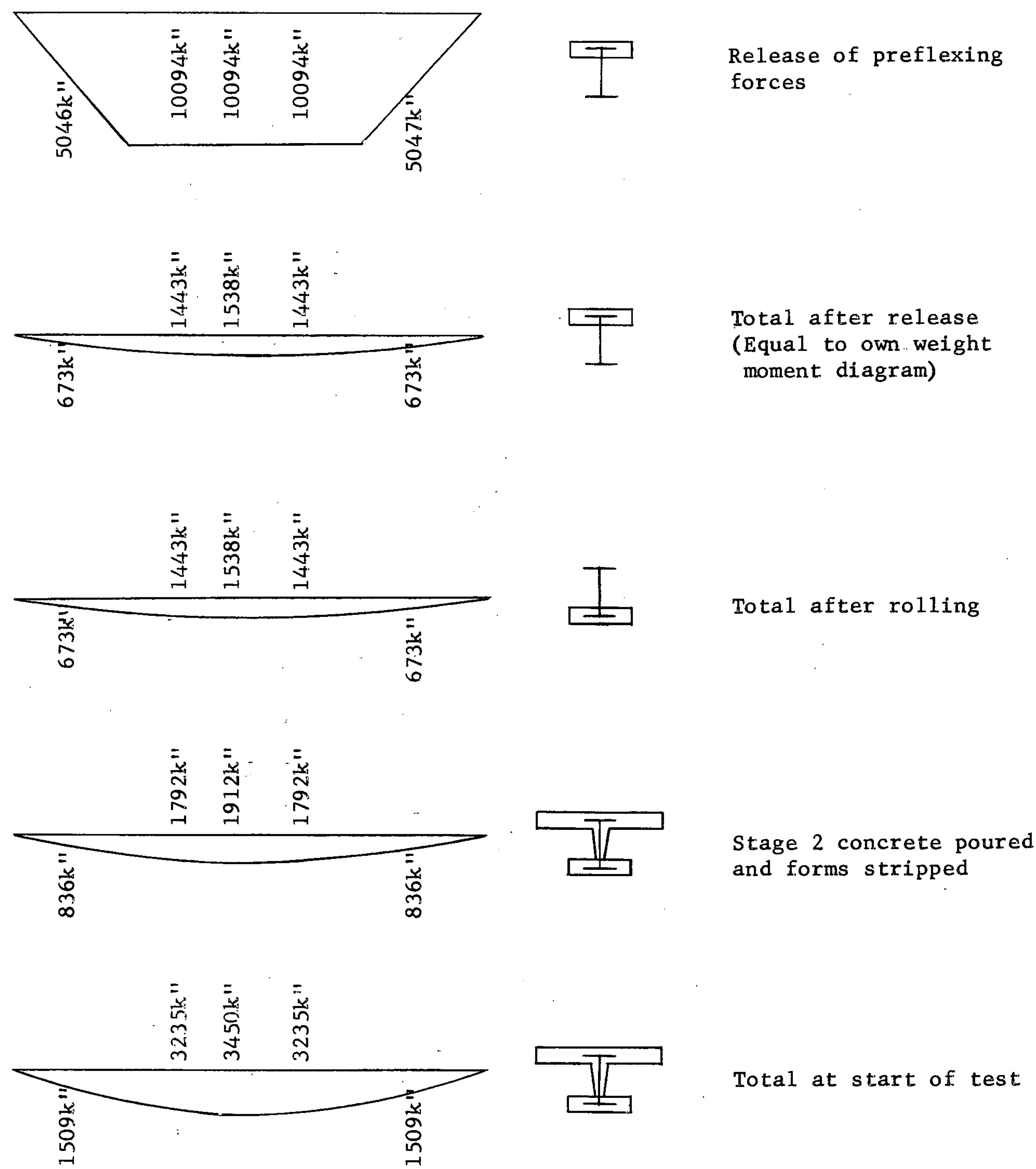
Total before pouring State
1 Concrete



Stage 1 Concrete poured
and forms stripped.
Weight acting on beam
on four supports



Total before release of
preflexing forces


5046k"
10094k"
10094k"
10094k"
5047k"
Release of preflexing forces
673k"
1443k"
1538k"
1443k"
673k"
Total after release (Equal to own weight moment diagram)
673k"
1443k"
1538k"
1443k"
673k"
Total after rolling
836k"
1792k"
1912k"
1792k"
836k"
Stage 2 concrete poured and forms stripped
1509k"
3235k"
3450k"
3235k"
1509k"
Total at start of test

APPENDIX II

STEEL INVESTIGATION:

STATIC TESTS

Satisfactory use of the Preflex technique requires a steel with a yield point of about 50,000 psi, with a long flat portion of the stress-strain curve between the onset of yielding and the beginning of strain hardening.

Standard ASTM flat tensile coupons were cut from the 5/8" thick web of a 30 WF 132 beam of A-441 steel used for preliminary investigation. The results of these tests follow:

TABLE 13 MATERIAL PROPERTIES OF 30 WF 132

Specimen No.	Yield Strength (0.2% Offset) (psi)	Tensile Strength (psi)	Elongation (8" G. L.) (%)
TS-1	53,200	73,400	27.5
TS-2	52,900	74,200	27.5
TS-3	51,100	74,200	27.5
Average	52,400	73,900	27.5

Similar coupons were cut from the 1" thick flanges of each of the end stubs of the 18 WF 114 A-441 beams used in the main testing program. The table on the following page gives the results of these tests. A marked decrease in the yield strength is shown when compared to the preceding table. Some decrease is to be expected since these coupons had a greater thickness.

TABLE 14 MATERIAL PROPERTIES OF 18 WF 114

Specimen No.	Yield Strength (0.2% Offset) (psi)	Tensile Strength (psi)
1	46,300	73,300
2	44,500	67,400
3	47,100	73,800
4	44,600	69,800
Average	45,600	71,100

The minimum steel coupon yield strength of 44,500 psi is slightly less than the required minimum value of 46,000 psi for this material.

To obtain the modulus of elasticity of the steel, round specimens, 0.505 inches in diameter were removed from each of the four end stubs. A Huggenberger extensometer was used to measure the strain over a one inch gage length. The results were as follows:

TABLE 15 MODULUS OF ELASTICITY OF STEEL

Specimen No.	Modulus of Elasticity (psi)
TH-1	28.2×10^6
TH-2	28.8×10^6
TH-3	28.5×10^6
TH-4	29.0×10^6

Taking the values given by the Huggenberger readings, a value for the modulus of elasticity of 28.5×10^6 psi, was used in the theoretical calculations for beams Re-1 and Px-1.

APPENDIX III

CONCRETE MIXES

AND

CYLINDER TESTS

III.1 CONCRETE MIXES

The proportions of cement, sand and gravel used in the Stage 1 (5000 psi design) and Stage 2 (4000 psi design) concrete is given in the table below. These two mix designs were chosen from test results of five trial mixes which were designed for 3500, 4000, 5000 and 5500 psi.

TABLE 16 CONCRETE MIXES

Ingredients	Stage 1 Mix (5000 psi) (lbs)	Stage 2 Mix (4000 psi) (lbs)
Cement	644	536
Sand	1162	1210
Coarse Aggregate	1983	1973
Water	317 (approx.)	320 (approx.)

The dry materials were added at the plant and mixed enroute. Water was added at the laboratory until a slump of 2-1/2 inches was obtained in each case.

Crushed traprock of 3/4 inch maximum size was used for the coarse aggregate. The fineness modulus of the sand was between 2.9 and 3.0. Lehigh Type I cement supplied in bags was used in each case.

III.2 CONCRETE CYLINDER TESTS

Standard ASTM concrete cylinders were cast from each of the four pourings in the fabrication of Re-1 and Px-1. Results of these tests follow:

TABLE 17   STAGE 1 CONCRETE CYLINDER TEST OF Re-1

Cylinder No.	Age at Time of Test (Days)	Ultimate Strength (psi)	Modulus of Elasticity (psi)
1	7	3410	-
2	7	3290	-
3	7	3440	-
4	7	3600	-
5	17	4380	-
9	32	5020	3.5×10^6
6	52	5280	3.2×10^6
12	65	5070	-
10	126	5240	3.7×10^6
8	126	5300	3.4×10^6

Average value of modulus of elasticity
for twenty-eight days age or over: 3.4×10^6 psi

TABLE 18   STAGE 1 CONCRETE CYLINDER TESTS OF Px-1

Cylinder No.	Age at Time of Test (Days)	Ultimate Strength (psi)	Modulus of Elasticity (psi)
1	7	3920	-
2	7	4370	-
3	7	3960	-
4	7	3860	-
5	17	5570	-
11	32	5420	3.7×10^6
10	65	5950	-
6	66	6360	3.7×10^6
12	66	5670	4.3×10^6
9	126	5600	4.2×10^6
7	126	6060	3.8×10^6

Average value of modulus of elasticity
for twenty-eight days or over: 3.9×10^6 psi

TABLE 19 STAGE 2 CONCRETE CYLINDER TEST OF Re-1

Cylinder No.	Age at Time of Test (Days)	Ultimate Strength (psi)	Modulus of Elasticity (psi)
26	8	3320	-
15	8	3360	-
27	8	3350	-
24	15	3840	-
22	15	4030	-
23	29	4510	-
8	29	4160	-
14	29	4500	3.7×10^6
1	32	4600	3.7×10^6
2	45	5090	-
11	106	4950	3.4×10^6
21	106	5290	3.9×10^6

Average value of modulus of elasticity for twenty-eight days age or over: 3.7×10^6 psi

TABLE 20 STAGE 2 CONCRETE CYLINDER TEST OF Px-1

Cylinder No.	Age at Time of Test (Days)	Ultimate Strength (psi)	Modulus of Elasticity (psi)
10	7	3320	-
9	7	3410	-
16	7	3070	-
18	14	3840	-
17	14	3970	-
5	14	3950	-
11	27	4420	-
11A	28	4260	3.5×10^6
8	28	4490	3.1×10^6
25	88	4280	3.6×10^6
22	88	4720	3.5×10^6

Average value of modulus of elasticity for twenty-eight days or over: 3.4×10^6 psi

A Strength vs. Age curve for all the concrete tests is given in Fig. 56.

APPENDIX IV

STATIC PUSHOUT TESTS

To determine the static load carrying capacity of the bent stud shear connectors when welded to the A-441 steel, pushout specimens were fabricated and tested.

A steel beam section modified from a 30 WF 132 to which were welded four studs on each of the flanges served as the main element of the push-out specimen. Figure 51 shows this part of the specimen. To simulate the condition in a beam, the flanges were encased in concrete with three inches of cover on the outside flange surfaces.

To test these specimens, load is applied such that the steel is pushed through the concrete and the shear connectors are loaded as they would be in a composite beam.

Two specimens were tested, one with one-half inch diameter "L" shaped studs of 2-1/4" height, the other with one-half inch diameter "L" shaped studs of 1-3/4" height. These corresponded to the type of studs used on the compression and tension flange respectively of the test beams.

The maximum load per connector for the 2-1/4" studs was 11.3 kips. For the 1-3/4" studs, the load was 12.4 kips per connector. These results were nearly identical to previous tests conducted at Lehigh University, and it was concluded that welding to the A-441 steel was just as satisfactory as ordinary structural steel for static loadings. Hence, the same allowable connector stress was used as in regular composite construction.

APPENDIX V

FATIGUE TESTS

V.1 FATIGUE COUPONS

Four series of fatigue tests were conducted using coupons machined from material removed from the web of a 30 WF 132, A-441 steel beam. The data and results of the four test series are given in Table 21.

The first series consisted of three specimens (designated as F-1, F-2 and F-3) to each of which four 1/2" diameter "L" shaped studs had been welded. The ends of the studs were tapered at approximately a 45^{o} angle so that just prior to welding, only the center point of the stud was in contact with the plate material. This was the type of stud used in both beams Re-1 and Px-1 and in all other tests unless noted otherwise.

The coupons were subjected to a pulsating load such that the tensile stress varied from 25 to 37 ksi. This type of test was performed to obtain the fatigue life of A-441 steel as welded.

Figures 52 and 53 show specimens F-1, F-2 and F-3 after failure. Failure occurred at a weld location in each case.

The second series of tests was performed in an attempt to improve on the results of the first series. The second series consisted of two specimens, one unwelded (F-4) and the other welded (F-5). The stress range was the same as that for the first series.

The welded specimen had five blunt ended studs welded to it with three different time settings. Each weld would have been considered acceptable by eye inspection. The welds differed in that they ranged from a short time setting which produced a "cold" weld to a longer time setting which produced a "hot" weld. It was felt at the time of welding that a weld with the highest time setting acceptable (i.e. one which was not so high that undercutting occurred) was the best type. The reason for this opinion was that the heat affected zone of the base material would be larger. Subsequent hardness tests on the welded material bore out this point. It was found that the hardness of the welded zone was considerably lower in the weld which had the longer time setting.

The results of Test Series 2 indicated that perhaps a blunt end stud is more satisfactory than a beveled stud, and a "hot" weld is more satisfactory than a "cold" weld. The third series of tests was conducted to better establish the results of Test Series 2. Also Test Series 3 was to simulate the testing program for a proposed beam fatigue test. The testing program was to be:

25 - 37 ksi	for	1,000,000 cycles
25 - 39.2 ksi	for	250,000 cycles
25 - 41.4 ksi	for	250,000 cycles

There were two specimens tested in the third series. Specimen F-6 had five beveled studs welded to it using the hottest permissable setting. Specimen F-7 was identical to F-6 except that blunt end studs were used. Both specimens were tested according to the test program given above. Both specimens survived the proposed test program; the tests were then continued to failure with the blunt end studs again performing better than the beveled studs.

Specimen F-8 was not fatigue tested. Since it was felt that a knowledge of the residual stresses induced by welding would be desirable, the areas in the vicinity of both studs were investigated.

Measurements of strain over a two inch gage length at the studs were taken prior to welding, after welding, and after a sectioning operation which relieved the residual stresses. It was found that a tensile stress of approximately 25 ksi was induced at the stud by the welding.

It was thought that pretensioning a coupon before welding might improve the fatigue resistance of the base material. (This is analogous to preflexing a beam prior to installing the studs.) Upon release of the pretensioning, the tensile stress introduced by the welding would tend to be relieved.

As a result Test Series 4 consisting of one specimen was conducted to investigate this theory. The specimen (F-9) was pretensioned vertically to an average stress of 35 ksi, and welded while in this state. The welding procedure employed was the best that could be obtained based on the previous experience. Blunt end studs were used. The testing program was the same as that for Series 3.

On the basis of this test, the pretensioning gave less satisfactory results than the similar non-pretensioned specimen (F-7). However, the fact that F-9 had to be welded while in the vertical position may have had some effect on its behavior, though visual inspection of the welds showed nothing to verify this.

From these nine fatigue tests, it may be concluded that the most satisfactory results are obtained with blunt end studs welded with the hottest setting permissable. With this arrangement, welded 5/8" thick plate material withstood 1,149,700 cycles at 1.5 times design tensile stress for a typical preflexed beam.

The results of the nine fatigue tests are given in Table 21.

V.2 FATIGUE PUSHOUT SPECIMEN

A pushout specimen with 2-1/4" high shear connectors was dynamically tested to simulate the action on the compression flange of a beam. The initial loads were such that the shearing force on each connector varied from zero to three kips, which were the values selected from previous pushout tests to give a fatigue life of 2,000,000 cycles. This condition was maintained for 2,000,000 cycles. The maximum load was increased in stages until finally 1,000,000 cycles at a maximum load of nine kips per connector were recorded with still no failure. The testing was then discontinued since it seemed that the bond between the concrete had still not broken and that actually the shear connectors were not carrying the computed load.

V.3 FATIGUE BEAM TEST

Two short preflexed beams with concrete cast on the tension flange were tested dynamically. The beams were 12 feet in length, simply supported with an alternating concentrated load at the center. Figure 54 shows the test setup. The purpose of these tests was to simulate the stress condition in the tension flange of a full scale beam. Two beams, B-1 and B-2, were tested.

The stud welding for both beams was performed similar to fatigue coupons F-1, F-2 and F-3, which means that beveled end studs were used and also the possibility of "cold" studs was likely. These conditions were used since the welding was performed prior to the investigation of the performance comparison of "hot" welds to "cold" welds.

The data and results of the tests are given in Table 22. Specimen B-1 failed after about 1,000,000 cycles at a tensile stress range of 20 to 37 ksi. Specimen B-2 withstood 2,000,000 cycles without failure at a tensile stress range of 28 to 37 ksi, the proposed working range for preflexed beams. Over 880,000 additional cycles of increased stress amplitude were then applied to B-2 without failure before testing was discontinued. It should be noted that the maximum shear connector load occurs where the beam stress is a minimum on the tension flange of a preflexed beam. Figure 55 shows the failure of B-1 after the concrete slab had been broken away. B-2 did not fail.

For a more detailed account of these investigations, given in Appendix V see "notes on Stud Welding to High Strength Steels" by Marcello Garavaglia, of the Consulting Engineering firm of Shupack and Zollman.

TABLE 21 FATIGUE COUPON TESTS

Series	Description	Specimen Number	Type of Stud	Tensile Stress (ksi) Maximum	Tensile Stress (ksi) Minimum	Total Cycles	Remarks
1	4 studs	F-1	Beveled ends	37	25	2,660,000	Failed at a weld
		F-2	Beveled ends	37	25	3,231,000	Failed at a weld
		F-3	Beveled ends	37	25	2,982,000	Failed at a weld
2	5 studs 3 time settings	F-4	None	37	25	4,457,000	No Failure
		F-5	Blunt ends	37	25	3,995,000	Failed at "colder" weld
3	5 studs "hot" setting	F-6	Beveled ends	37	25	1,007,000	
				39.2	25	287,400	
				41.4	25	281,100	Failed at a weld
		F-7	Blunt ends	37	25	1,007,000	
				39.2	25	287,000	
				41.4	25	1,149,700	Failed at a weld
4	"hot" setting plate pre-stressed welded vertically	F-9	Blunt ends	37	25	1,007,000	
				39.2	25	287,400	
				41.4	25	553,000	Failed at a weld

TABLE 22 FATIGUE BEAM TESTS

Beam Number	Tensile Stress (ksi) Maximum	Tensile Stress (ksi) Minimum	Shear Connector Loads (kips) Maximum	Shear Connector Loads (kips) Minimum	Total Cycles	Remarks
B-1	37	20	3	0.5	1,045,000	Failed at the row of studs nearest to the center of the beam
B-2	37	28	1.5	0.5	2,000,000	
	37	24	2	0.5	550,000	
	37	20	3	0.5	333,000	Test was stopped due to mechanical difficulties of the testing apparatus. No failure had occurred

APPENDIX VI

DESIGN NOTES FOR PREFLEXED BEAMS

As explained in Section 2.1 the design criteria for the preflexed beams is based on European practice applied to American materials.

VI.1 STEEL

Type of Steel	ASTM A-441
Minimum Yield Strength:	f_y = 50,000 psi for thicknesses of 3/4" and under
	f_y = 46,000 psi for thicknesses of 3/4" to 1-1/2"
Modulus of Elasticity:	29,000,000 psi
Allowable Flexural Stress at Preflexing:	$f_s = 0.8\ f_y$
Allowable Flexural Stress under Full Design Load:	$f_s = 0.8\ f_y$

VI.2 CONCRETE

Stage 1 - 28 day Strength:	f'_c = 5000 psi
Allowable Compressive Stress at Release of Preflexing Forces:	$0.6\ f'_c$ = 3000 psi
Stage 2 - 28 day Strength:	f'_c = 4000 psi
Allowable Compressive Stress:	$0.4\ f'_c$ = 1600 psi

VI.3 MODULAR RATIO

A. For stress analysis - n of Stage 1 concrete = 8.4

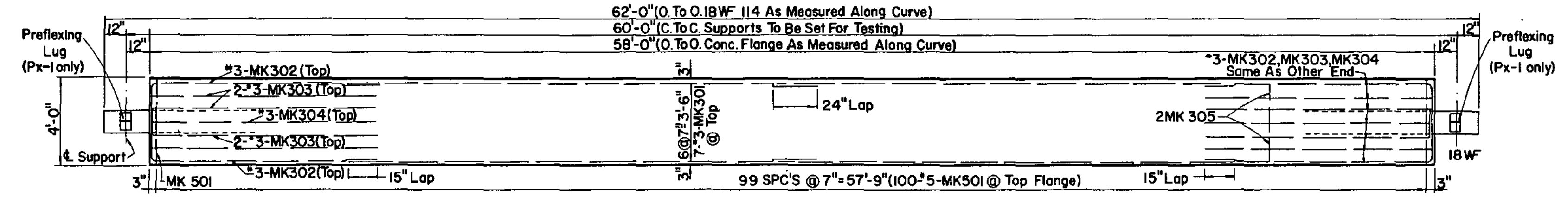
n of Stage 2 concrete = 9.1

B. For deflection analysis - n of Stage 1 concrete $= \frac{8.4}{1.5} = 5.6$

n of Stage 2 concrete $= \frac{9.1}{1.5} = 6.1$

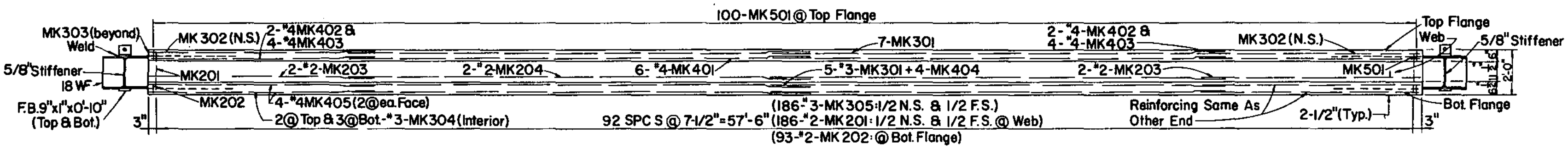
NOMENCLATURE

Px-1	Designation for the preflexed test beam 60 feet in length
Re-1	Designation for the non-preflexed test beam 60 feet in length
Stage 1 Concrete:	The slab of concrete which encases the tension flange of Re-1 and Px-1
Stage 2 Concrete:	The slab of concrete which encases the compression flange and web of Re-1 and Px-1
I	Moment of inertia
S_1	Section modulus at the outer fiber of the steel flange which will be the tension flange in the final position
S_2	Section modulus at the outer fiber of the steel flange which will be the compression flange in the final position
S_{c1}	Section modulus at outer fiber of Stage 1 concrete flange
S_{c2}	Section modulus at outer fiber of Stage 2 concrete flange
M	Applied moment at the cross section under consideration
σ	Stress
ϵ	Unit Strain
σ_1	Stress at outer fiber of steel flange which will be the tension flange in the final position
ϵ_1	Strain at outer fiber of steel flange which will be the tension flange in the final position
σ_2	Stress at outer fiber of steel flange which will be the compression flange in the final position
ϵ_2	Strain at outer fiber of steel flange which will be the compression flange in the final position
σ_{c1}	Stress at outer fiber of Stage 1 concrete flange
ϵ_{c1}	Strain at outer fiber of Stage 1 concrete flange
σ_{c2}	Stress at outer fiber of Stage 2 concrete flange
ϵ_{c2}	Strain at outer flange of Stage 2 concrete flange
$\Delta\sigma$	Increment of stress due to change of load under consideration
WF	Wide Flange Beam
psi	Pounds per square inch
ksi	Thousand pounds per square inch
kips	Thousand pounds
Station 1	Midspan of beam
Station 2	7-1/2 feet east of midspan
Station 3	22-1/2 feet east of midspan
Station 2'	7-1/2 feet west of midspan
Station 3'	22-1/2 feet west of midspan
$\delta_{℄}$	Deflection at midspan
G. L.	Gage length
f_y	Yield strength
f_s	Allowable steel stress
f_c'	28 day compressive strength of concrete
n	Modular ratio, $\frac{\text{Steel Modulus of Elasticity}}{\text{Concrete Modulus of Elasticity}}$
L	Span Length

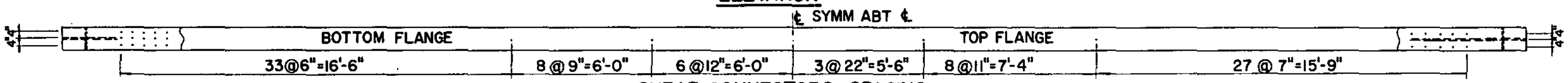


PLAN VIEW

Scale: 3/8"=1'-0"

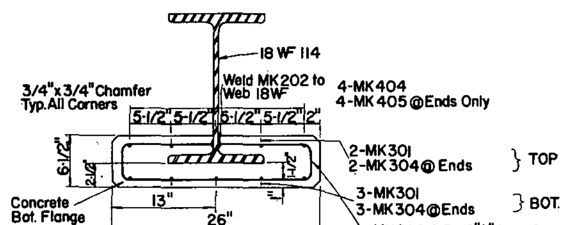


ELEVATION

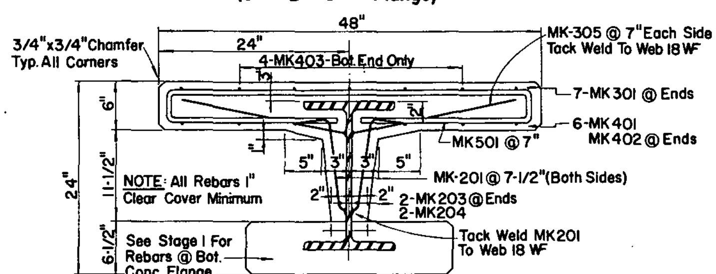


SHEAR CONNECTORS SPACING

Scale: 3/8"=1'-0"



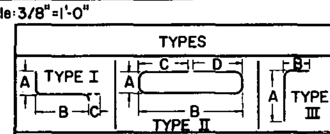
STAGE "1"
(Cast Bot. Conc. Flange)



STAGE "2"
(Complete Casting)

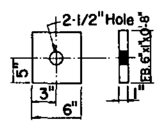
TYPICAL SECTIONS THRU BM.

Scale: 1-1/2"=1'-0"



REINFORCING SCHEDULE — ONE BEAM

MK. NO.	Size	Length	NO. Req'd	Type	A	B	C	D	E	F	G
201	#2	1'-5"	186	I	0-4	1-0	0-1				
202	#2	4'-8"	93	II	0-4½	2-0	0-11½	0-11½			
203	#2	10'-3"	4	STR							
204	#2	22'-0"	4	STR.							
301	#3	22'-0"	24	STR.							
302	#3	12'-6"	4	III	10-3	2-3					
303	#3	10'-7"	8	III	10-3	0-4					
304	#3	10'-3"	12	STR.							
305	#3	1'-9"	186	STR.							
401	#4	22'-0"	12	STR.							
402	#4	11'-9"	4	III	10-3	1-6					
403	#4	10'-3"	8	STR.							
404	#4	22'-0"	8	STR.							
405	#4	10'-3"	8	STR.							
501	#5	8'-3"	100	II	0-4	3-10	1-0½	1-0½			



PREFLEXING LUG DETAIL

MATERIAL SPECIFICATION

1. Steel Beam Tri-Ten Steel, f_{yld}=46,000 PSI Min., Free Of Grease, Scale, Paint, And Foreign Matter.
2. Mild Steel Rebars, Intermediate Hard Grade f_s=20,000 PSI, Free Of Rust, Scale, Grease, And Foreign Matter. Rebars Deformed, Except #2 Size.
3. Stage I Concrete (2.35 cu.yd.) f_c'=5000 PSI @ 28 Days, Stone Concrete Max. Aggregate Size 1 in.
4. Stage 2 Concrete (5.10 cu.yd.) f_c'=4000 PSI @ 28 Days, Stone Concrete Max. Aggregate Size 1 in.
5. Cement-Lehigh Type I.

GENERAL NOTES

1. Stage I Concrete Cured To Attain f_c'=5000 PSI Before Releasing Loads: Damp Cured With Wet Burlap Min. 28 Days.
2. Stage 2 Concrete Cured To Attain f_c'=4000 PSI Prior To Applying Test Loads: Damp Cured With Wet Burlap Min. 28 Days.
3. Beam Handled In Such A Manner As To Prevent Damage To Concrete.
4. Stage 2 Concrete Cast With Forms Bearing On Bottom Flange. Beam Not Shored.

FIG. 1 FABRICATION DRAWING
(BEAMS RE-1 & PX-1)

FIG. 2 STUD WELDING OPERATION

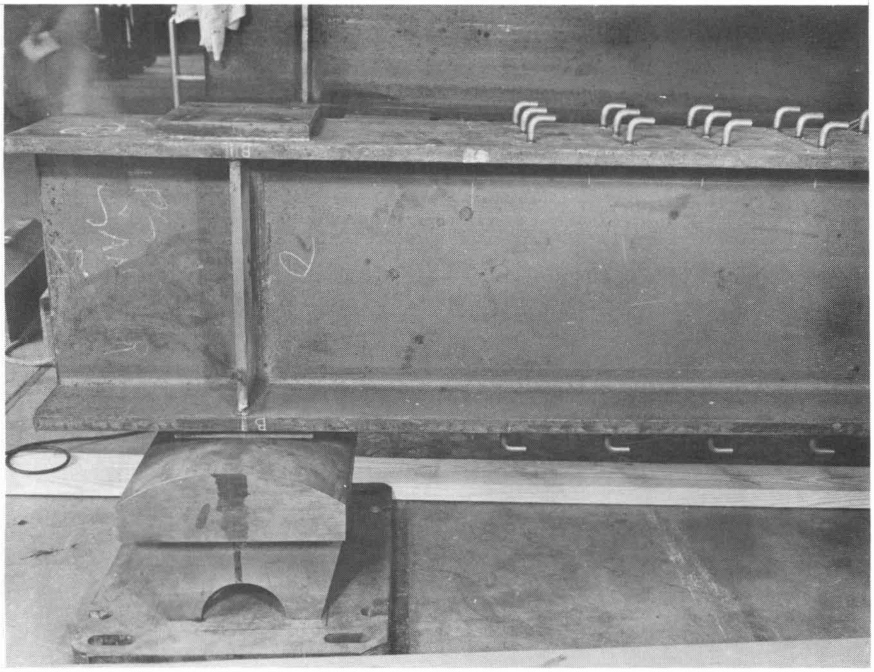


FIG. 3 Re-1, END SUPPORT

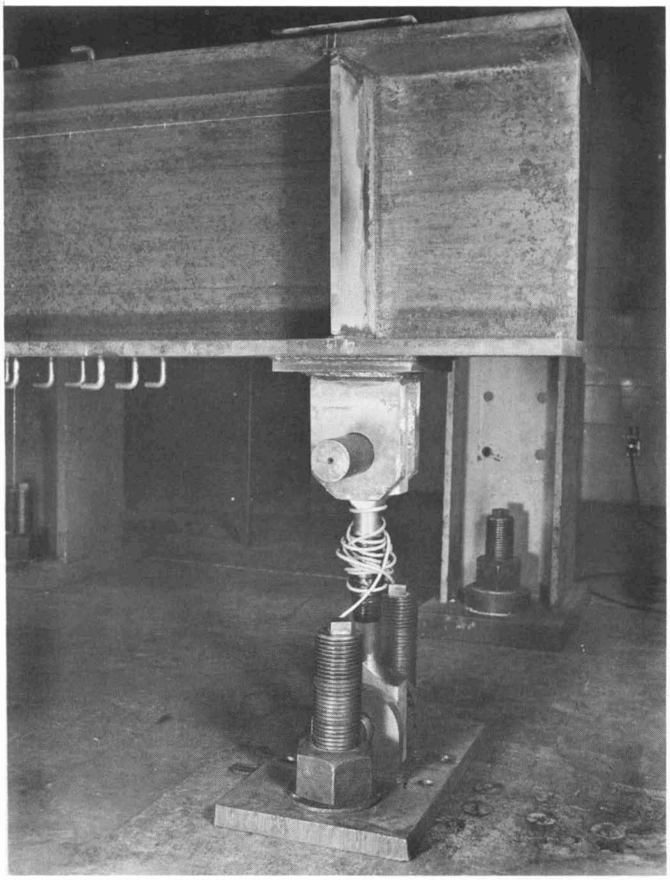


FIG. 4 Px-1, END SUPPORT

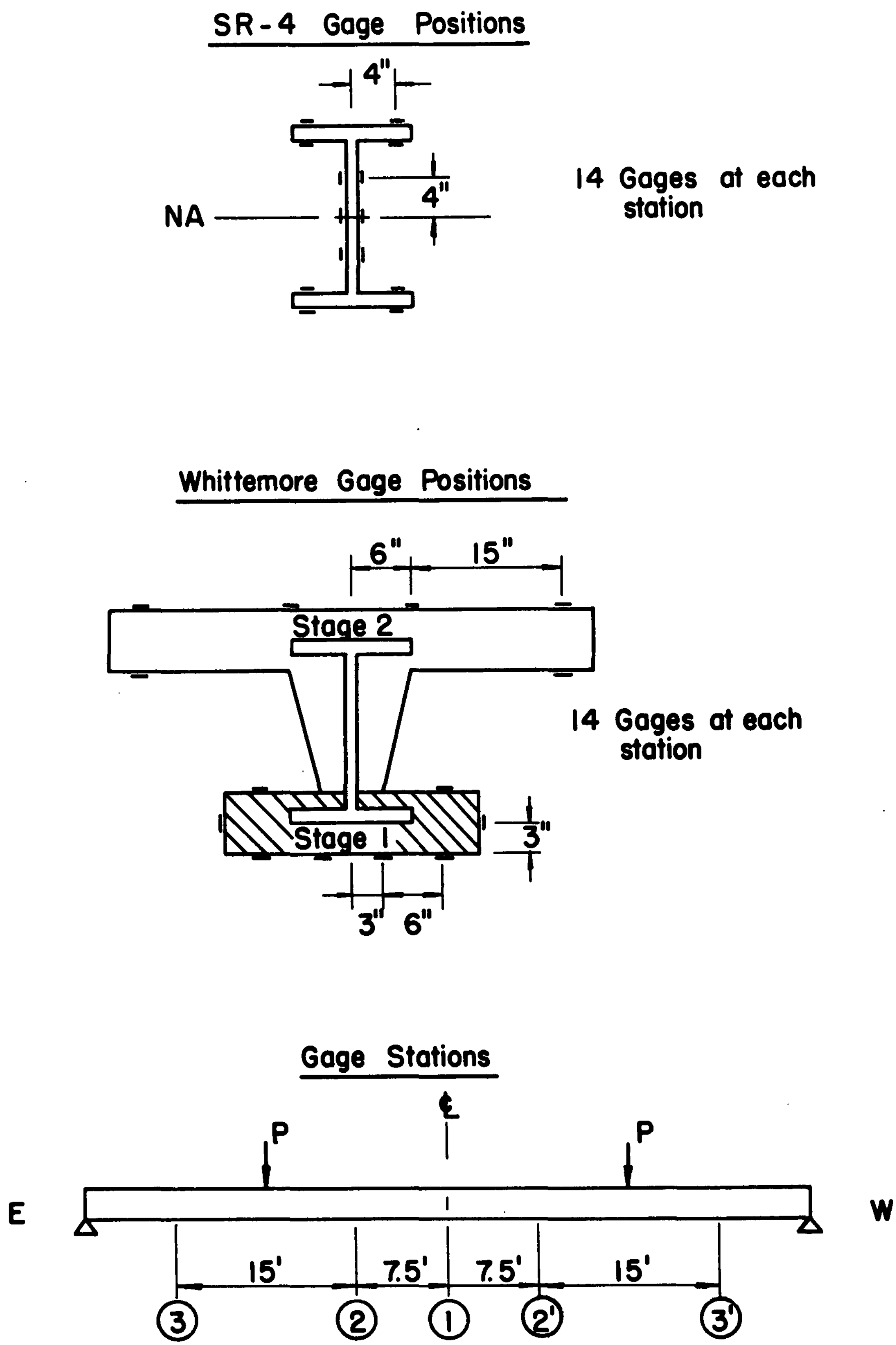


FIG. 5 GAGE AND STATION LOCATIONS

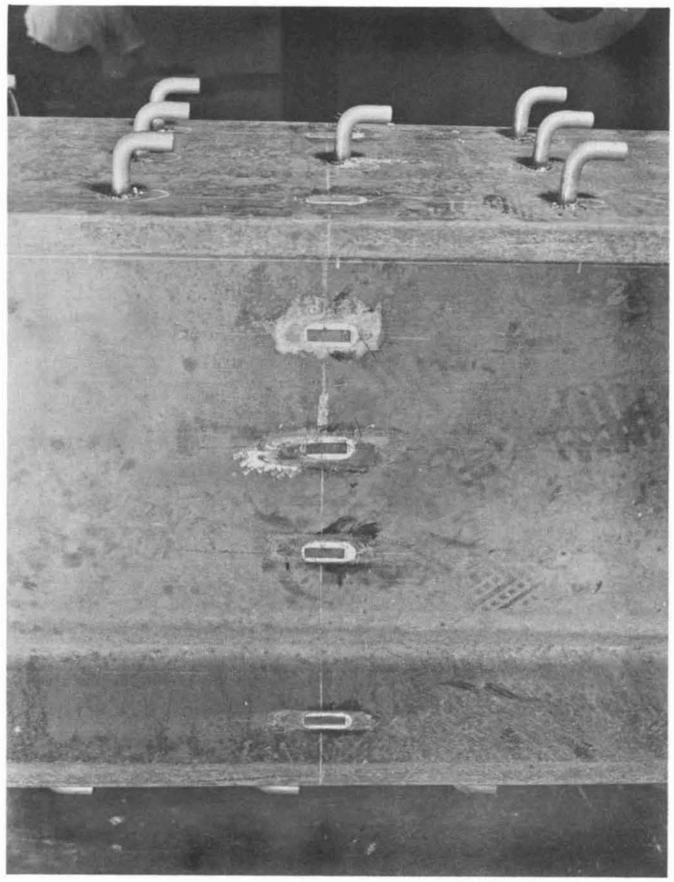


FIG. 6 STRAIN GAGES AT TYPICAL CROSS SECTION

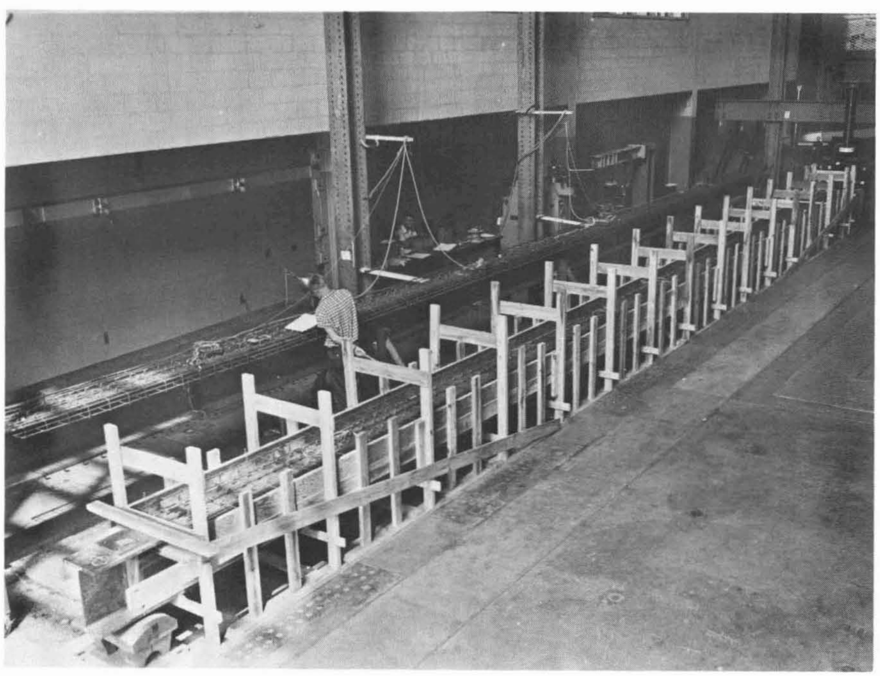


FIG. 7 FORMWORK FOR Re-1

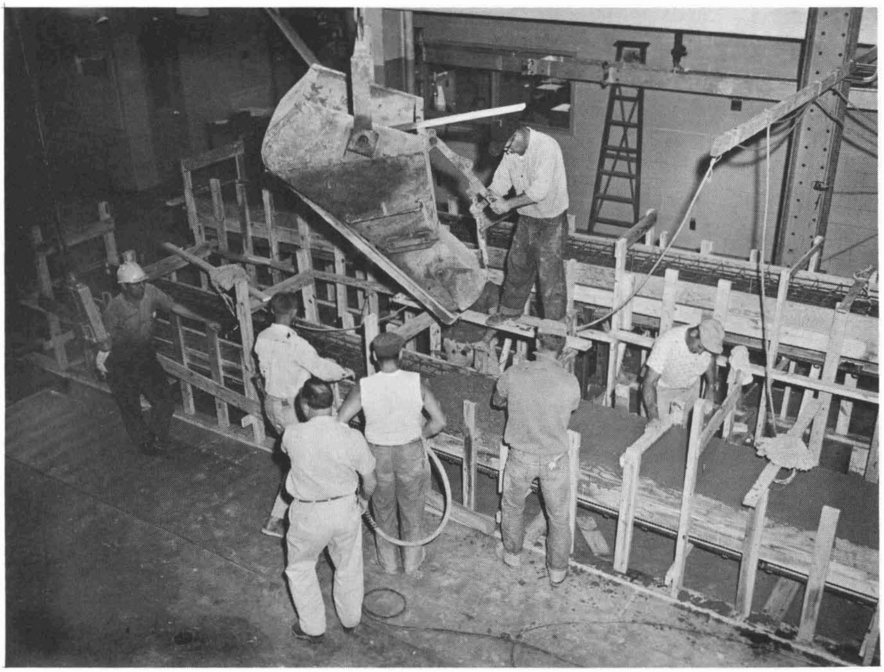


FIG. 8 POURING STAGE I CONCRETE, Re-I

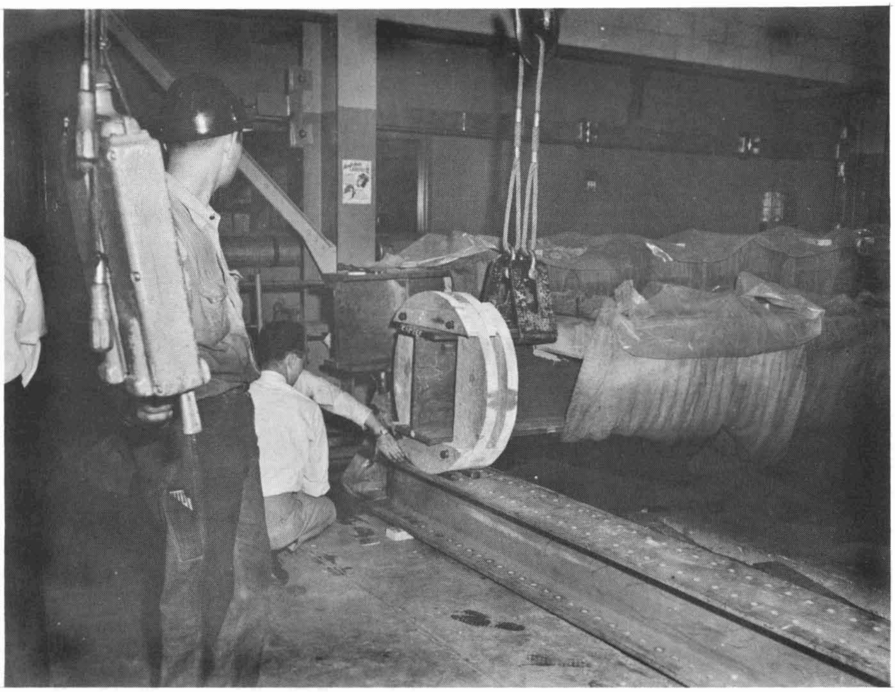


FIG. 9 Re-I ABOUT TO BE ROLLED

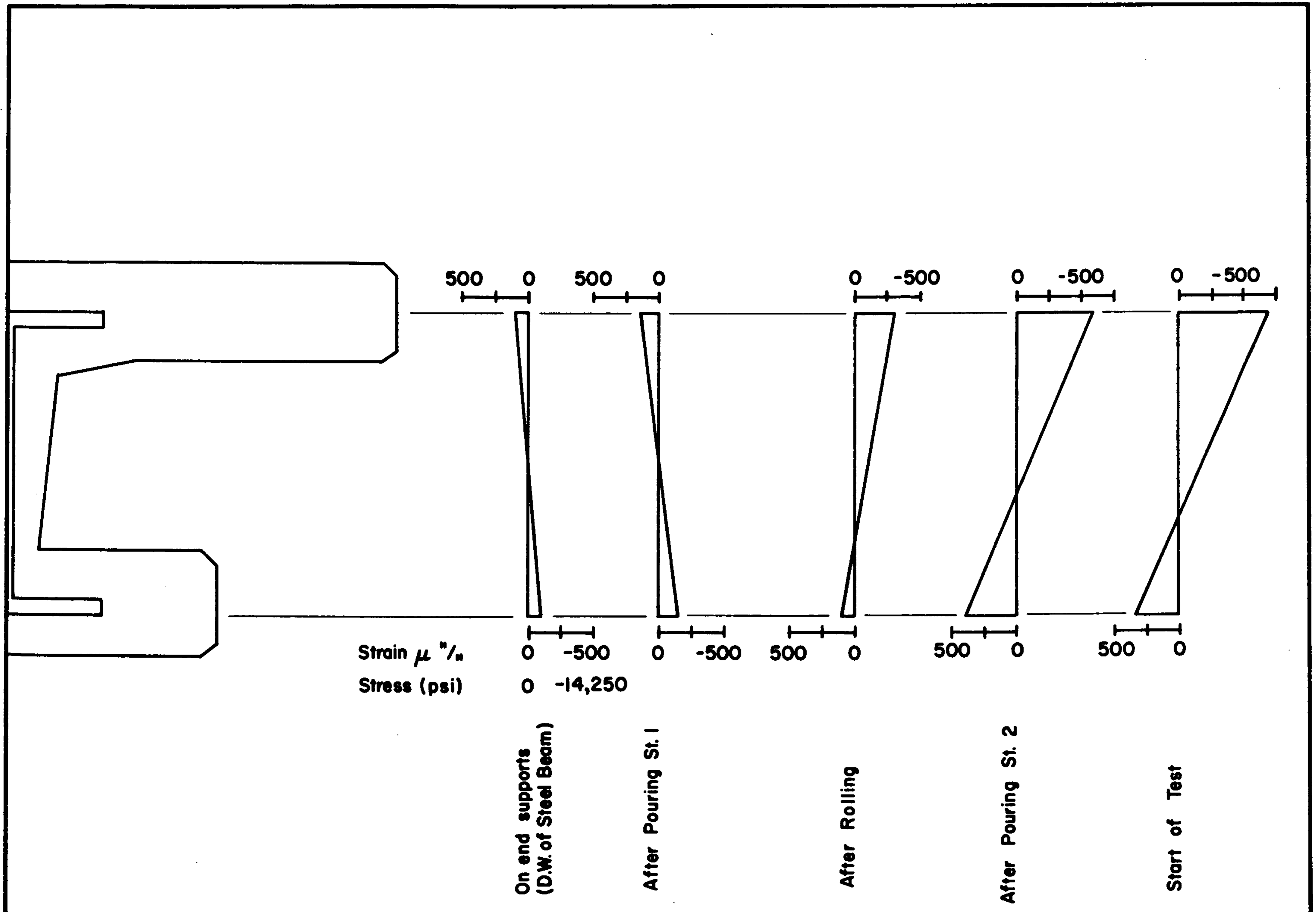


FIG. 10 Re-1, STEEL STRAINS AT STATION 1 CURING PERIOD TO START OF TEST

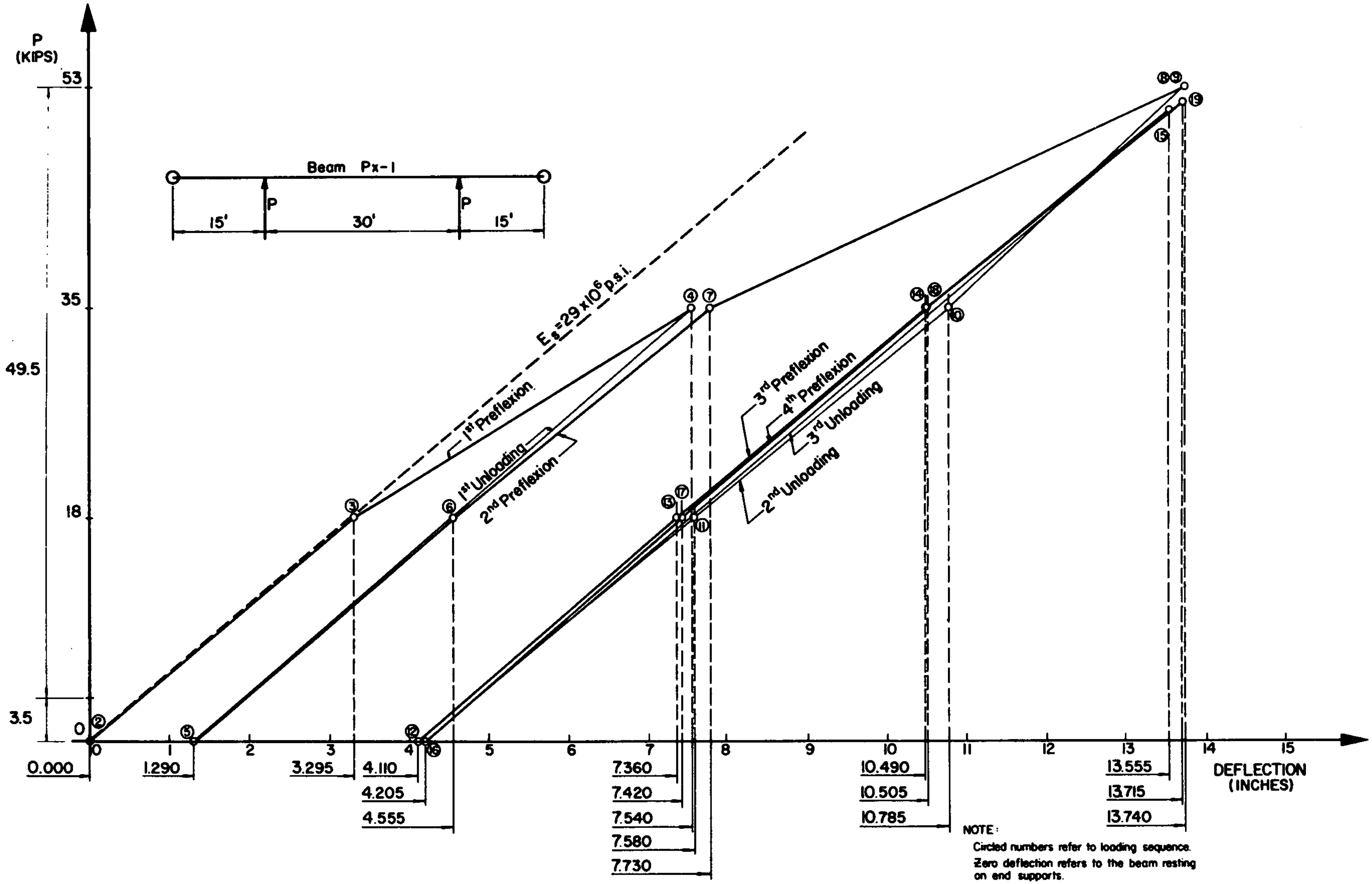


FIG.11 PREFLEXING DIAGRAM Px-1 (MIDSPAN)

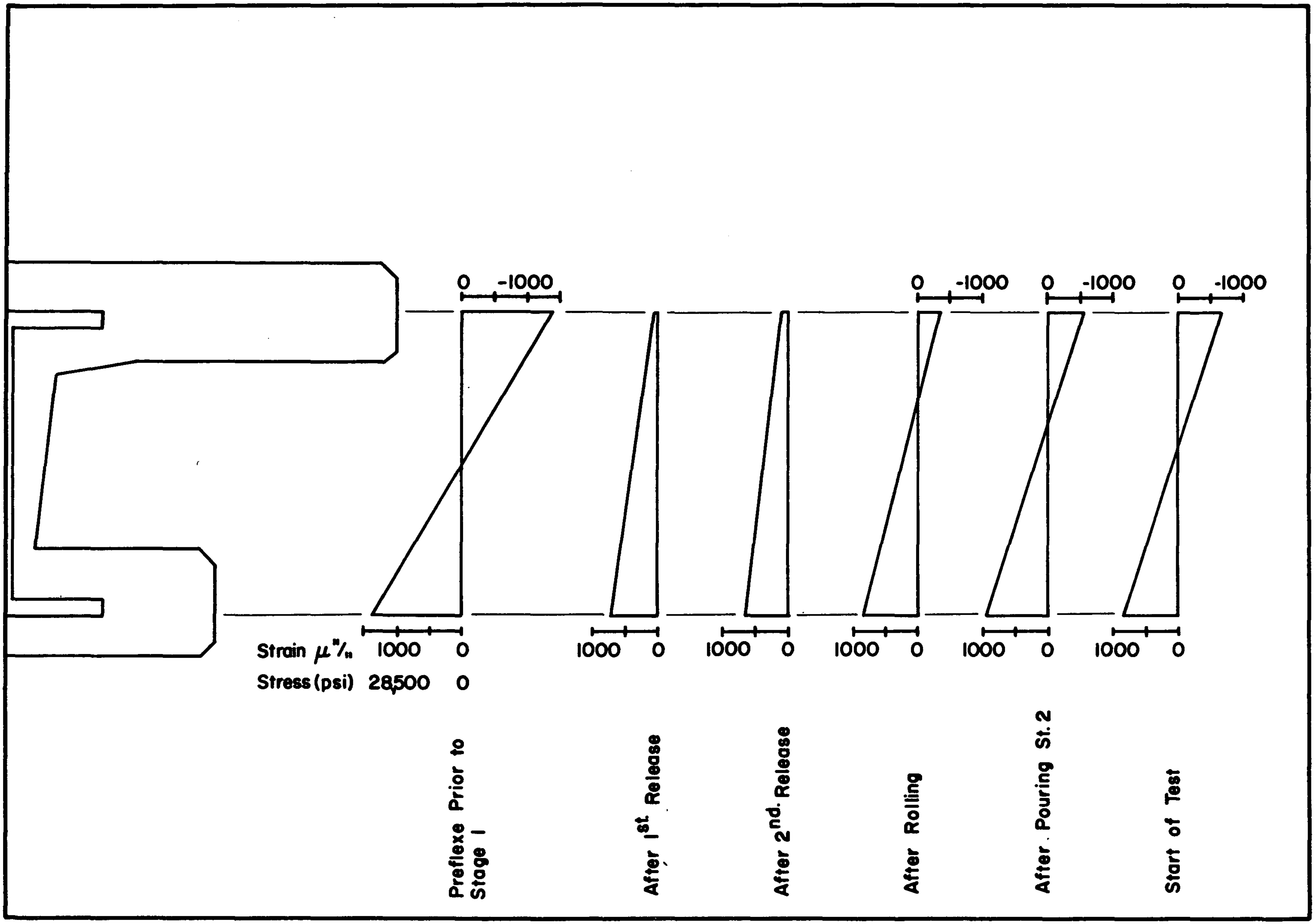


FIG. 12 Px-1, STEEL STRAINS AT STATION 1, END OF PREFLEXING TO START OF TEST

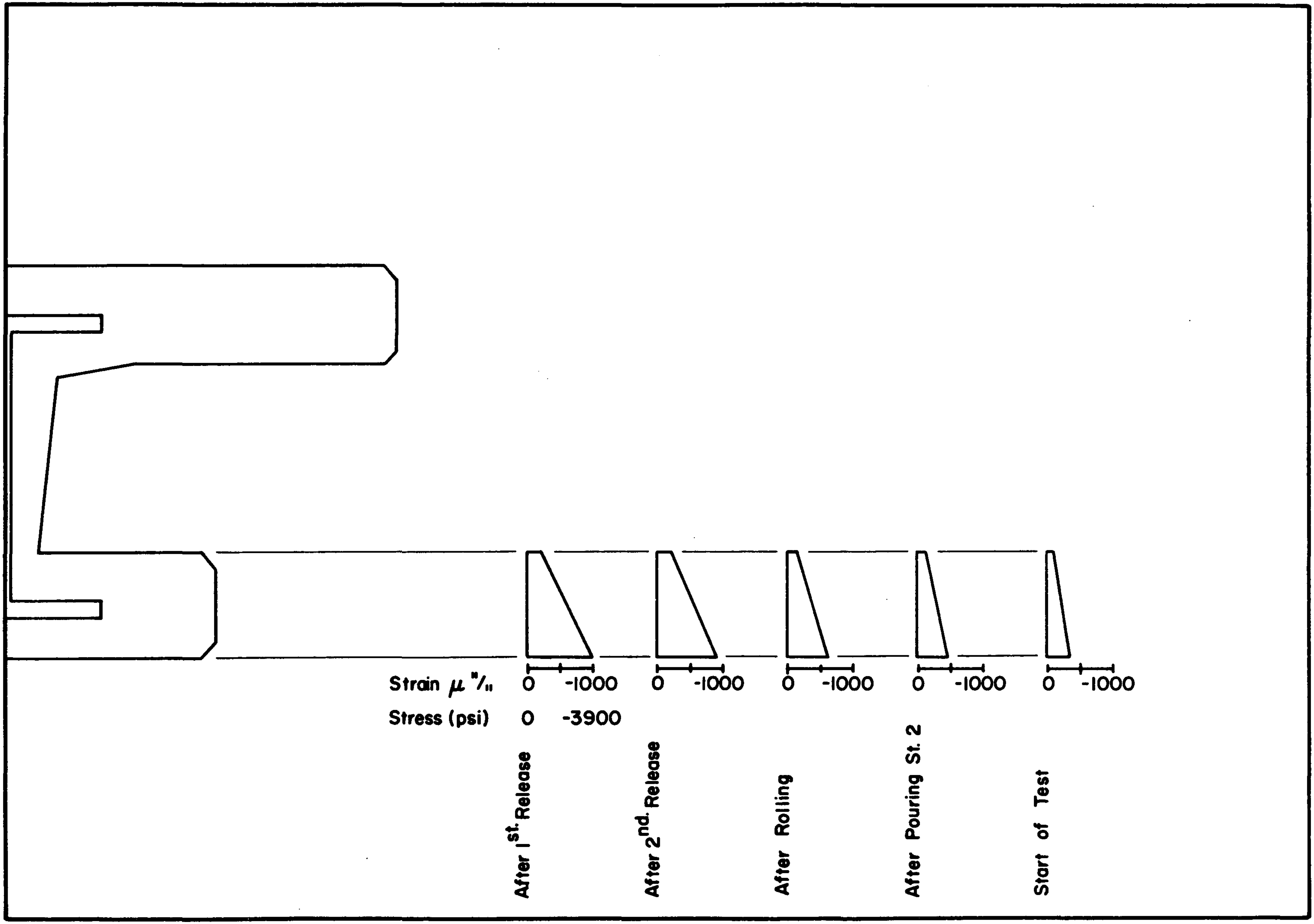


FIG. 13 Px-1, STAGE I CONCRETE STRAINS AT STATION 1, RELEASE OF PREFLEXING LOADS TO START OF TEST

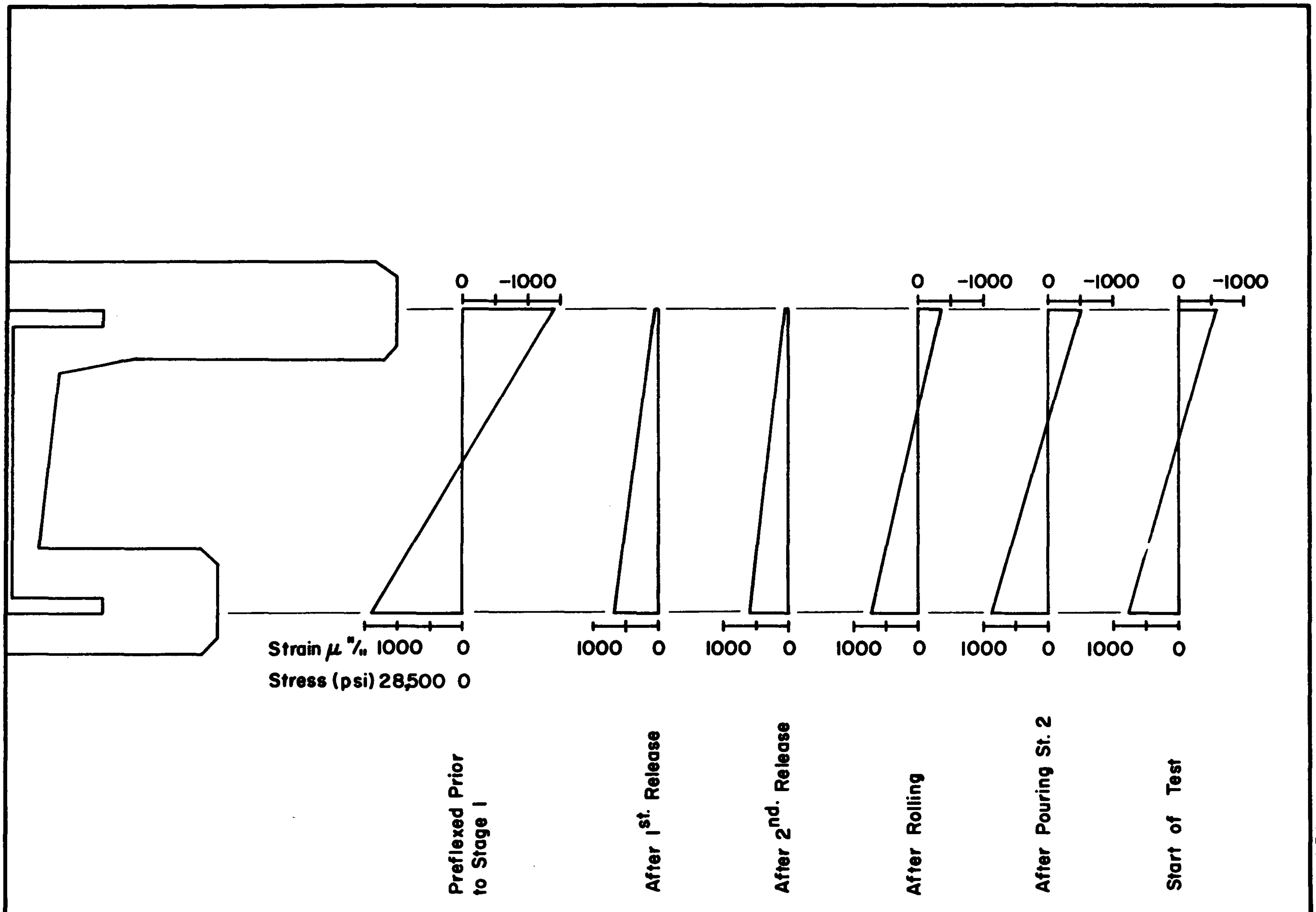


FIG. 14 Px-1, STEEL STRAINS AT STATION 2' END OF PREFLEXING TO START OF TEST

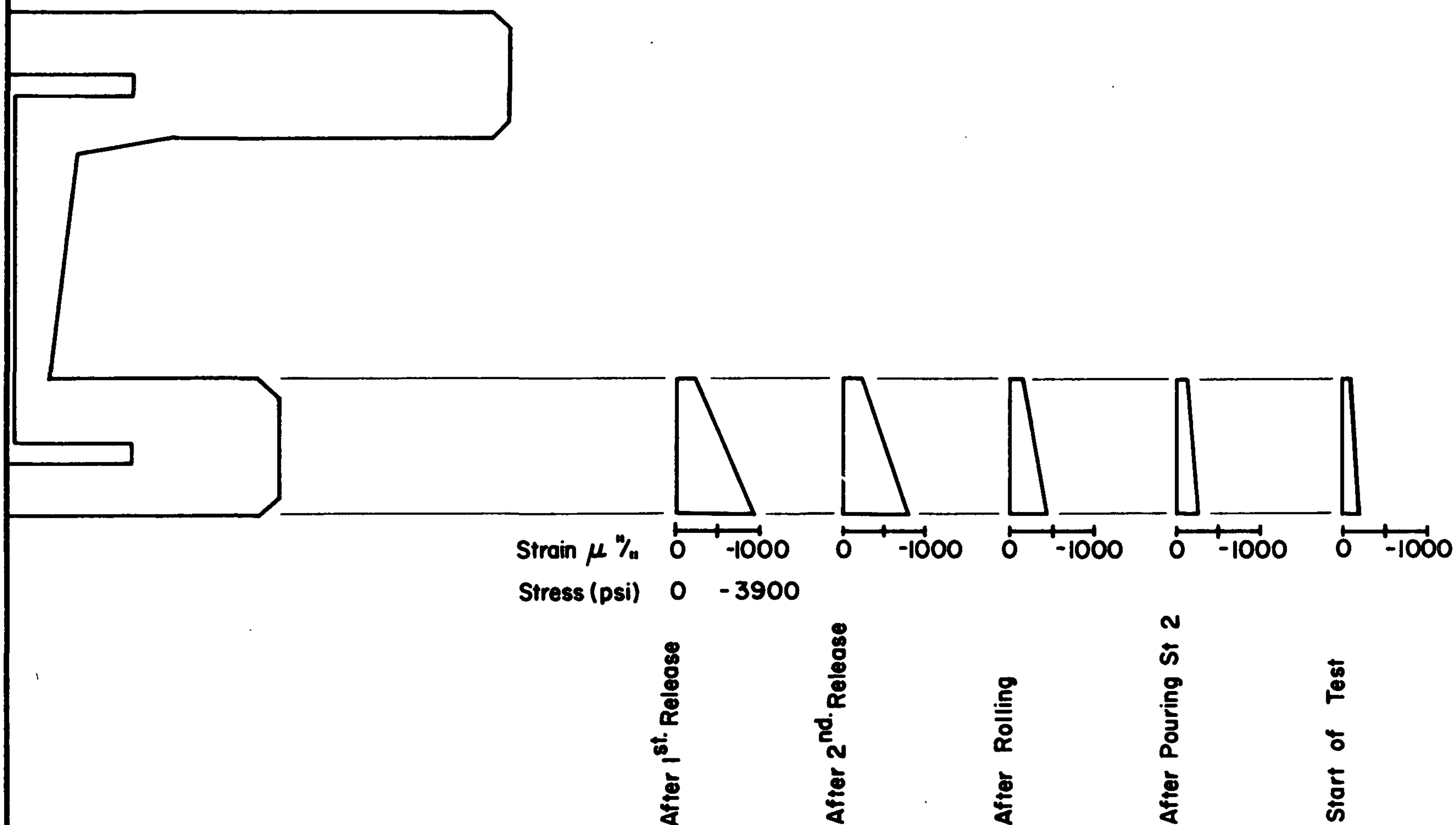


FIG. 15 Px-1, STAGE 1 CONCRETE STRAINS AT STATION 2' RELEASE OF PREFLEXING LOADS TO START OF TEST

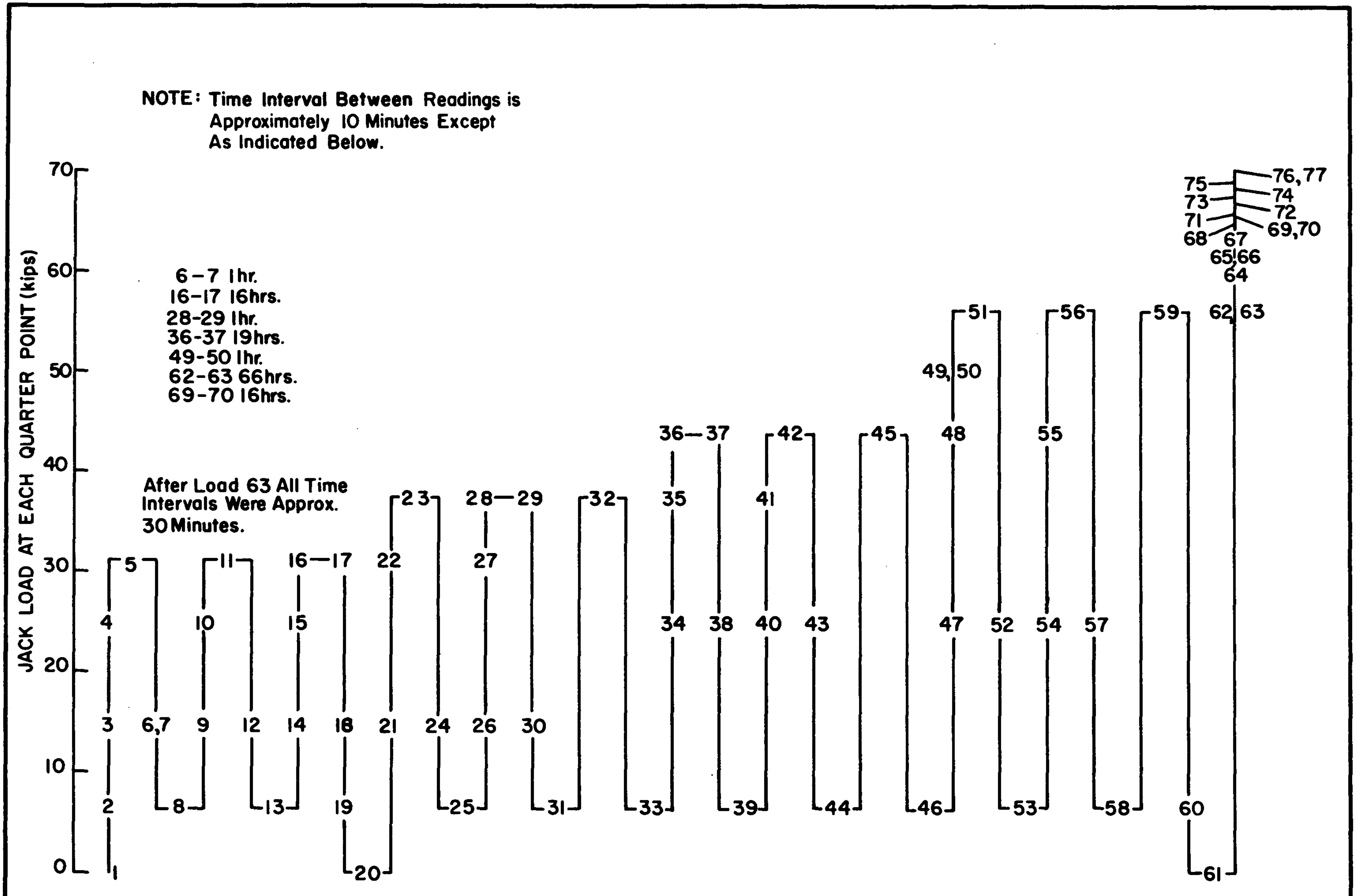


FIG. 16 Re-1, SEQUENCE OF LOADING

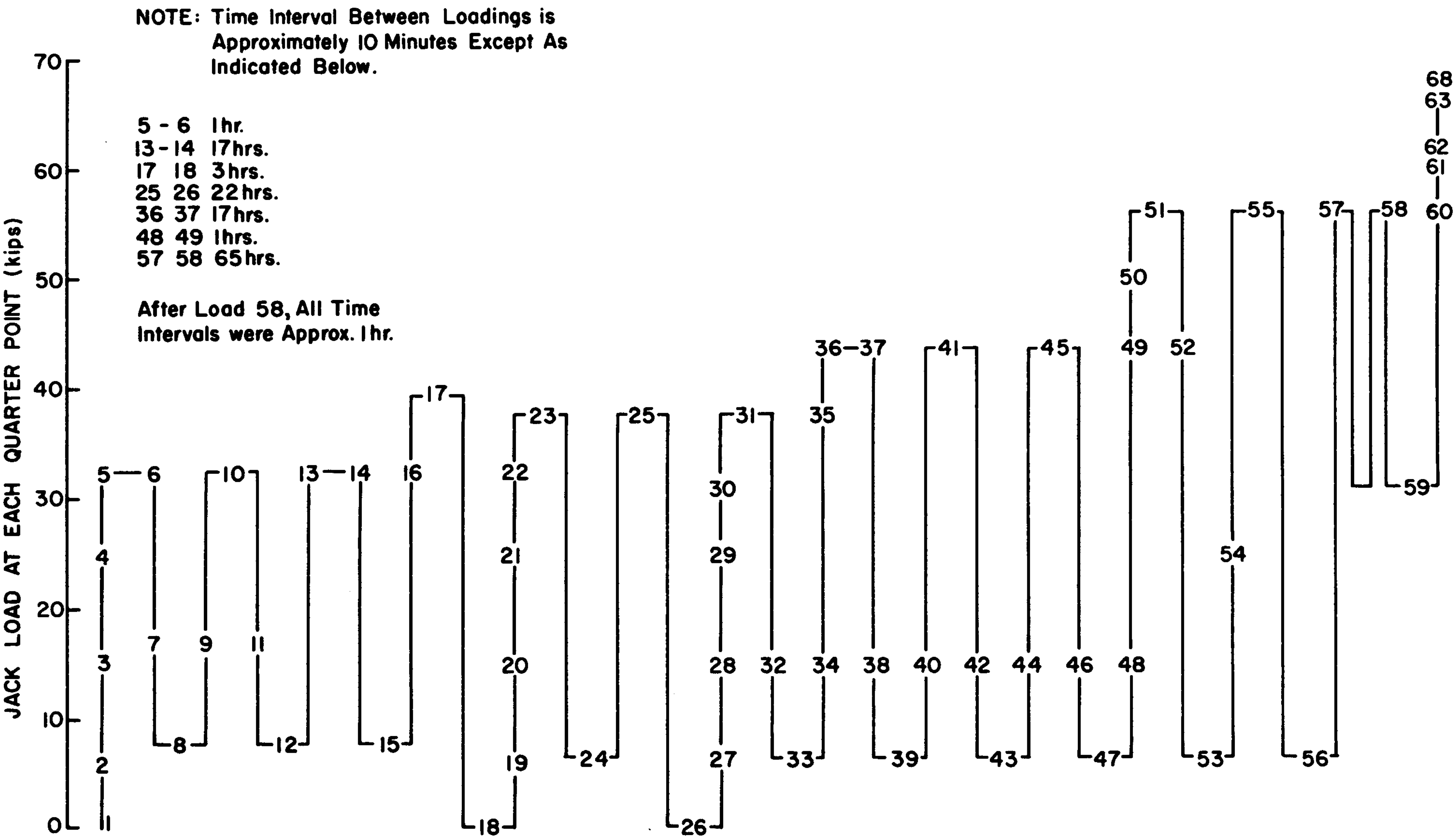


FIG. 17 Px-1, SEQUENCE OF LOADING

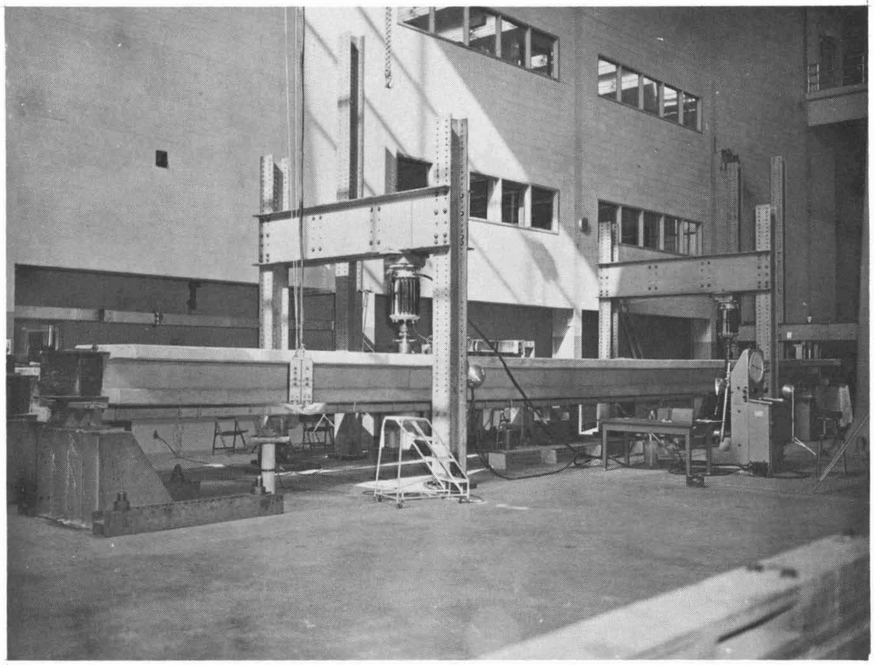


FIG. 18 Re-1 AT DESIGN LOAD

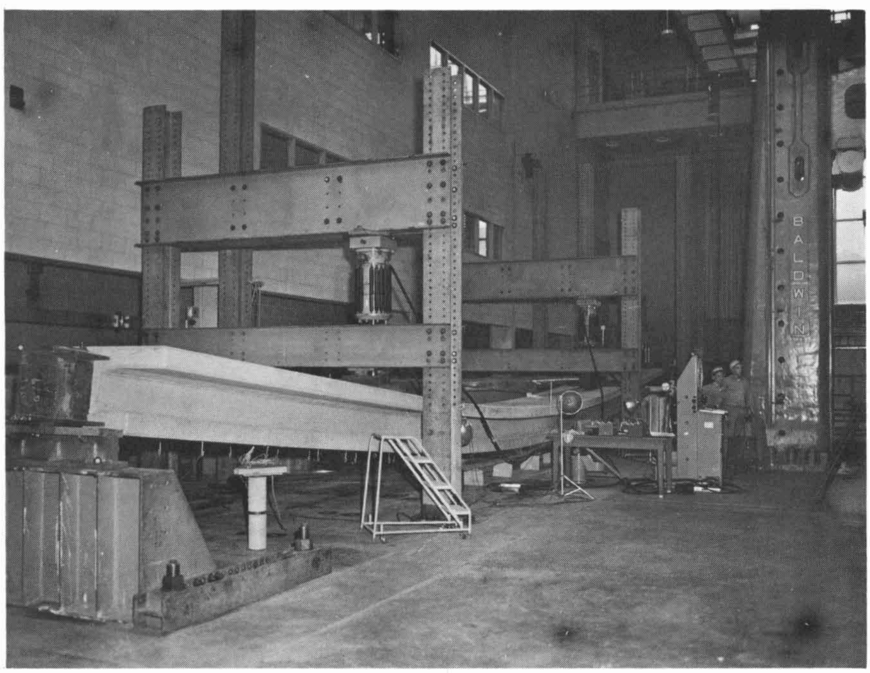


FIG. 19 BLOCKING ARRANGEMENT FOR RE-STROKING JACKS

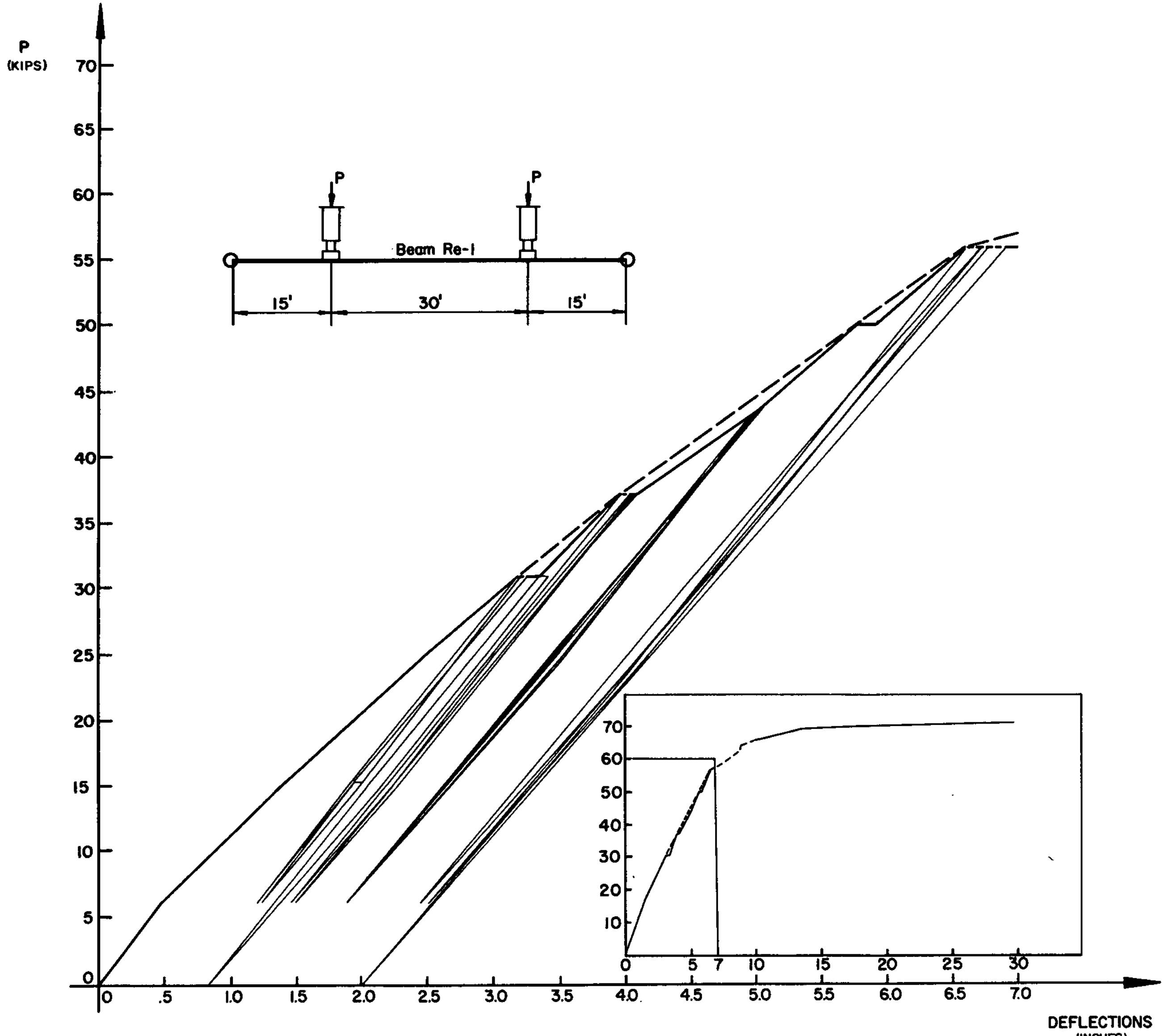


FIG. 20 Re-1, LOAD-DEFLECTION CURVE

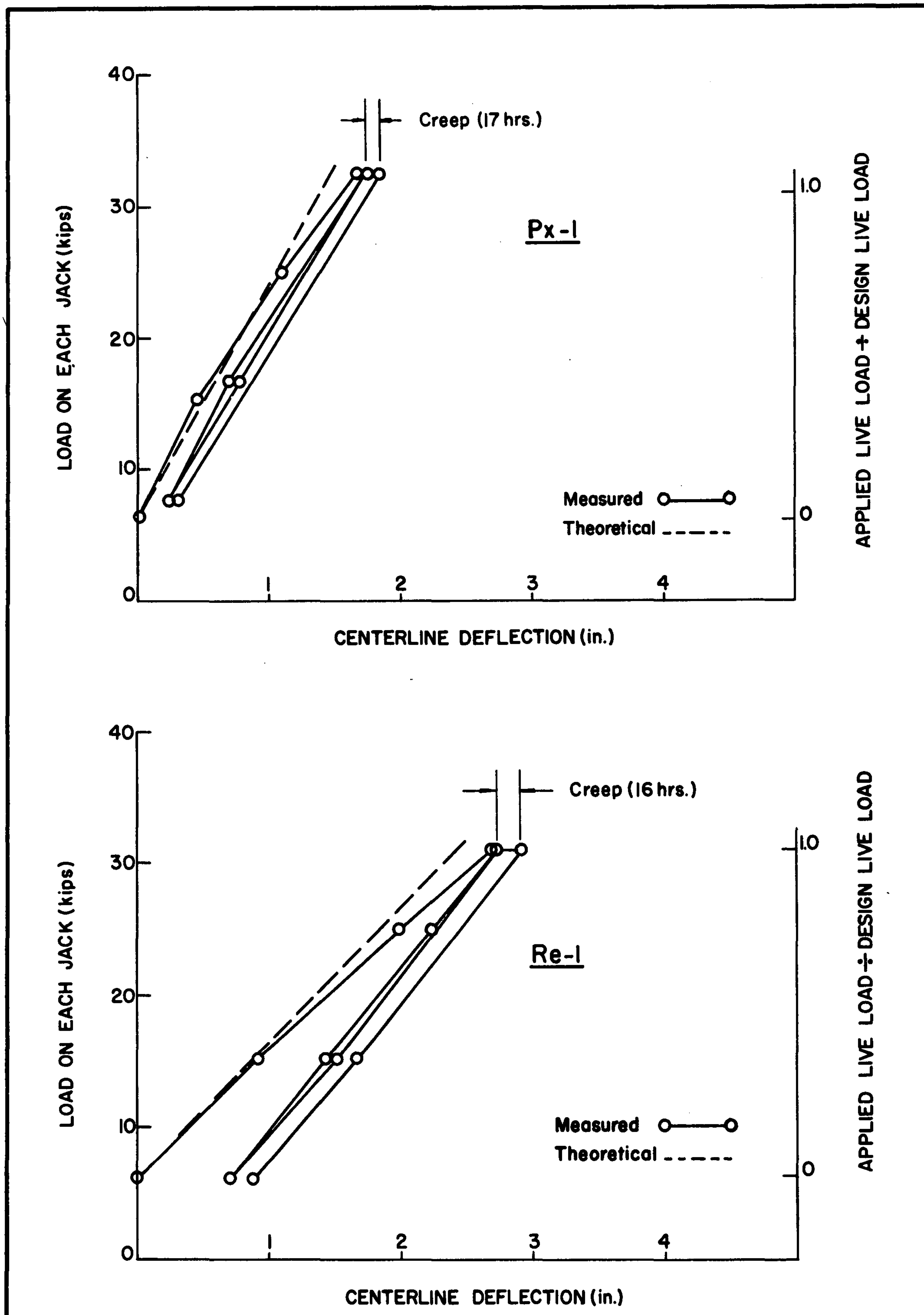


FIG. 21 LIVE LOAD DEFLECTIONS AT DESIGN LOAD

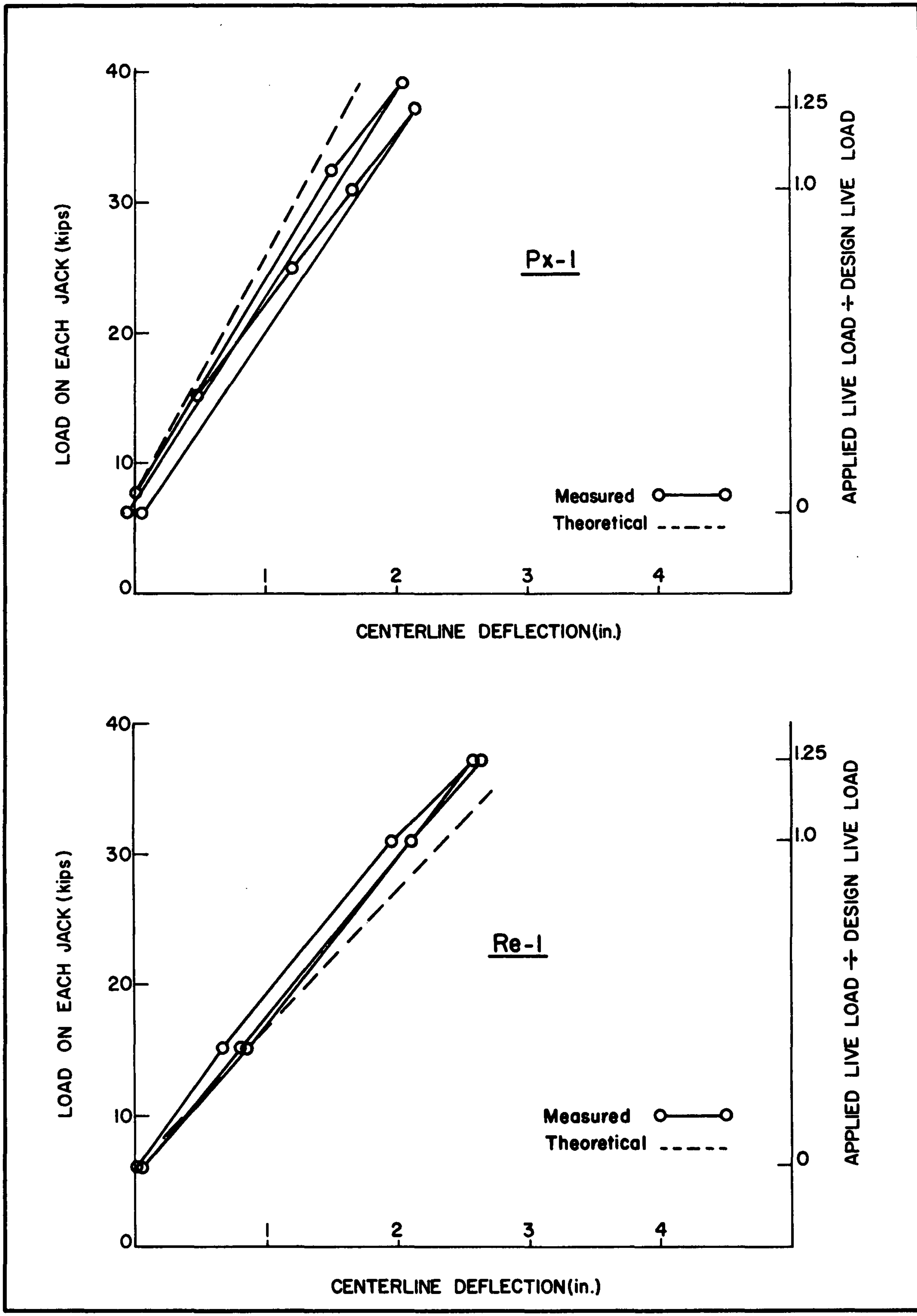


FIG. 22 LIVE LOAD DEFLECTION AT 1.25 x DESIGN LOAD

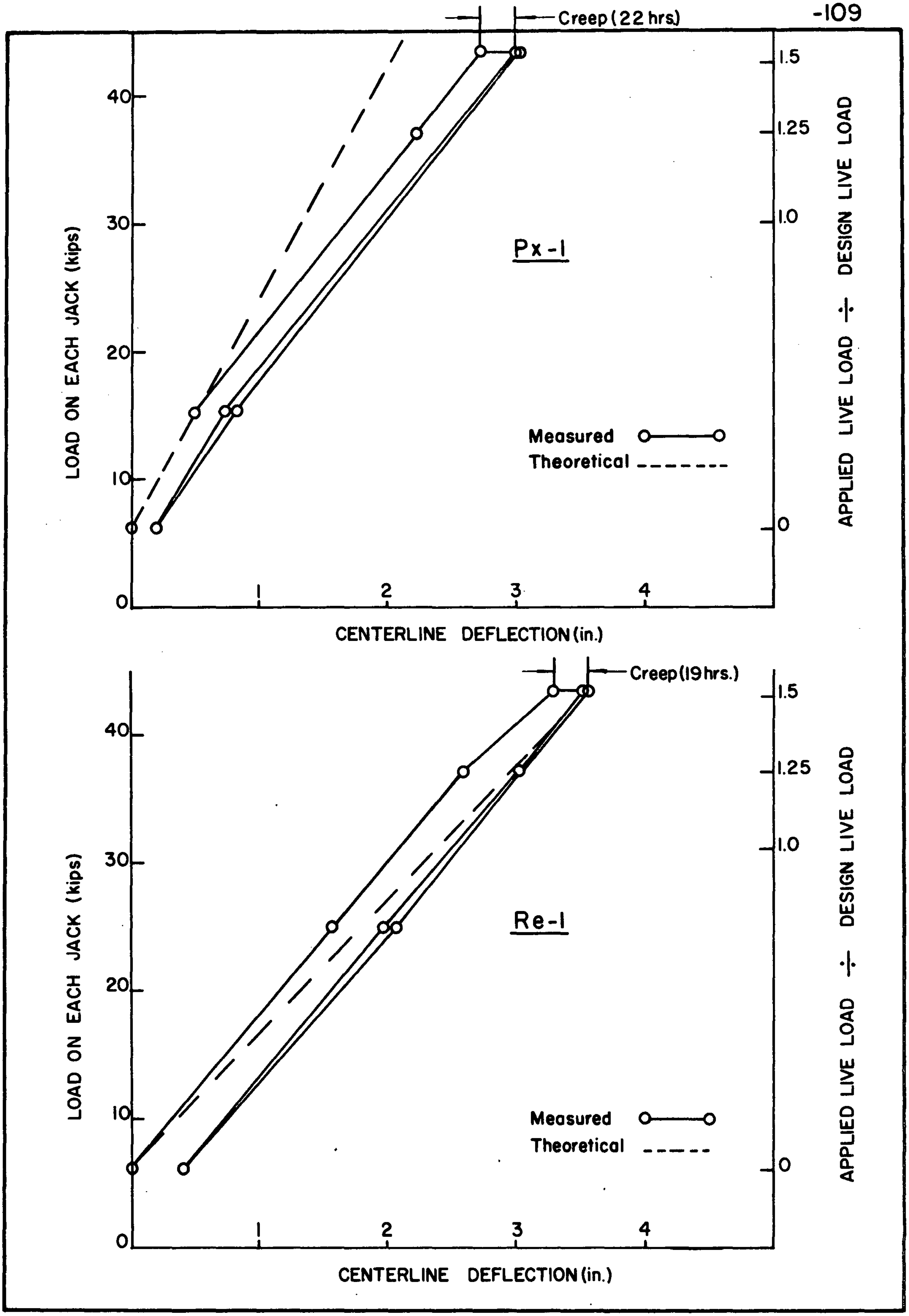
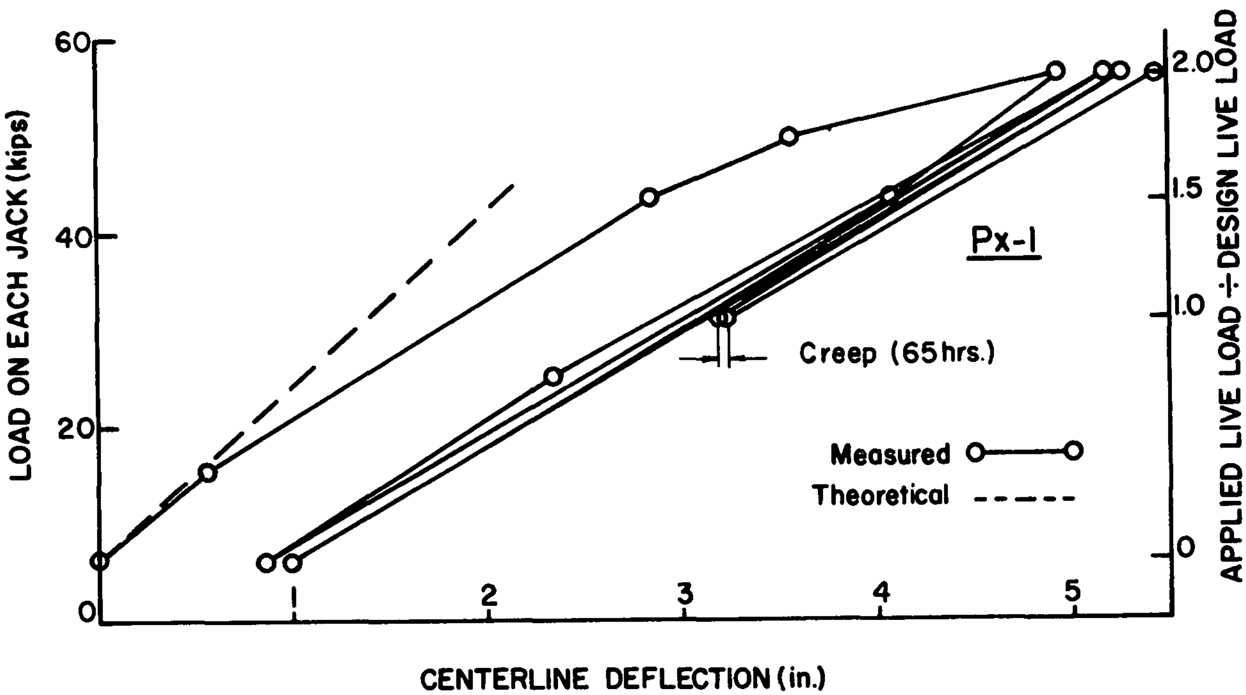


FIG. 23 LIVE LOAD DEFLECTIONS AT 1.5 x DESIGN LOAD



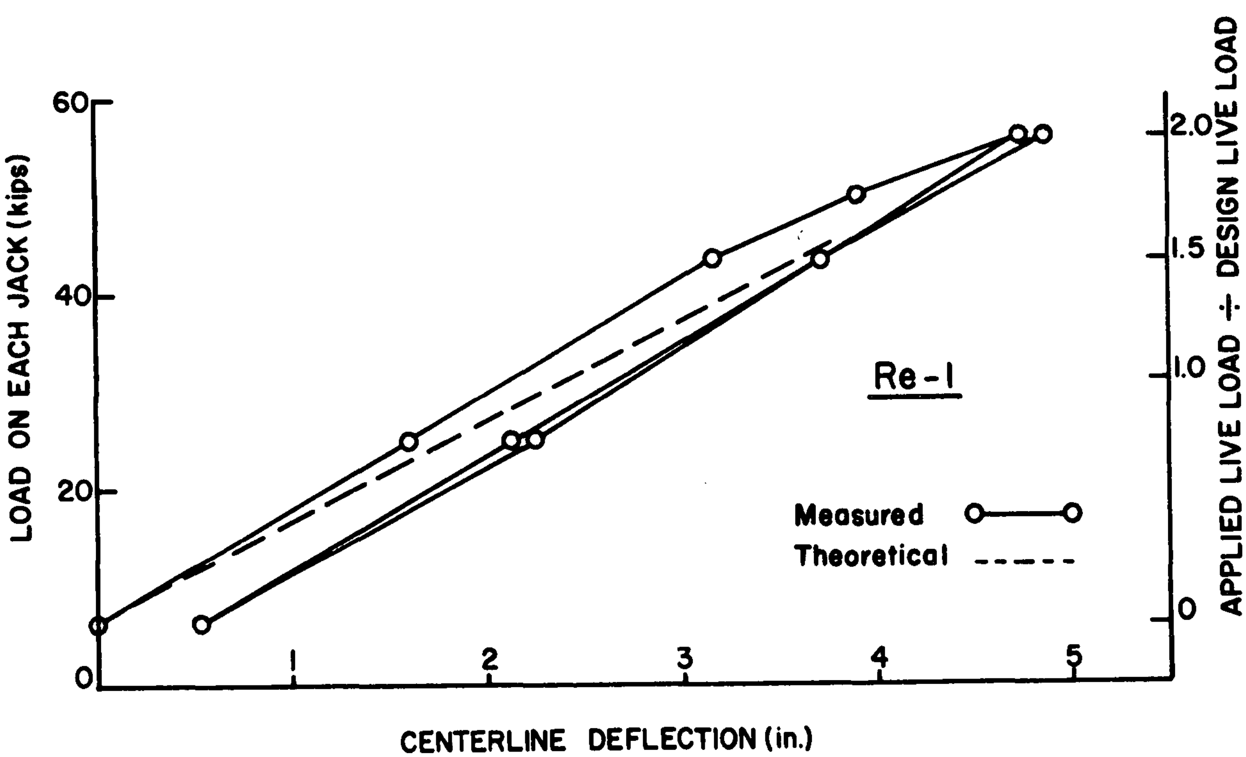


FIG. 24 LIVE LOAD DEFLECTIONS AT 2 x DESIGN LOAD

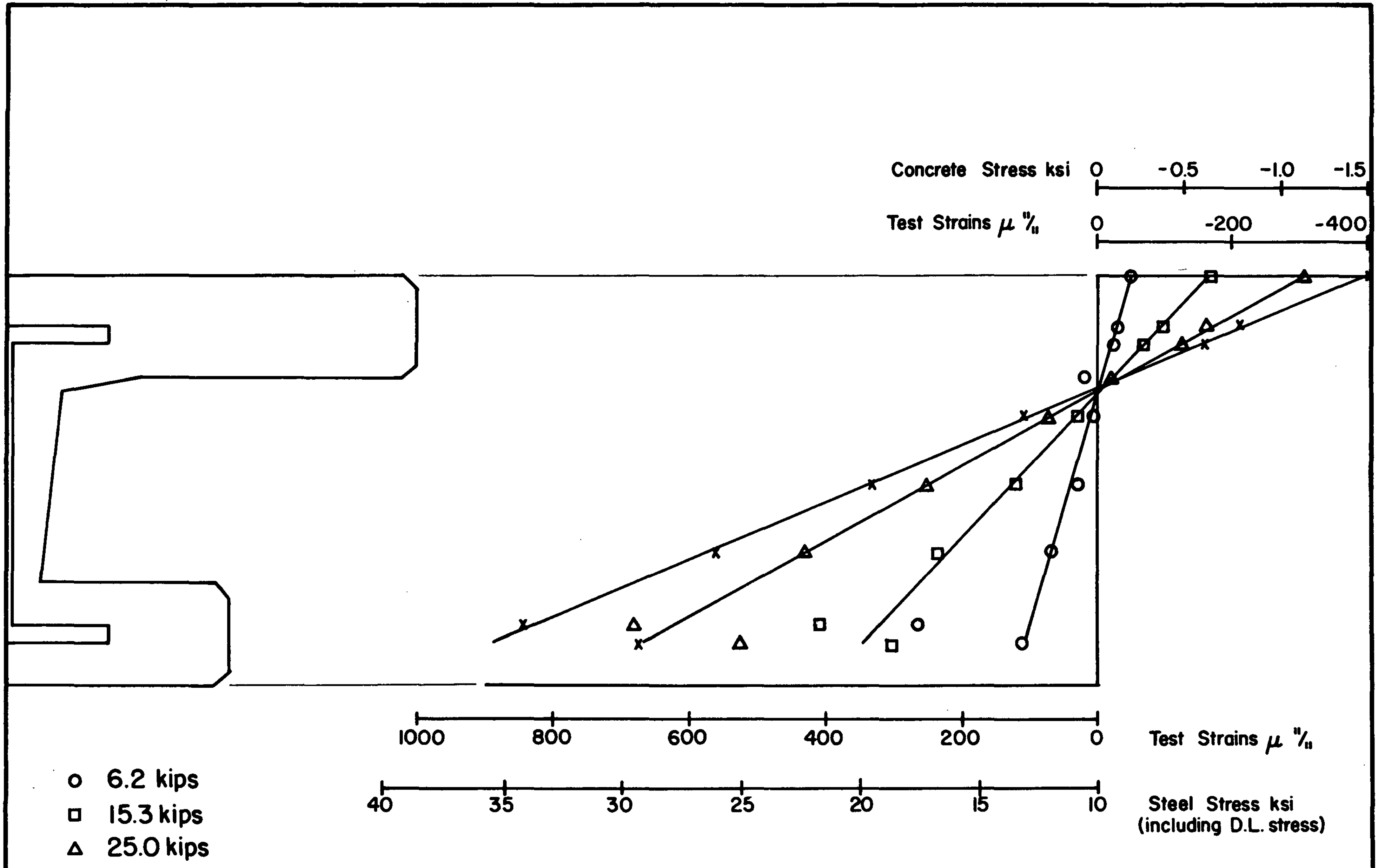


FIG. 25 Re-I, STRAINS AT STATION I FIRST LOAD SEQUENCE

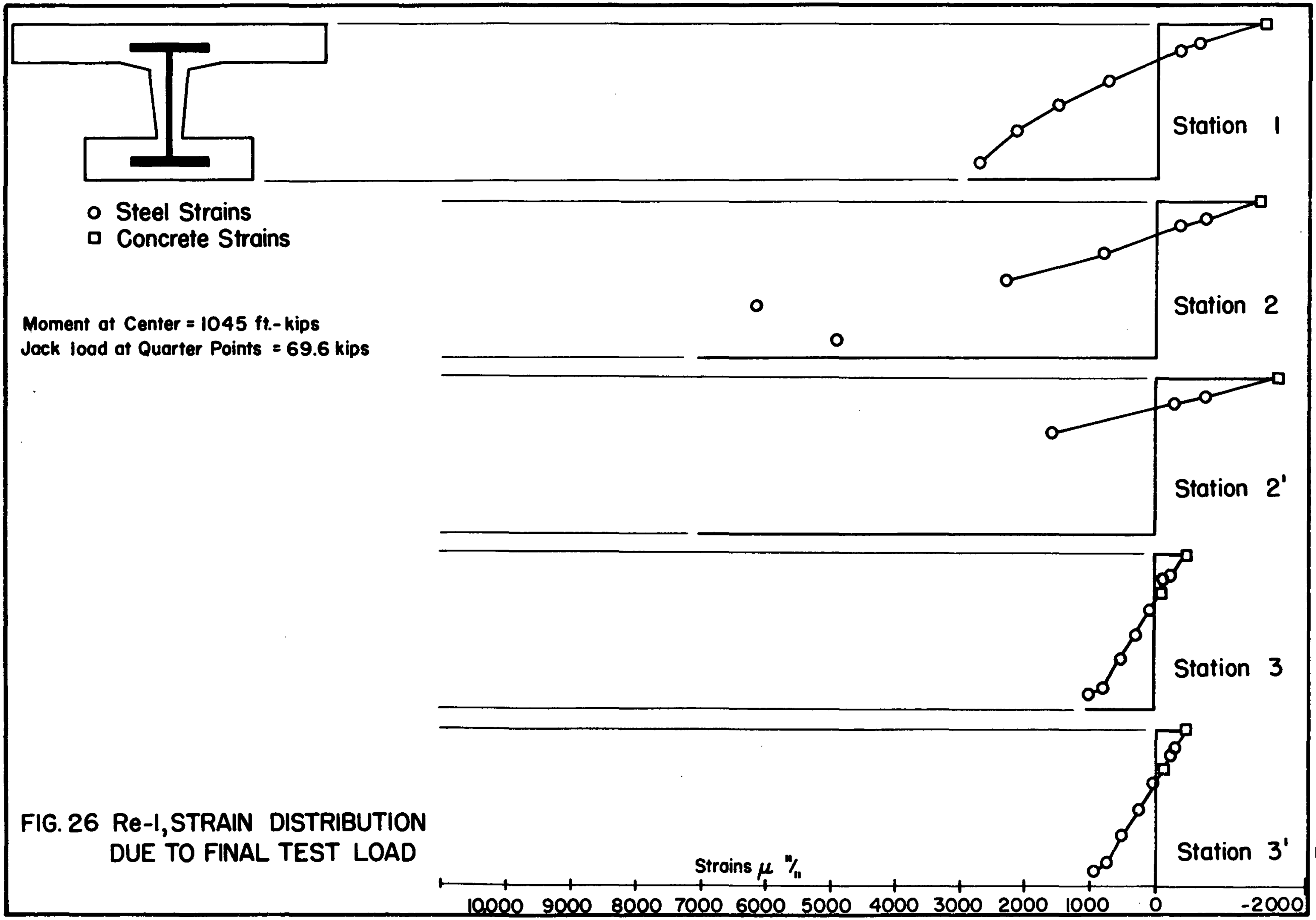
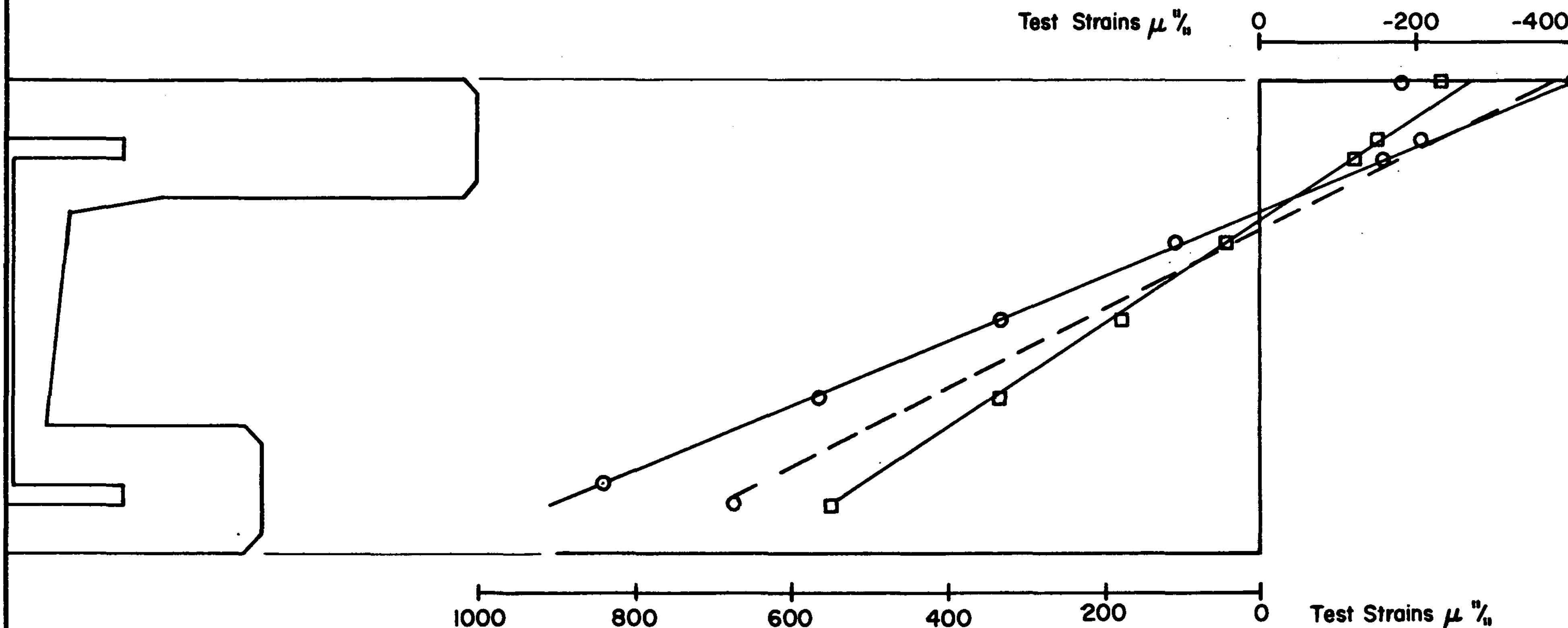


FIG. 26 Re-1, STRAIN DISTRIBUTION DUE TO FINAL TEST LOAD



o——o Average of test points first cycle
□——□ Average of test points second cycle
- - - - - Theoretical line assuming Stage I inactive

Applied Load : Design Load
31.0 kips at each jack

FIG. 27 Re-1, MEASURED AND THEORETICAL STRAINS AT STATION I

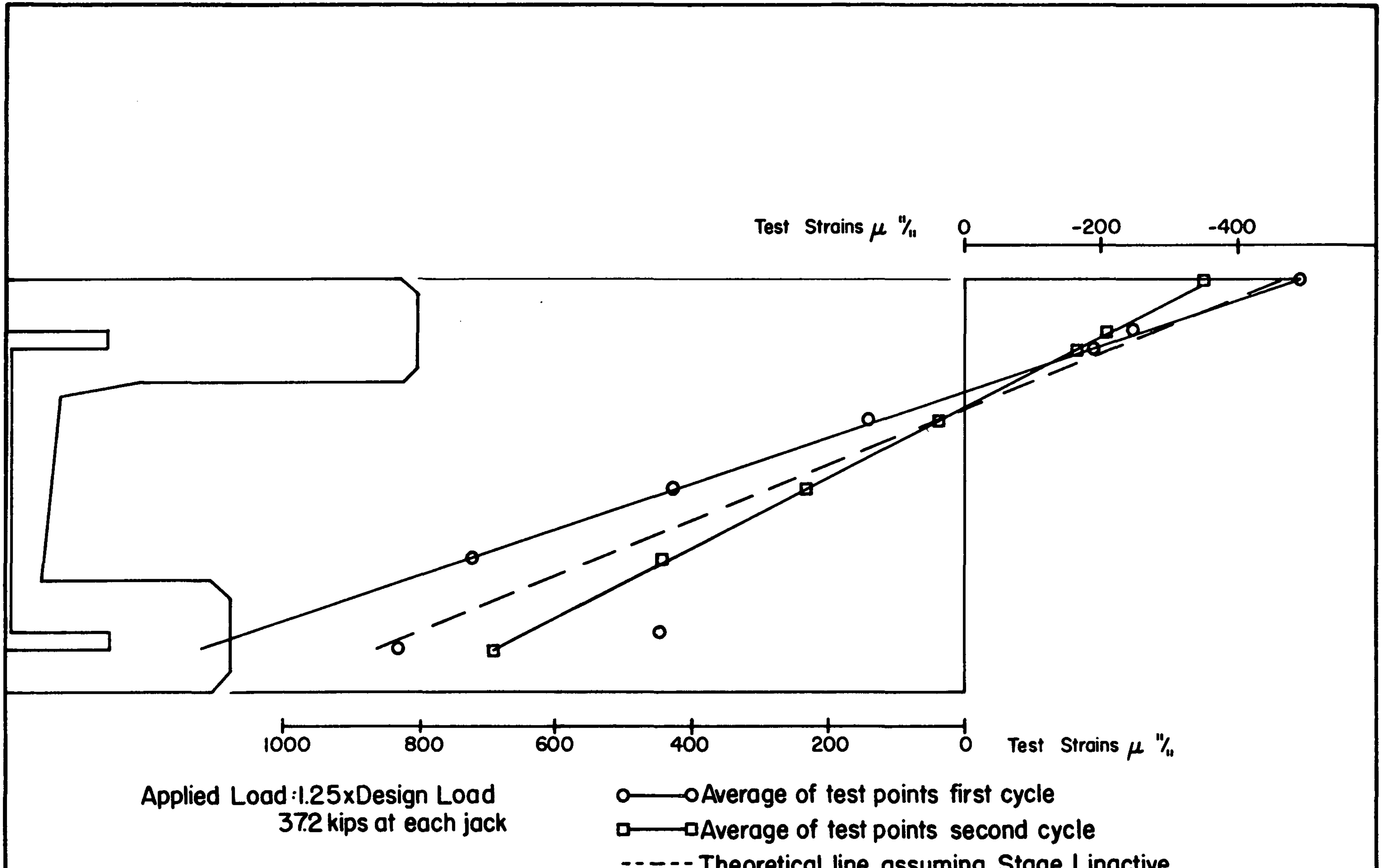
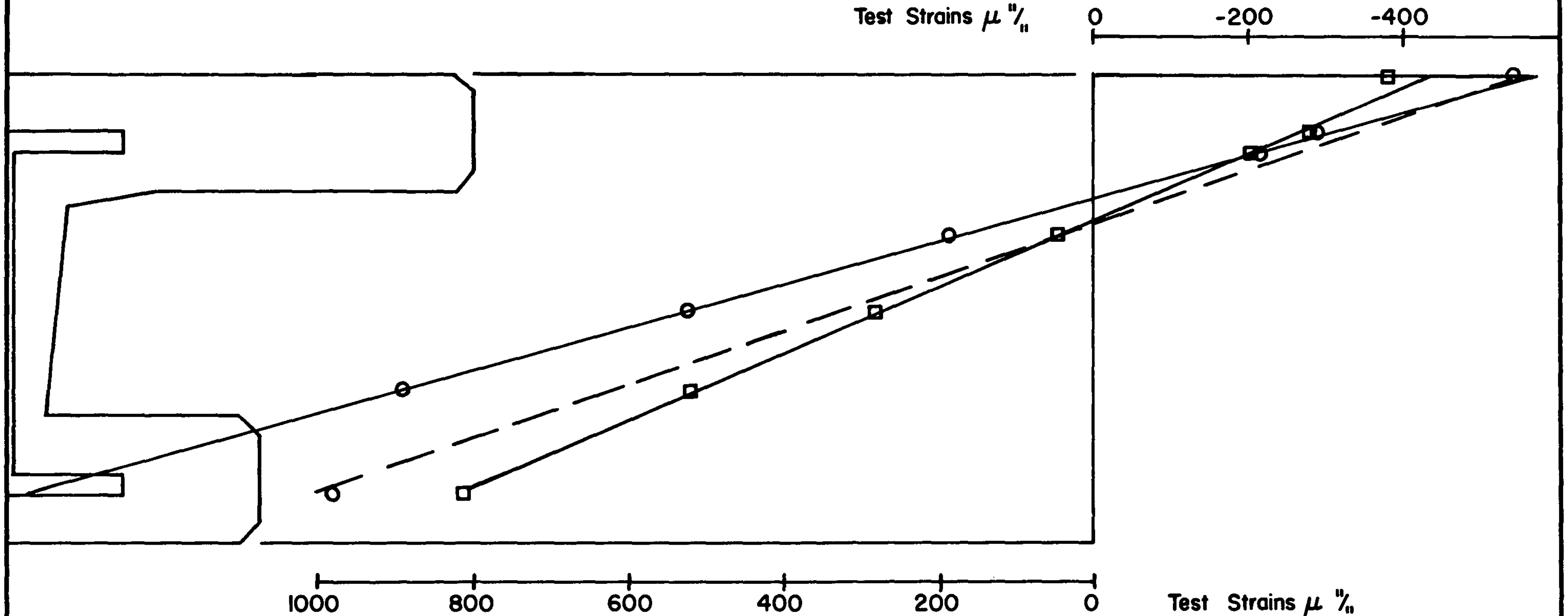


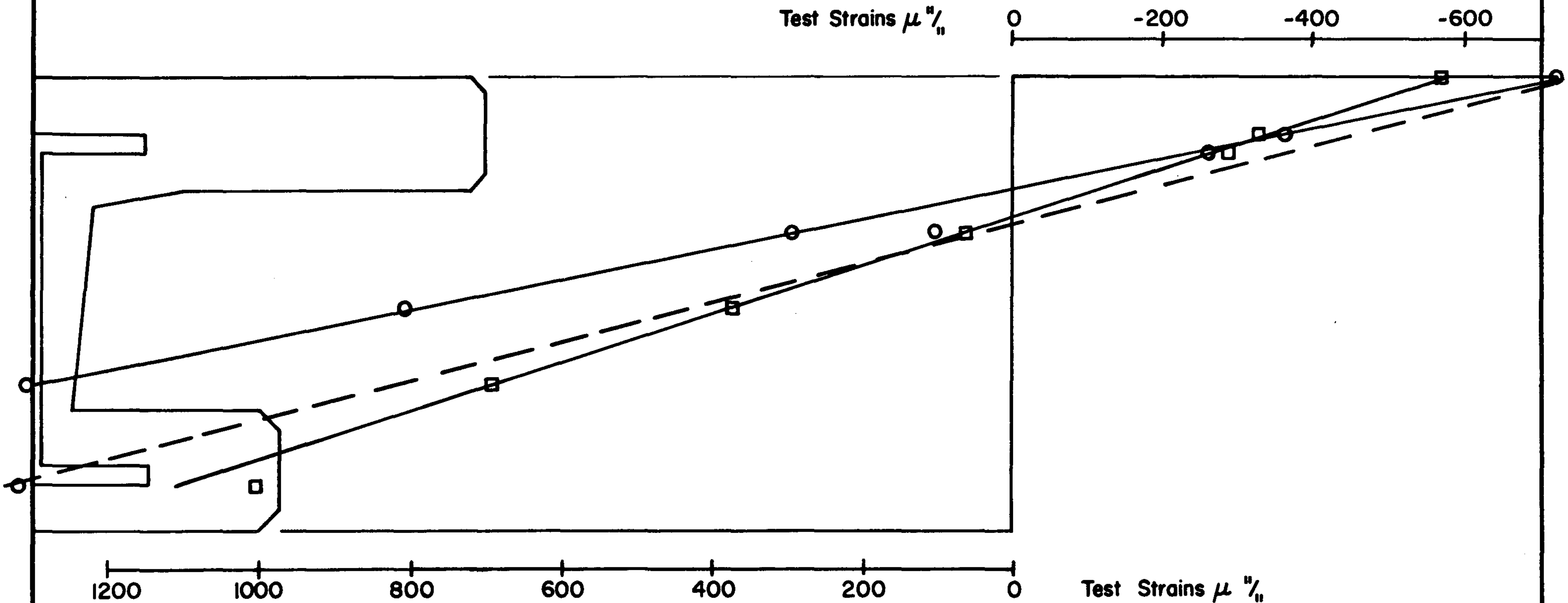
FIG. 28 Re-1, MEASURED AND THEORETICAL STRAINS AT STATION 1



Applied Load: 1.5xDesign Load
43.5 kips at each jack

o——o Average of test points first cycle
□——□ Average of test points second cycle
----- Theoretical line assuming Stage I inactive

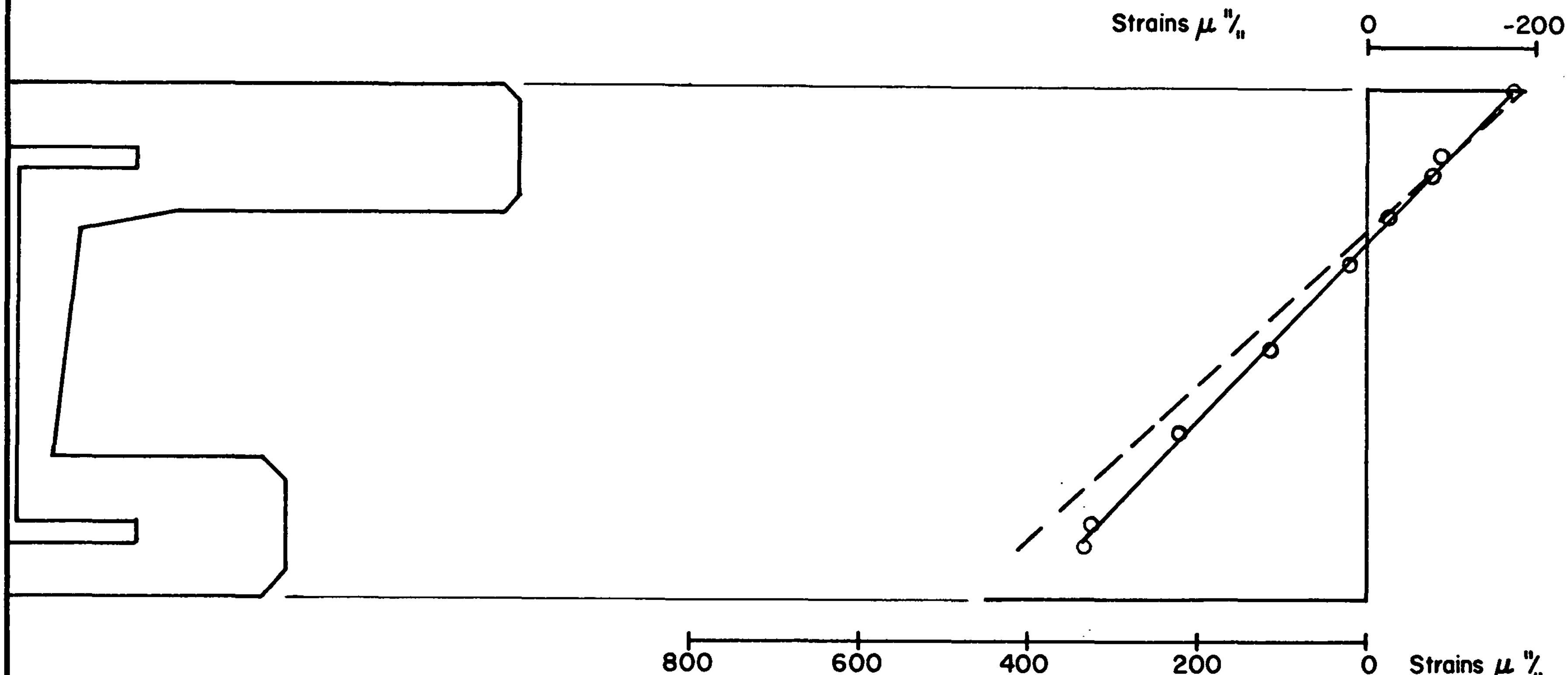
FIG. 29 Re-1, MEASURED AND THEORETICAL STRAINS AT STATION 1



Applied Load : 2 x Design Load
56.0kips at each jack

o——o Average of test points first cycle
□——□ Average of test points first cycle
----- Theoretical line assuming Stage I inactive

FIG. 30 Re-I, MEASURED AND THEORETICAL STRAINS AT STATION I



Applied Load: Design Load
31.0 kips at each jack

o——o Average Test Points
- - - - - Theoretical line assuming uncracked section

FIG. 31 Re-1, MEASURED AND THEORETICAL STRAINS AT STATION 3

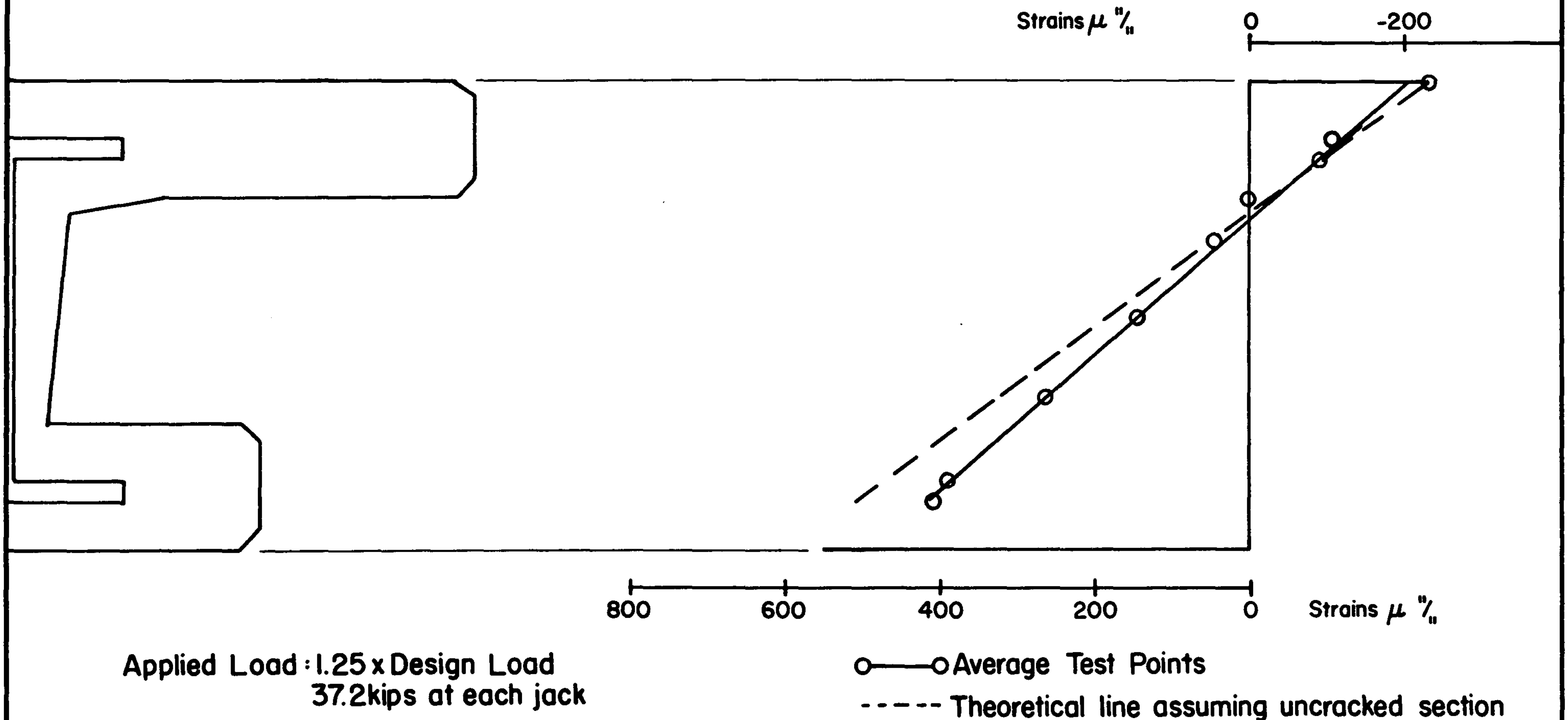
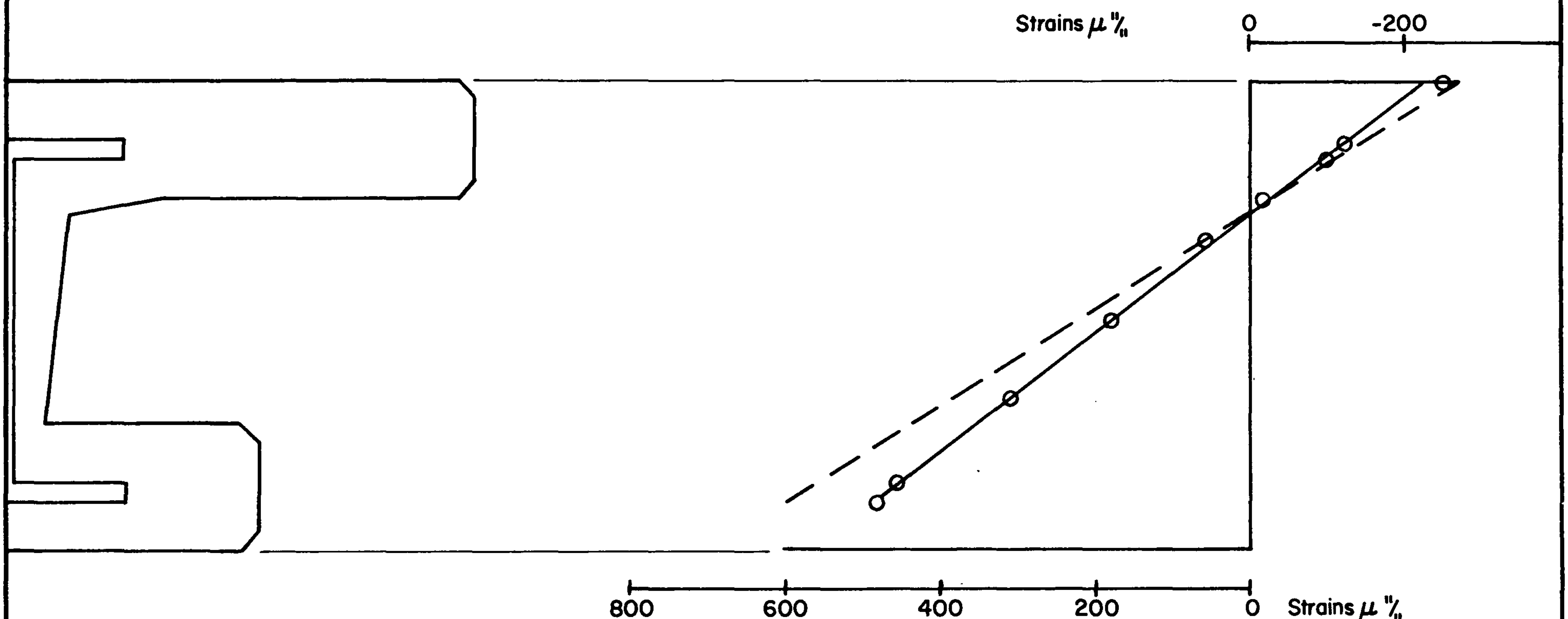


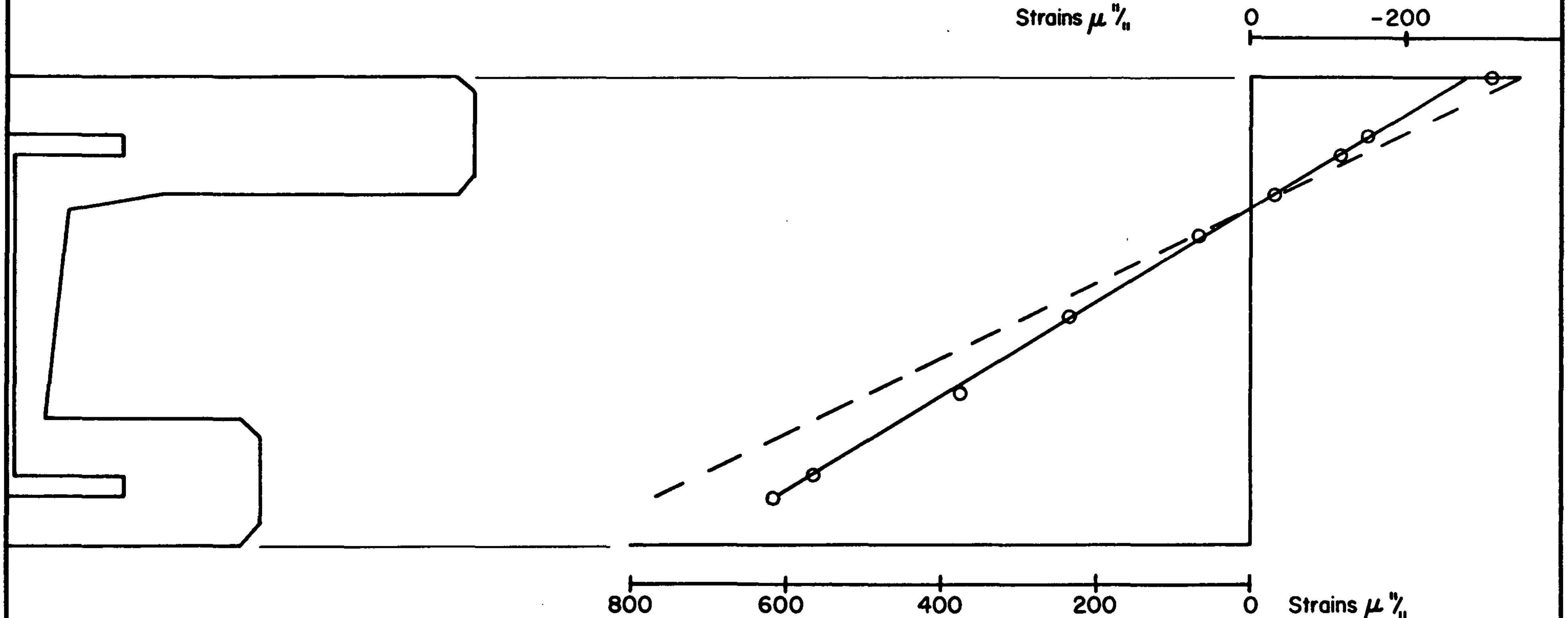
FIG. 32 Re-1, MEASURED AND THEORETICAL STRAINS AT STATION 3



Applied Load : 1.5 x Design Load
43.5 kips at each jack

o——o Average Test Points
- - - - - Theoretical line assuming uncracked section

FIG. 33 Re-1, MEASURED AND THEORETICAL STRAINS AT STATION 3



Applied Load: 2 x Design Load
56.0kips at each jack

o——o Average Test Points
- - - - - Theoretical line assuming uncracked section

FIG. 34 Re-1, MEASURED AND THEORETICAL STRAINS AT STATION 3

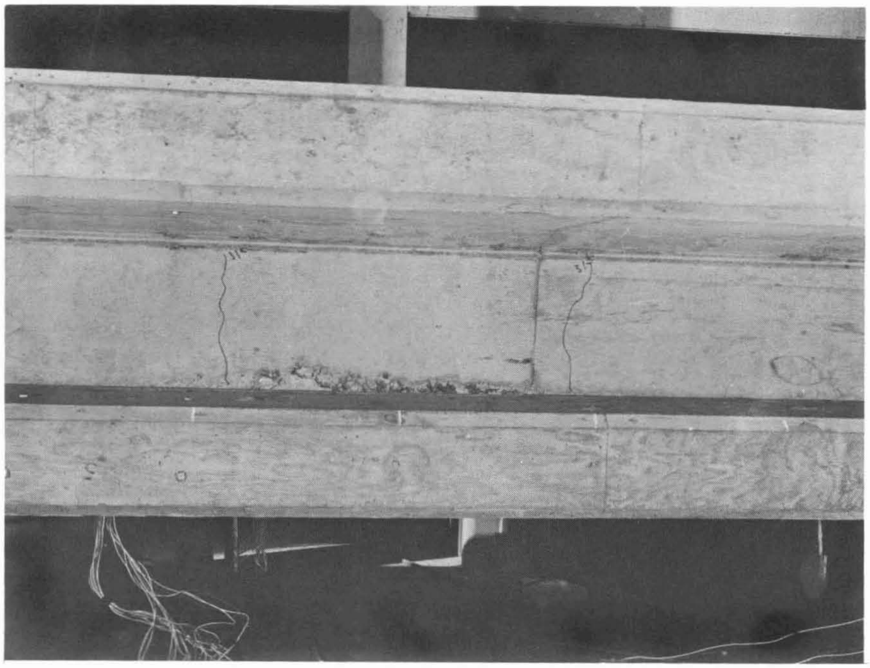


FIG. 35 TENSILE CRACKS (MARKED) IN Re-1 AT DESIGN LOAD

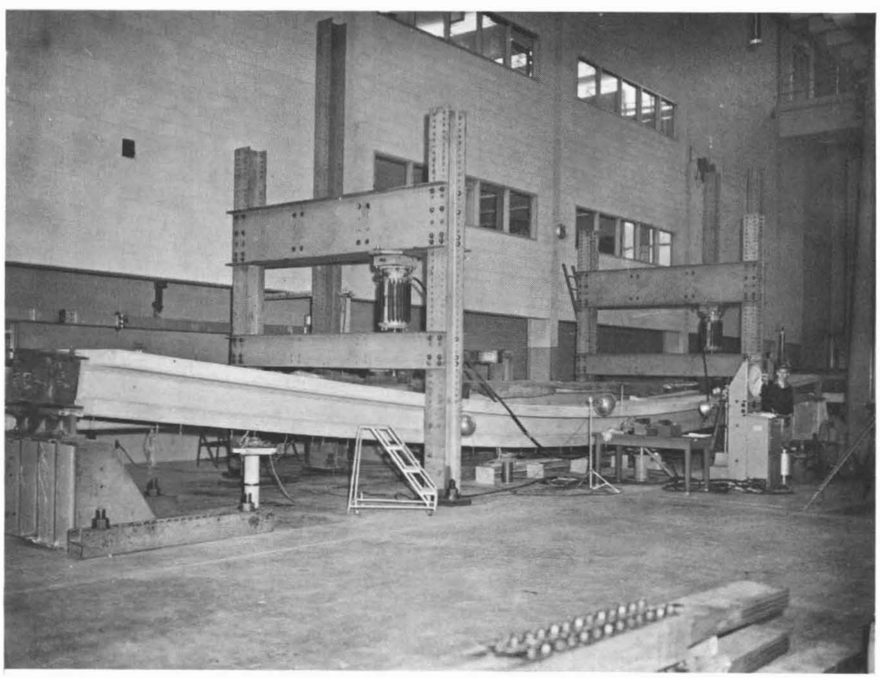


FIG. 36 Re -1 AT ULTIMATE LOAD

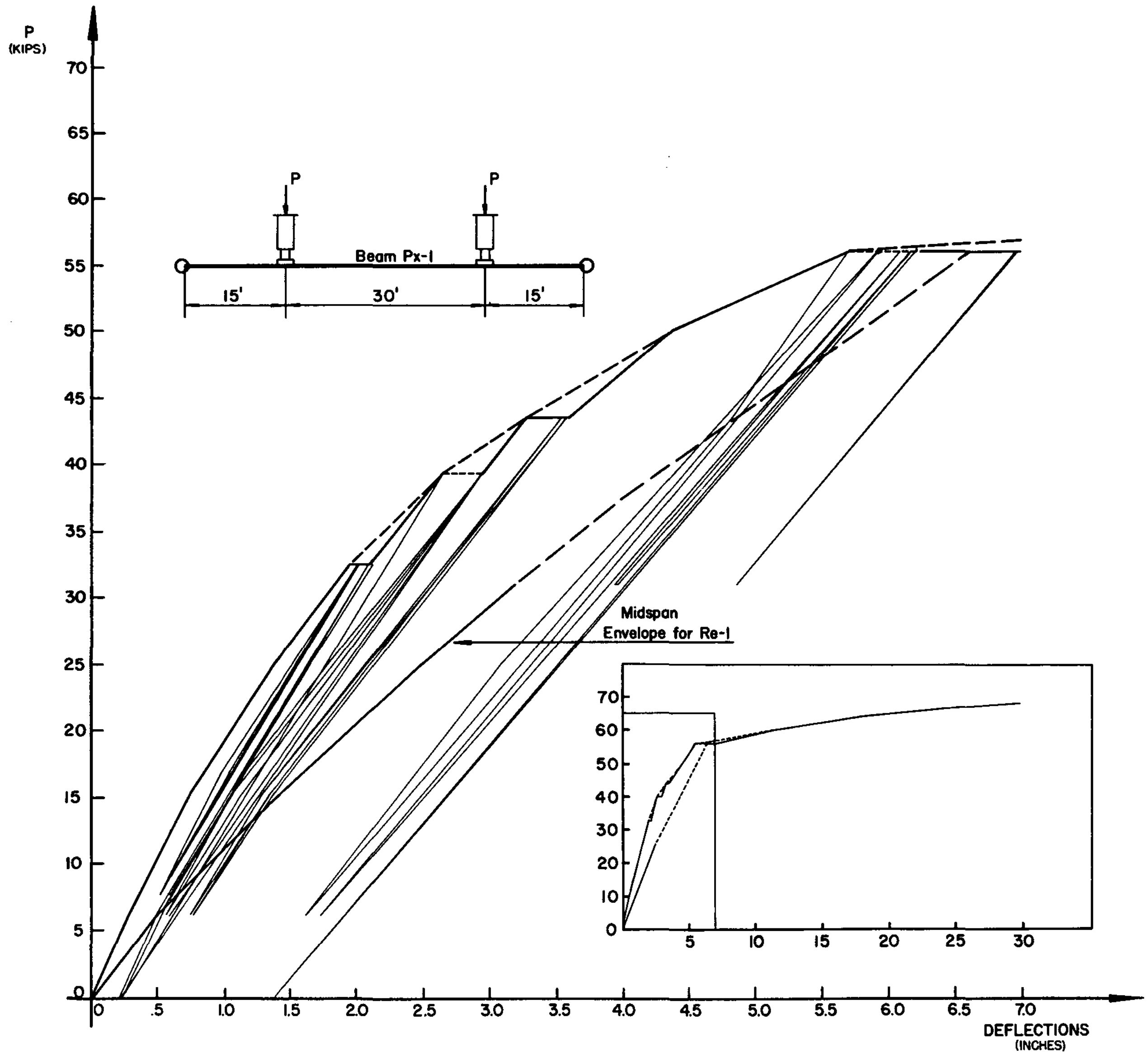


FIG. 37 Px-1 LOAD-DEFLECTION CURVE

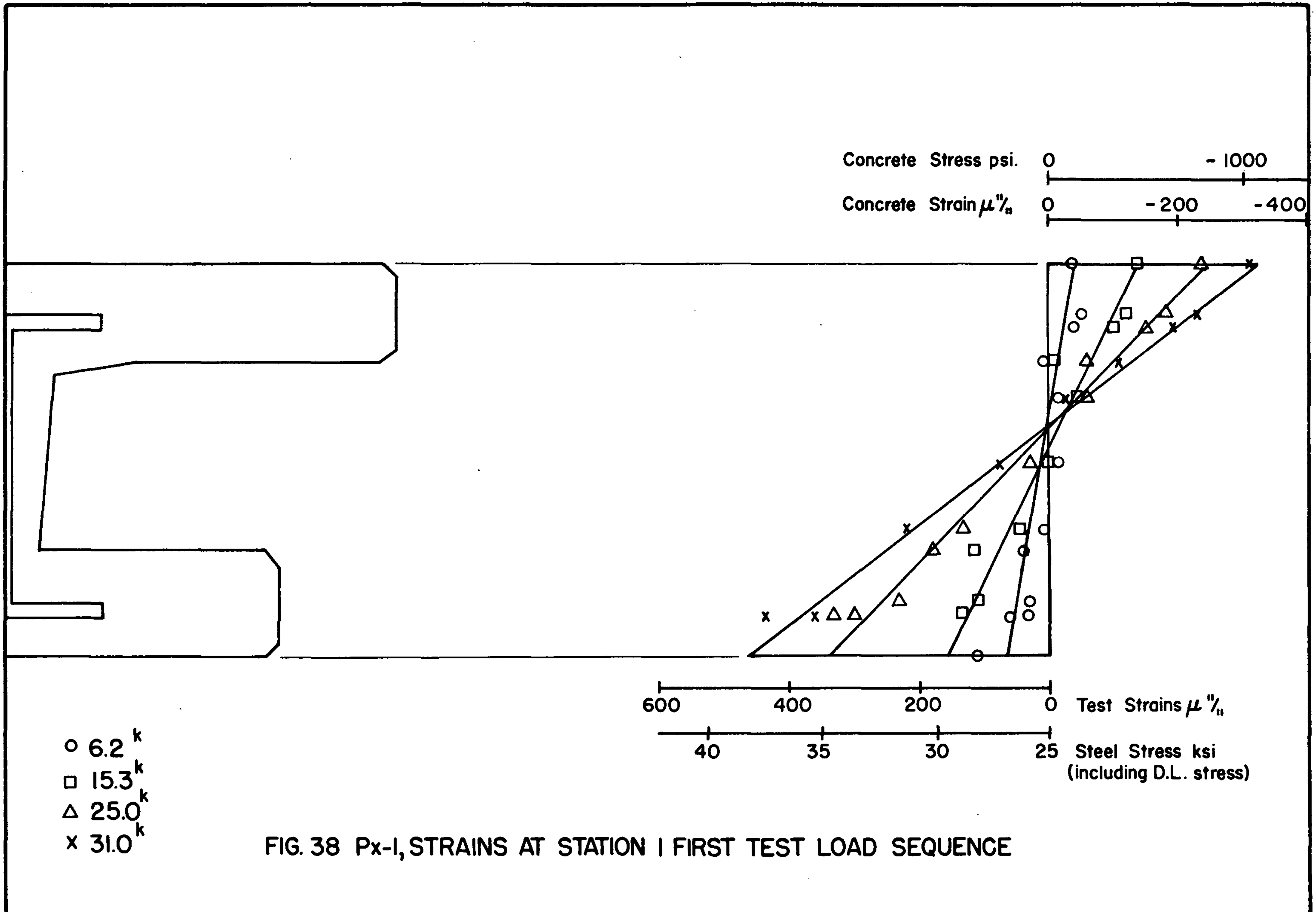


FIG. 38 Px-1, STRAINS AT STATION 1 FIRST TEST LOAD SEQUENCE

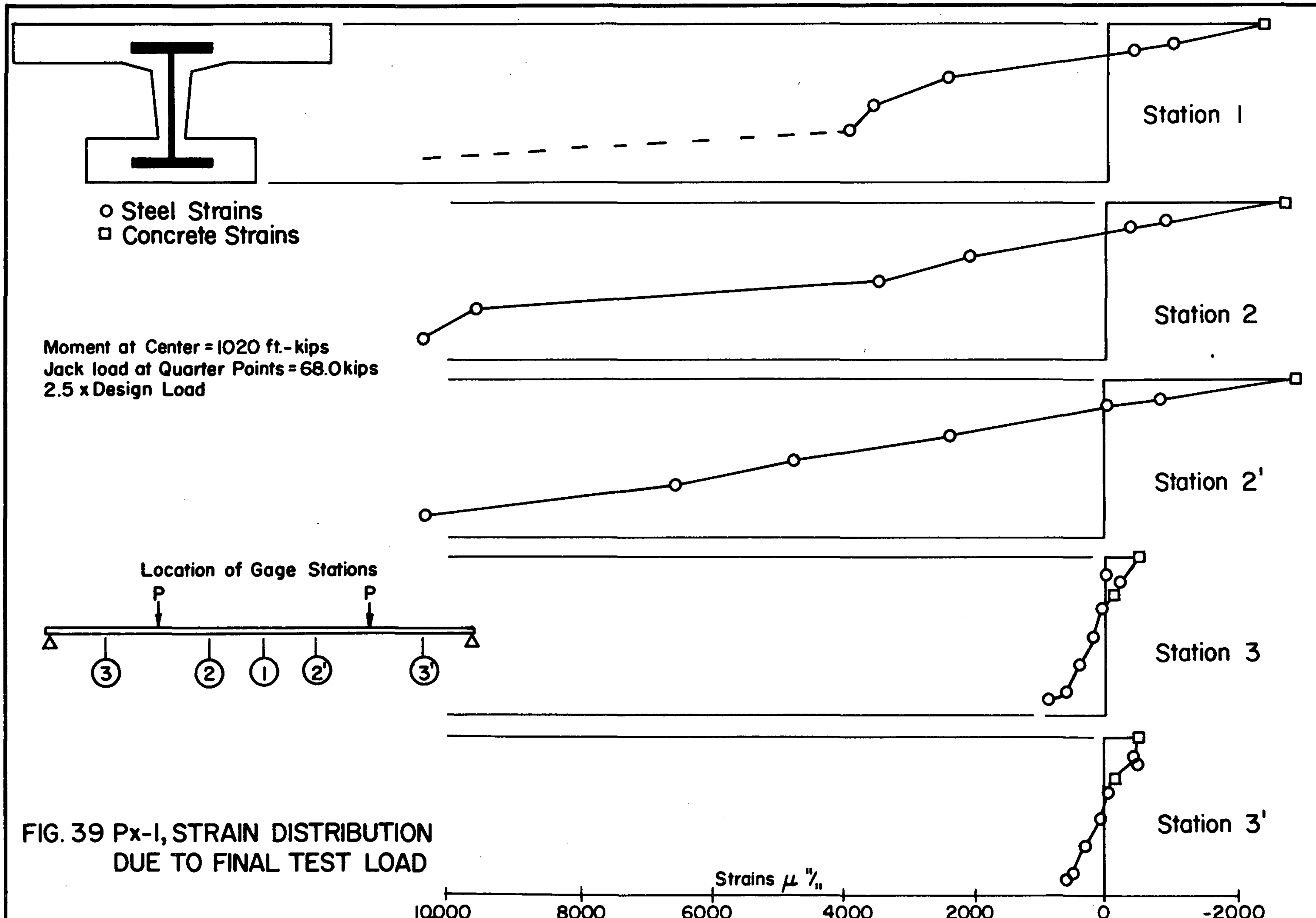
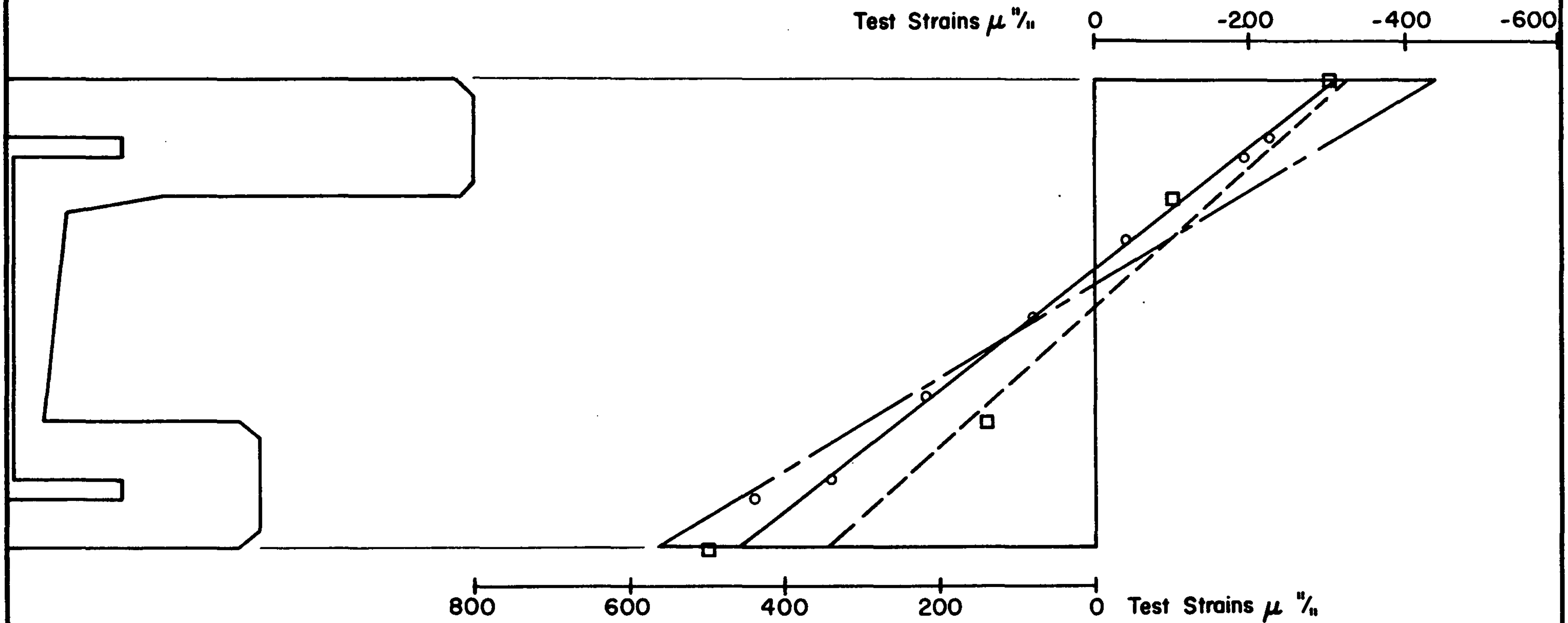


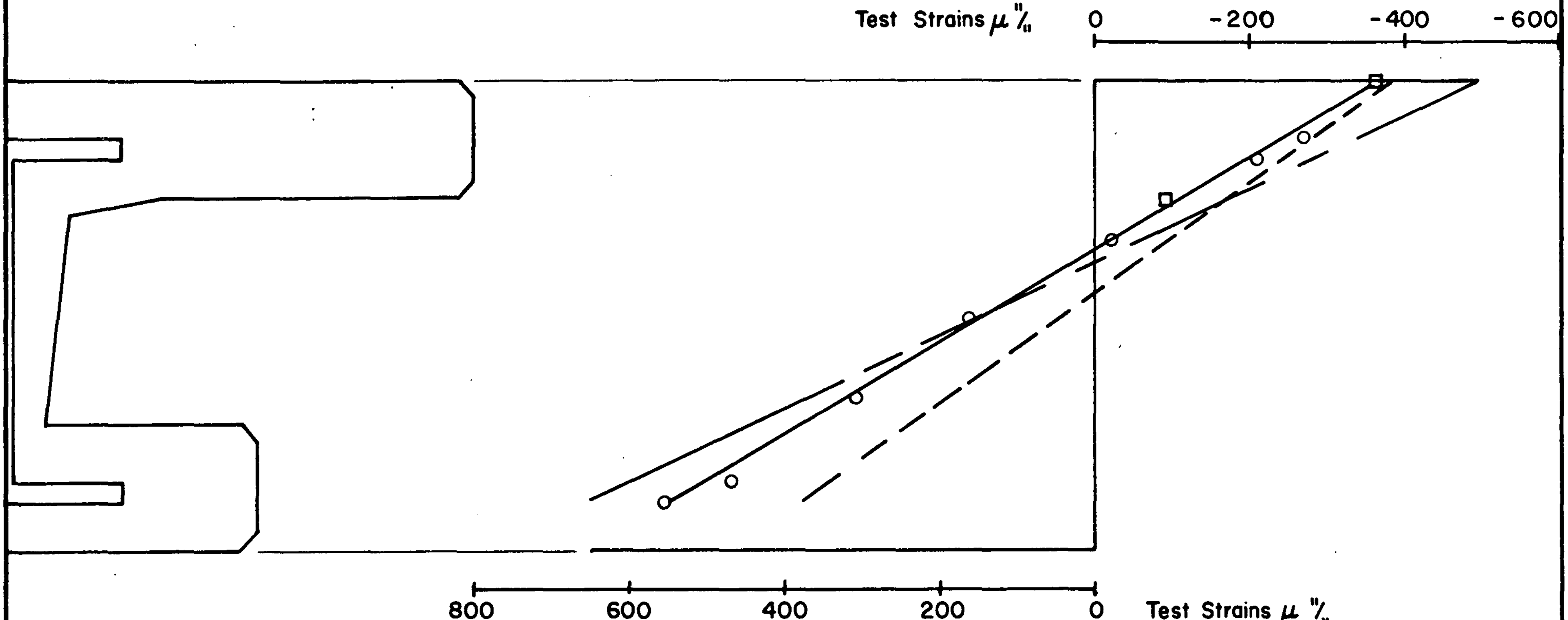
FIG. 39 Px-1, STRAIN DISTRIBUTION DUE TO FINAL TEST LOAD



Applied Load : 1.06 x Design Load
32.5 kips at each jack

o——o Average of Test Points
– – – Theoretical Line assuming uncracked section
—·— Theoretical Line for section cracked at 25 kips

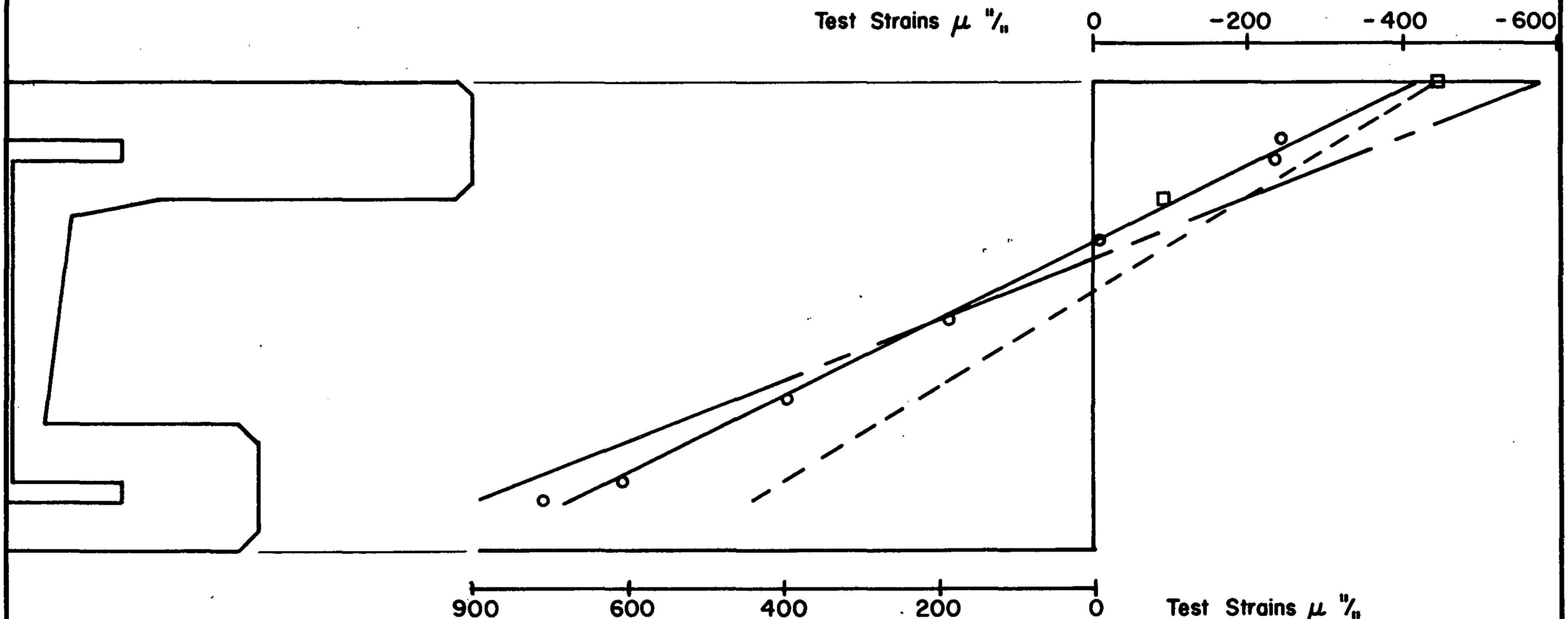
FIG. 40 Px-1, MEASURED AND THEORETICAL STRAINS AT STATION 1



Applied Load: 1.25 x Design Load
37.2 kips at each jack

o——o Average of Test Points
- - - - - Theoretical line assuming uncracked section
— — — Theoretical line for section cracked at 25 kips

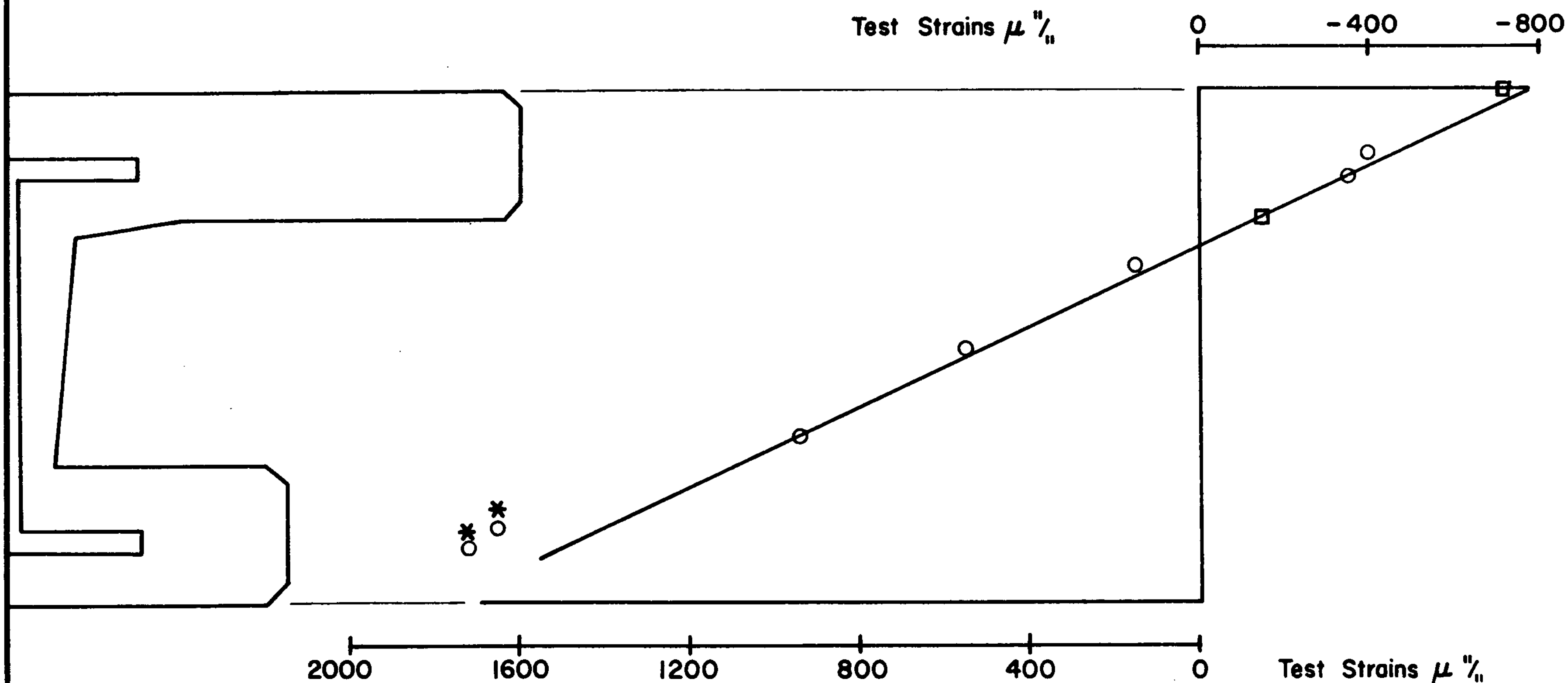
FIG. 41 Px-1, MEASURED AND THEORETICAL STRAINS AT STATION 1



Applied Load : 1.5 x Design Load
43.5 kips at each jack

o——o Average of Test Points
- - - - - Theoretical line assuming uncracked section
— — — Theoretical line for section cracked at 25 Kips

FIG.42 Px-1, MEASURED AND THEORETICAL STRAINS AT STATION 1



Applied Load : 2 x Design Load
56.0kips at each jack

* Denotes excessive yielding

Note: No comparison with theoretical value because of excessive yielding.

FIG. 43 Px-1, STRAINS AT STATION 1

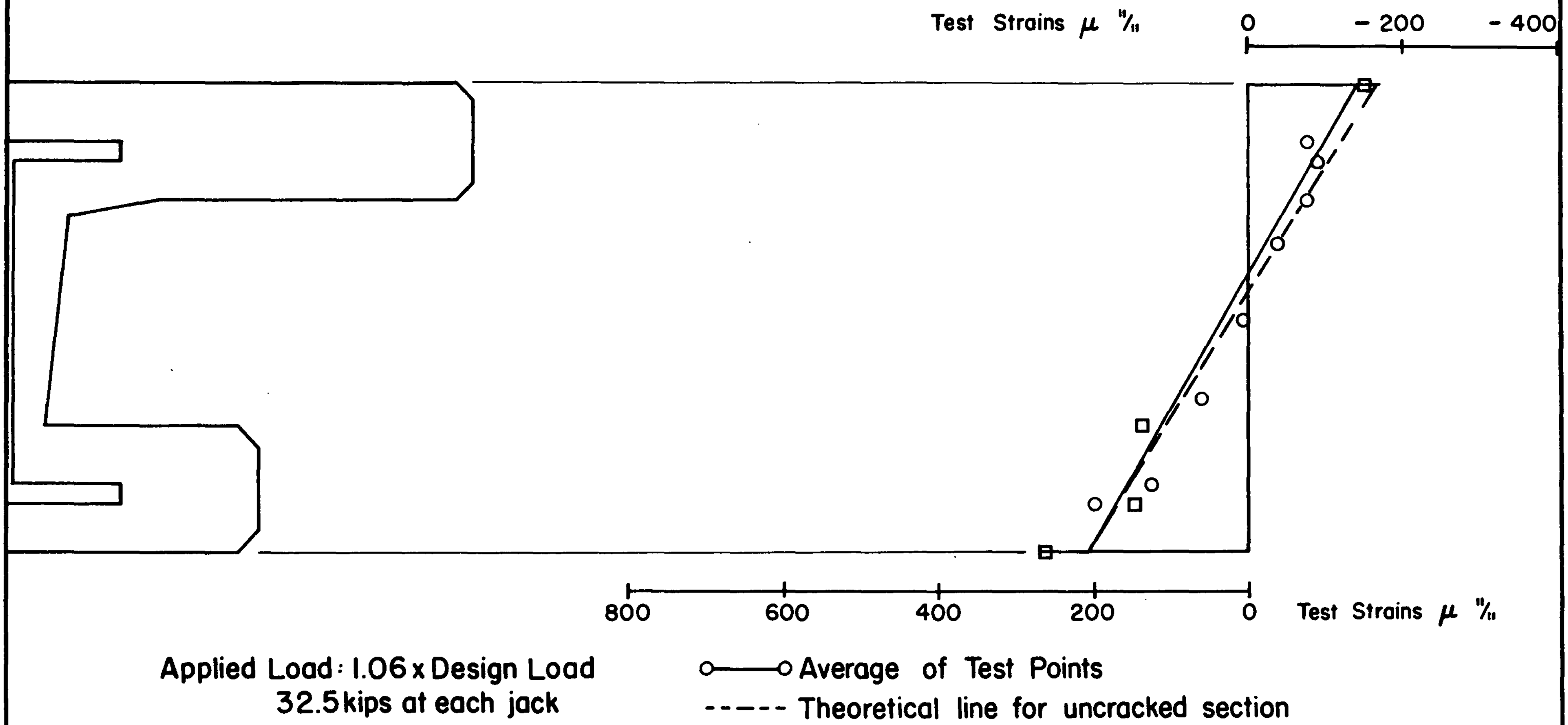
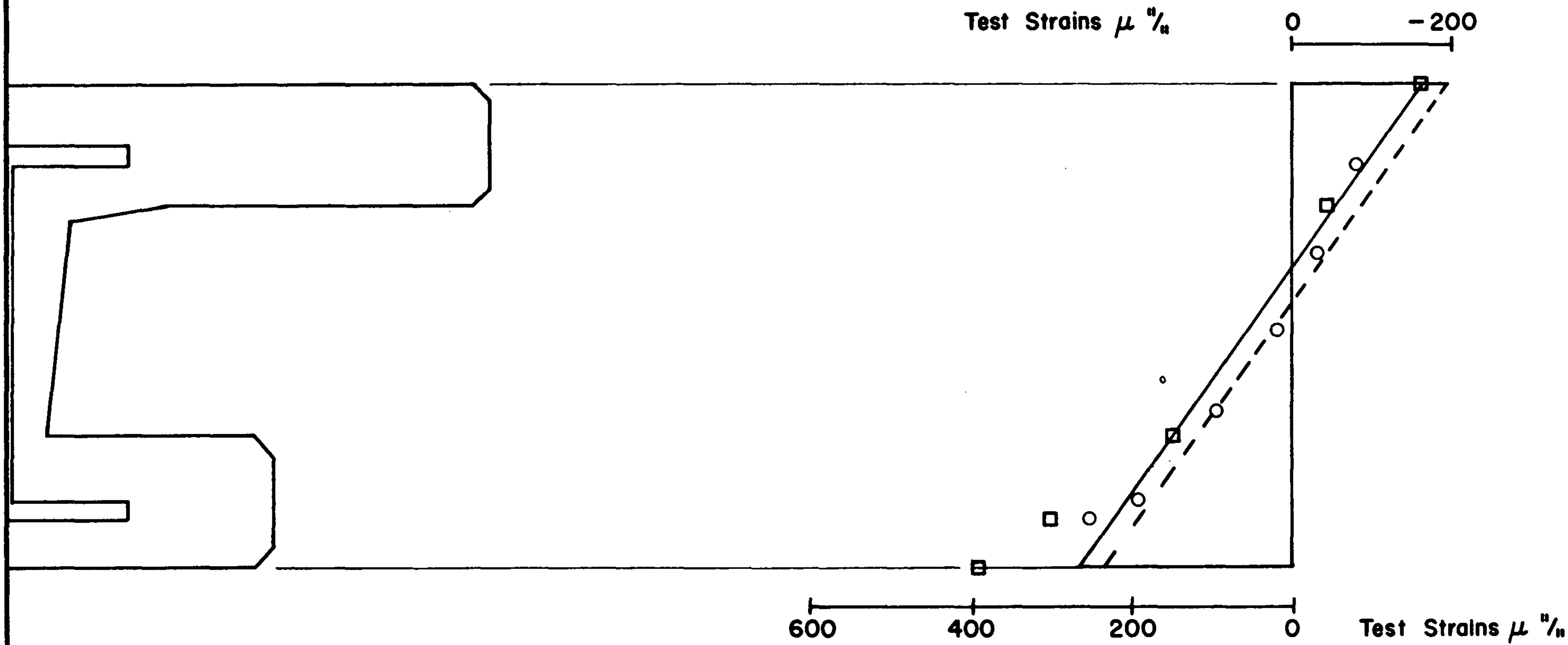


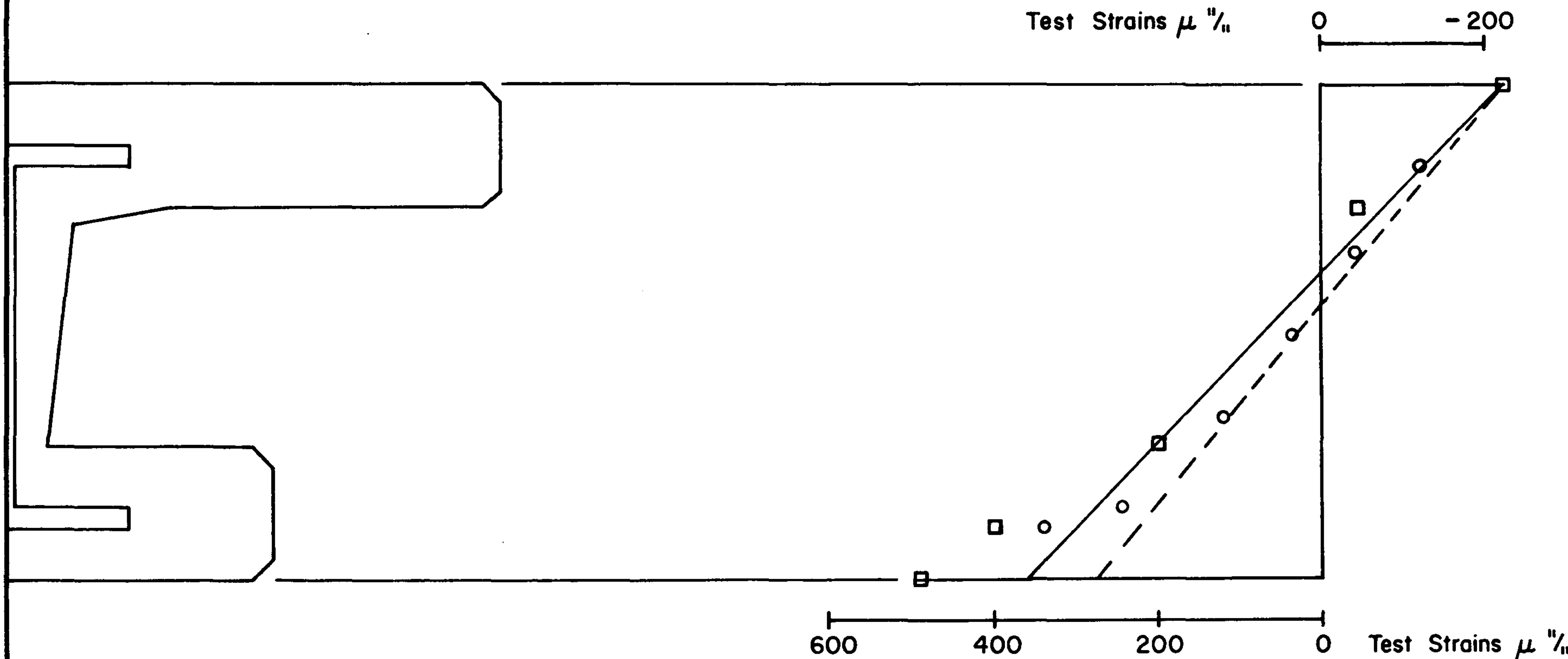
FIG. 44 Px-1, MEASURED AND THEORETICAL STRAINS AT STATION 3



Applied Load: 1.25 x Design Load
37.2 kips at each jack

o———o Average of Test Points
- - - - - Theoretical line for uncracked section

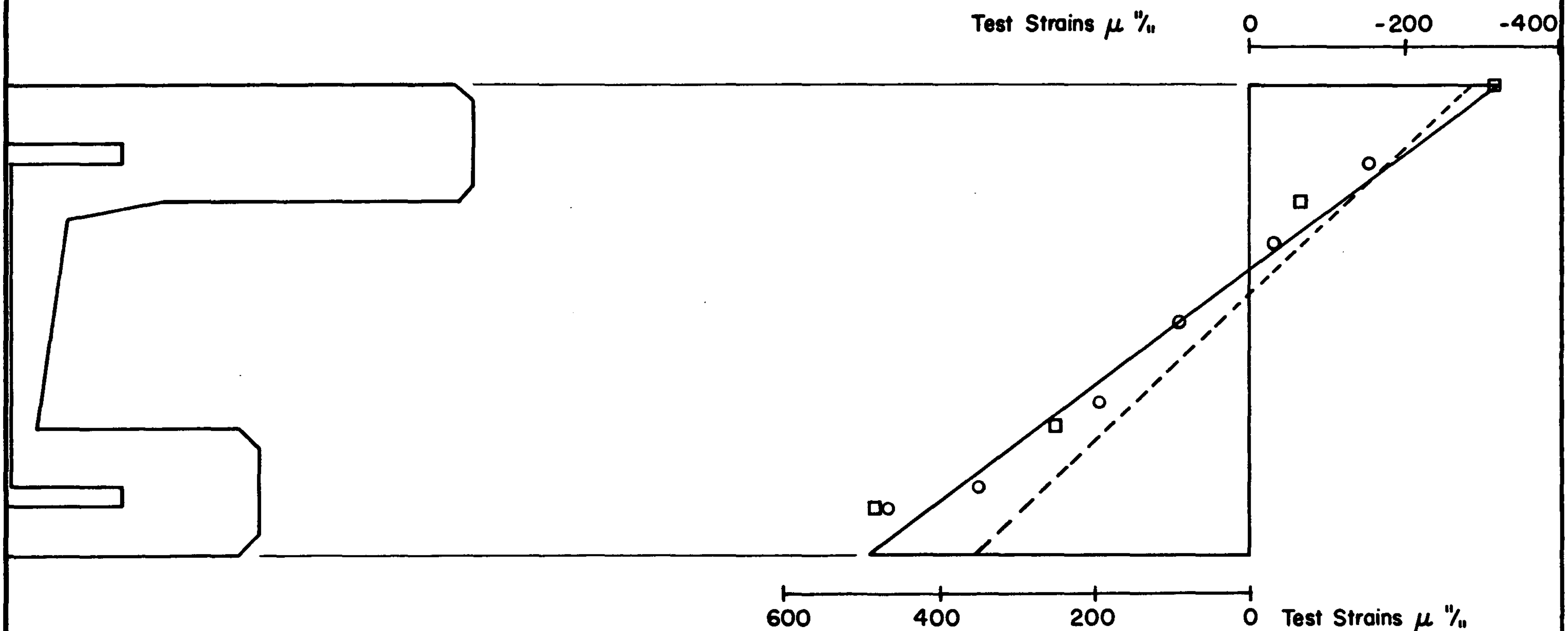
FIG. 45 Px-1, MEASURED AND THEORETICAL STRAINS AT STATION 3



Applied Load: 1.5 x Design Load
43.5 kips at each jack

o——o Average of Test Points
- - - - Theoretical line for uncracked section

FIG. 46 Px-1, MEASURED AND THEORETICAL STRAINS AT STATION 3



Applied Load : 2.0 x Design Load
56.0 kips at each jack

o——o Average of Test Values
- - - - Theoretical line for uncracked section

FIG. 47 Px-1, MEASURED AND THEORETICAL STRAINS AT STATION 3

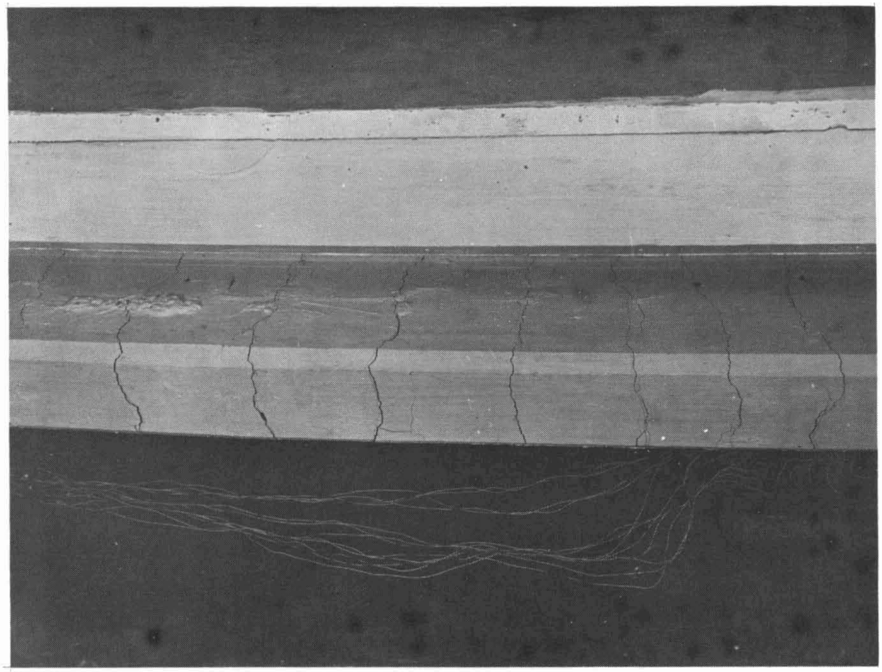


FIG. 48 CRACKING OF CONCRETE, Px-1 AT ULTIMATE LOAD

FIG. 49 CRACKING OF CONCRETE, Px-1 AT ULTIMATE LOAD

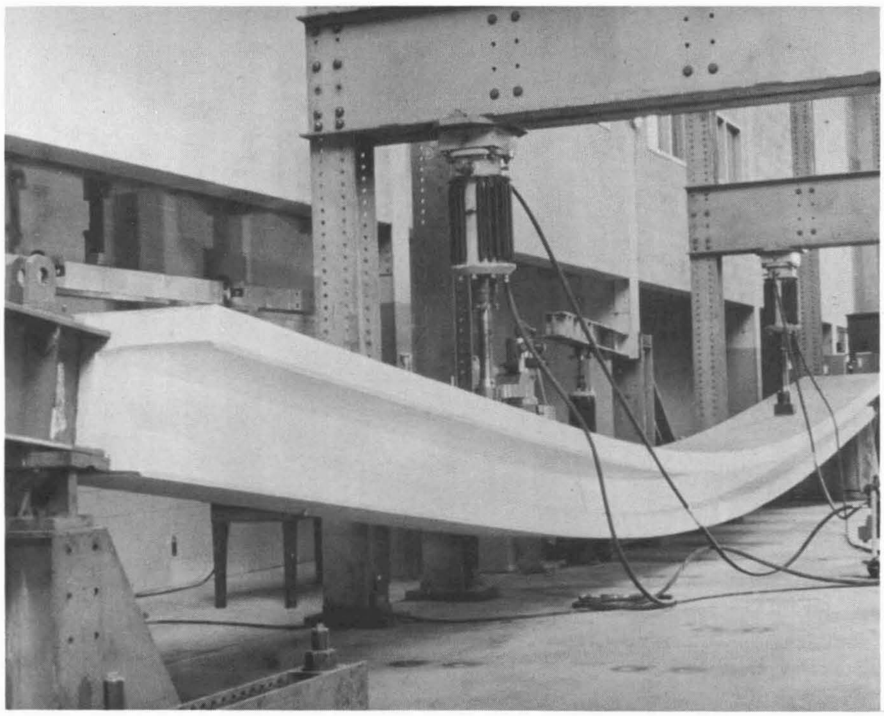


FIG. 50 Px-1 AT ULTIMATE LOAD

FIG. 51 BEAM STUB FOR PUSH-OUT SPECIMEN

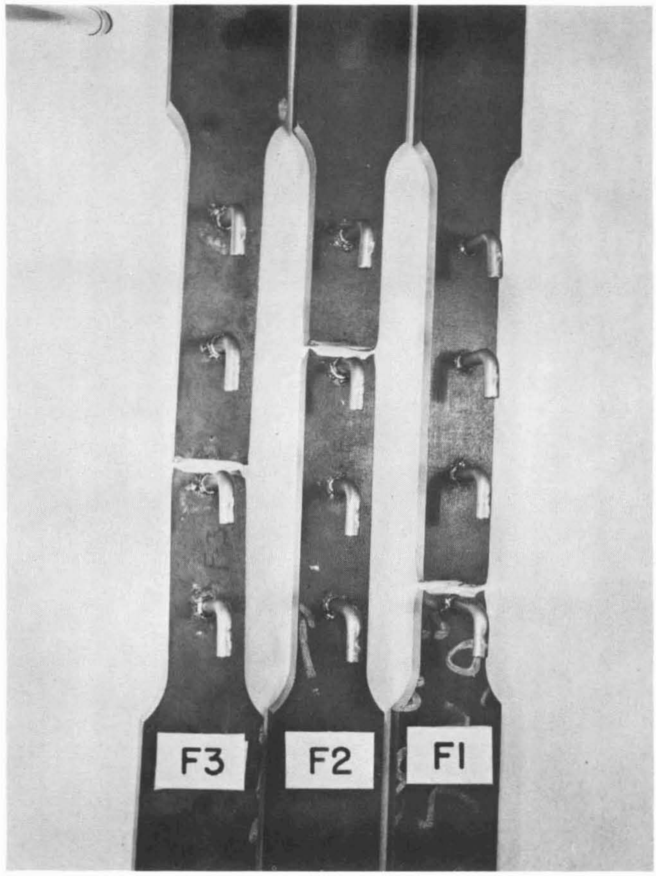
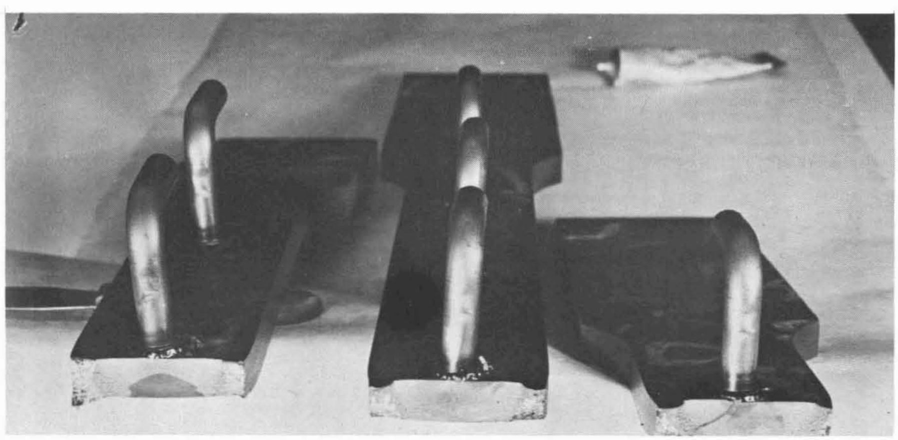


FIG. 52 FATIGUE SPECIMENS AFTER FAILURE



F3 F2 F1

FIG. 53 FAILURE SURFACES

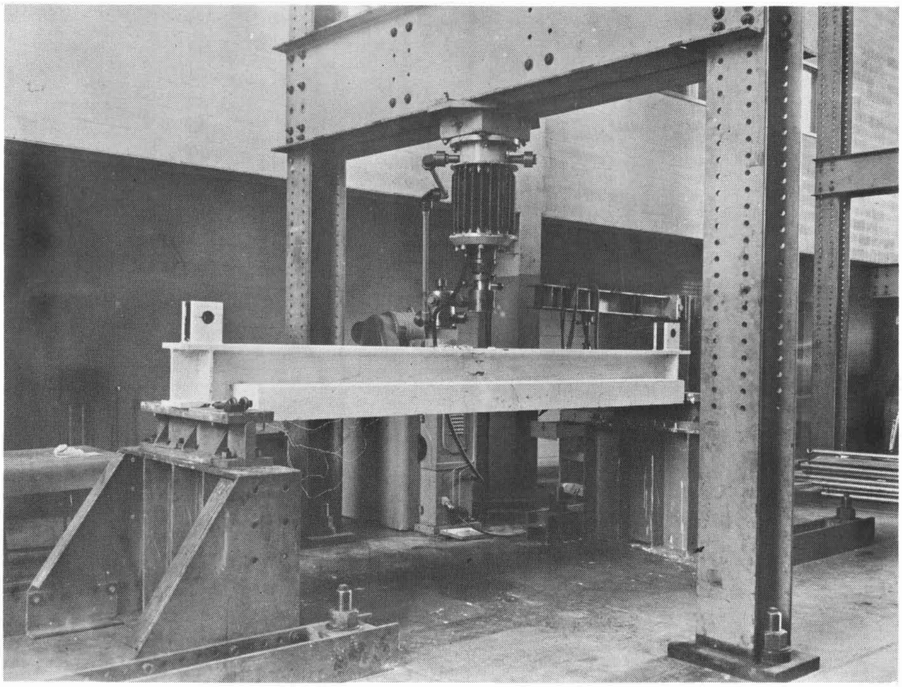


FIG. 54 FATIGUE BEAM B-I SET UP

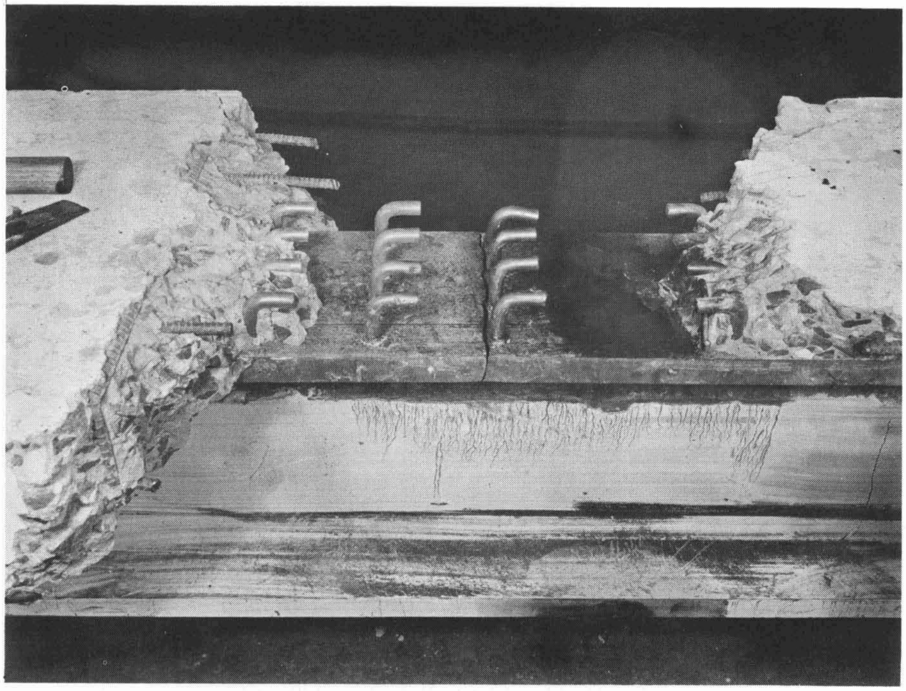


FIG. 55 FAILURE OF FATIGUE BEAM B-I

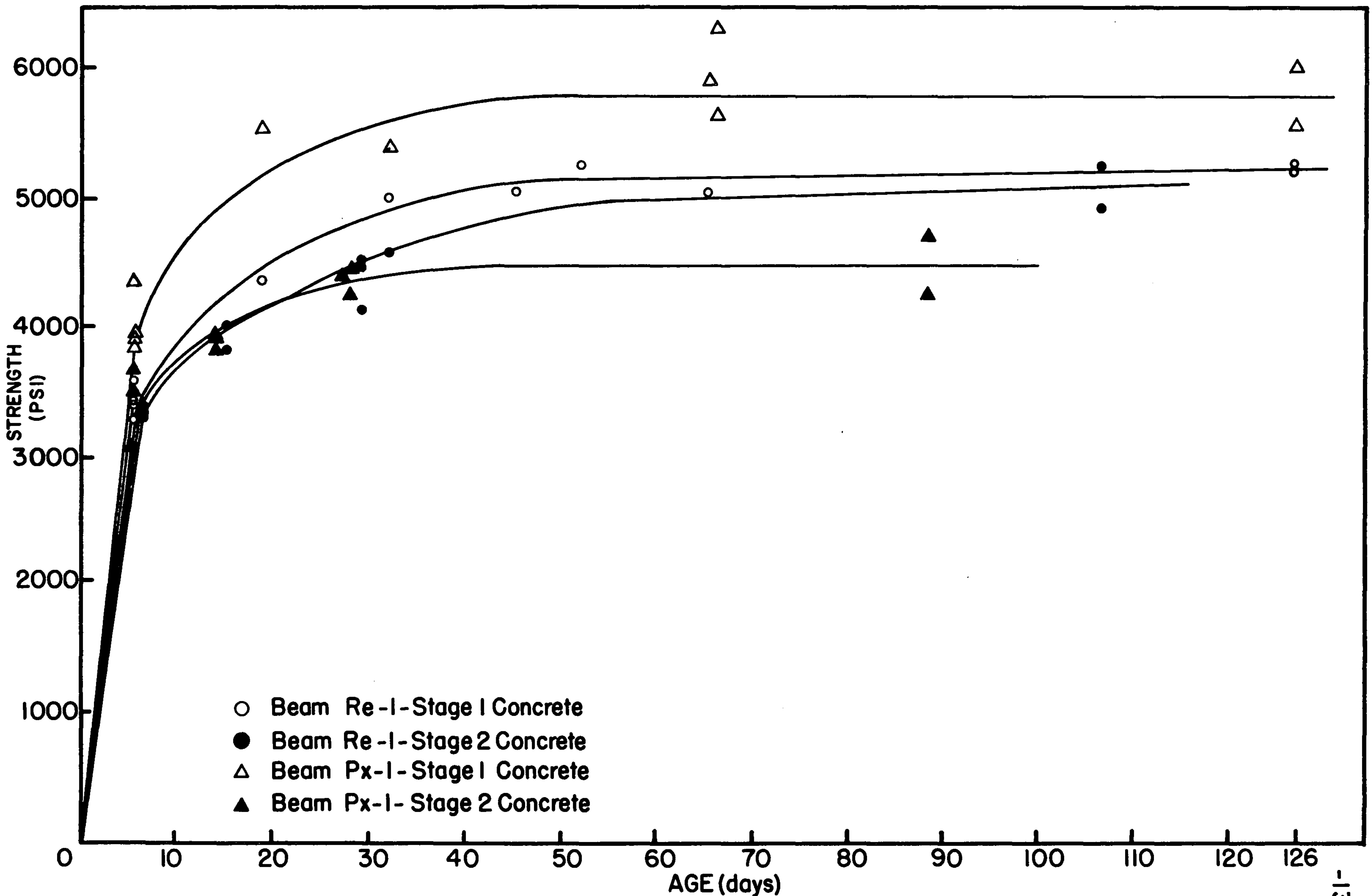


FIG. 56 STRENGTH VS. AGE OF CONCRETE CYLINDERS

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