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COMPOSITE DESIGN FOR BUILDINGS

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by

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ABSTRACT

At present there is no general specification for the design of composite beams for buildings. In order to establish such a specification the AISC is currently sponsoring a research program entitled "Investigation of Composite Design for Buildings,". The tests reported herein are part of this investigation.

The tests included in this report are of two types, beam tests and pushout tests. Six composite beams were tested in order to establish the carrying capacity and load-deflection characteristics of composite sections of the concrete-steel type. Six pushout specimens were tested in order to determine the strength and deformation characteristics of various types of shear connections.

Information concerning the behavior of composite beams and welded stud and channel shear connectors was obtained. Based on these findings recommendations for the design of composite beams for buildings are suggested.

TESTS OF COMPOSITE BEAMS FOR BUILDINGS

1. INTRODUCTION

The term "composite construction" denotes construction in which two materials are interconnected and act as an integral unit in resisting any imposed loading. Composite construction has found wide application in bridges and buildings where the floor slab or deck and the supporting members or beams are interconnected and act as a unit. This unit is referred to as a composite beam.

The elements of a composite beam consist of a slab, a supporting member, and some connection between the two. The slab is usually of concrete with the supporting member being either steel, concrete, or timber. Thus composite beams are referred to as concrete-steel, concrete-concrete, or concrete-timber depending on the supporting member used. The slab and beam act as a unit in resisting the load imposed on the composite section and the connection between the two resists the horizontal shear between and any tendency toward separation of the slab and beam.

Composite construction takes advantage of the strength of the concrete slab in the direction of the supporting member. This strength is neglected in non-composite construction. By connecting the slab and the beam, the slab acts in the same manner as a cover plate on a beam. The resultant composite section has a higher section modulus or stiffness and consequently smaller deflections than the same non-composite section. Also because of the combined action of slab and beam, smaller beam sections than those required in non-composite construction may be used to provide the same load carrying capacity.

At present, there is no general specification for the design of composite beams for buildings. There are, however, local building codes such as the New York City Building Code[1] which contain provisions for the application of composite design to building construction. In addition, there is a specification, AASHO Specification 1.9.5[2], which governs the design of composite beams for highway bridges.

In the absence of a directly applicable specification, composite design has nevertheless been used in building construction and has resulted in substantial savings in material.[3] The designer must, however, adapt the highway specification for the design of the composite section in the absence of an applicable local building code. Application of this highway specification to buildings will lead to a conservative design since this specification was formulated for bridges in which the loading is fatigue loading whereas in buildings the loading is primarily static loading.

Recognizing the advantages of composite construction and the lack of a specification directly applicable to buildings the American Institute of Steel Construction is presently sponsoring a research project entitled "Investigation of Composite Design for Buildings" aimed at establishing a specification for composite design in steel framed buildings. The tests reported herein are part of this research program.

The design of composite beams can be based on three different approaches as follows:

(a) Design of beams without, or with only a nominal amount of, shear devices. Here, interaction between steel beam and concrete slab is induced essentially by bond and friction forces. Design procedures can be devised either by considering composite action and limiting the horizontal shearing stress (see, e.g., ref. 4) or by disregarding composite action in the analysis but allowing an increase in allowable stress for the steel beams.

(b) Design of beams with shear devices on an allowable steel stress equal to 20 ksi. Under this condition the shear devices together with frictional forces should provide composite action up to initial yielding of the steel beam.

(c) Design of beams with shear devices on an ultimate load basis. The shear devices together with frictional forces must be able to transmit the horizontal shear up to the formation of a plastic hinge. Obviously this requirement will lead to more or heavier shear devices than procedure (b). However, assuming that a safety factor of 2 against ultimate load is appropriate, the working stress of the beam will increase to about 25 ksi. In other words, for a given loading the size of the beam can be considerably diminished. In addition, the procedure, being based on the ability of steel beams and connector devices to deform plastically, provides a rational basis to neglect such influences as shrinkage, creep and thermal stresses on the load carrying capacity. Furthermore, no distinction between the cases of shoring or no shoring needs to be made as the ultimate load carrying capacity is not influenced by it.

The ultimate moment capacity of a composite beam can be predicted with a very high degree of accuracy. This is so because no instability such as local or lateral buckling can occur. The strength is essentially governed by the yield stress of the steel section on which close quality control as to its minimum value is maintained. The variation of strength of concrete has a relatively minor influence.

To devise design procedures covering all three approaches, the study of the following four detail problems is required:

(1) Interaction created by bond and friction only. Behavior of beams after breaking of bond.

(2) Strength and deformation characteristics of shear devices.

(3) Influence of slip on the load-deflection curve of a composite beam. Limiting value of slip to be established if such a value exists.

(4) Distribution and spacing of shear devices along the beam.

This report describes a series of tests designed to answer problems 1 to 3 of the above-mentioned problems.* Subsequent tests have been designed to answer problem 4 and other problems brought out by the experimental investigations.

The tests in this report included six simple span isolated composite beam specimens and six pushout specimens. Following is a description of these specimens, the test results, and conclusions on which design recommendations are based.

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* Proposal dated November 4, 1959 to AISC Committee on Composite Design for Buildings.

2. GENERAL DESCRIPTION OF TEST SERIES

Since the investigation was aimed at establishing a specification for building construction the composite sections tested were of the type commonly encountered in building construction, i.e., a concrete slab connected to a wide flange structural shape. The dimensions of the specimens tested were of the same magnitude as those encountered in ordinary buildings.

Two beams, B1 and B2, were included to obtain data on the interaction created by bond and friction only. No shear connection was used for these beams, the slab being poured directly on the top flange of the beam. Any interaction observed in these beams under loading would be considered as due to bond between the slab and beam and/or friction developed between slab and beam under the points of load application. In order to evaluate the interaction caused by bond and friction separately, two methods of applying load to these specimens were used. For B1 the loads were hung from the steel beam so as to eliminate any localized friction under the points of load application. This localized friction was present in B2 under the load points because the load was applied to the top of the slab. By comparing the results of B1 and B2, interaction caused by bond and friction could be evaluated separately.

Beams B3 and B4 were included to obtain data on the strength of 1/2 inch diameter L shaped studs. The two types of loading described for B1 and B2 were also used for B3 and B4 to eliminate any localized friction effects under the load points and determine the true strength of the studs.

The shear connection in Beam B5 consisted of channel sections welded to the top flange of the steel beam. The object of this beam test was to evaluate the strength of this particular type of connector.

Beam 6 had exactly one half the number of connectors as B3 and was tested in the same manner. Thus the effect of an extremely weak shear connection could be evaluated.

By measuring slip between the slab and beam in each beam test, data on the influence of slip on the load-deflection curve of a composite beam could be obtained.

Six pushout specimens were included to establish the deformation characteristics of various types of shear connectors and to determine whether simple tests of this type could be used to predict the strength of the shear connectors in a composite beam. The types of shear connectors in the pushout specimens included channel sections, 1/2 inch diameter L studs, 1/2 inch diameter straight headed studs, and 3/4 inch diameter straight headed studs.

3. DESIGN AND FABRICATION OF TEST SPECIMENS

3.1 Beam Specimens

All the specimens provided with shear devices were designed on an ultimate basis. An ultimate strength design was necessary in order to evaluate the feasibility of design approach (c). Information concerning design approach (b) could also be obtained by analyzing the behavior of the specimens while the stresses in the steel section were still in the elastic range.

The ultimate moment of the composite section was determined by the internal couple method. This approach is similar to that used in ultimate strength design in concrete. In this method the stresses at a given cross section of the member are replaced by resultant compressive and tensile forces located at the centroids of the areas stressed in tension and compression respectively. The moment at the section is then equal to the product of either of these forces and the distance between them. The design procedure used for the shear connection considered equilibrium of the concrete slab as a free body between sections of zero moment and full plastic moment and is based on the assumption that the shear connectors possess sufficient ductility so that a redistribution of the horizontal shearing forces is possible. This same assumption is used in the design of a riveted or bolted connection. Analysis of previous tests (5) (6) established the validity of this assumption.

No design calculations were required for B1 and B2 in which the only shear connection provided was bond. Design values for the connector forces which would allow reaching the ultimate moment of the section prior to connector failure for beams B3 through B6 were obtained from a previous test.[5] A value of 16 kips per connector was used for the 1/2 inch L studs. (pg. 15 ref. 5) Design calculations are included in the Appendix.

Each specimen consisted of a 4' wide by 4" thick concrete slab connected to a 12WF27 steel beam. Slab reinforcement consisted of a 6" x 6" mesh of 1/4 inch diameter rods placed at mid depth of the slab. Bond, 1/2 inch diameter L shaped studs, and 4 inch lengths of channel sections (3 ⊔ 4.1) were used in the various beams for the shear connection. Fig. 1 gives the specimen dimensions and the connector spacings.

No special preparation of the top flange was used for beams B1 and B2 in which the only shear connection was bond. The beams, as received, had a considerable amount of rust on them. The loose rust was removed prior to pouring of the slab. In pouring the slabs for B1 and B2, tie rods spaced at intervals of approximately two feet along the entire length of the member were provided between the slab and beam to prevent breaking of the bond in handling of the specimens. (Fig. 1). These rods were burned free from the beams prior to testing.

The stud shear connectors were of the solid flux type attached to the steel beams by a conventional stud welding process. The channel sections on Beam B5 were attached by means of a single pass 3/16" fillet weld along the toe and heel of the channel.

The steel beams were from the same rolling and the concrete for the six specimens was from one mix. The concrete was a commercial ready mix type with a maximum aggregate size of 3/4 inch. By keeping the physical properties of the materials constant the only variable was the shear connection between slab and beam and comparison of the test results was facilitated.

3.2 Pushout Specimens

At the outset of the tests a pushout specimen with two slabs 20" x 28" x 4" thick connected with one row of shear connectors to each flange of an 8WF17 steel member was decided upon, (See Fig. 2). This specimen was chosen because it was felt that it could be used for testing any type of connector including new types which might be developed. Thus with a standard specimen, independent test results could be compared with those in this series of tests.

The pushout specimens were cast from the same mix as the beam specimens. The type and dimensions of the connectors used in each pushout specimen are given in Fig. 2.

The L connector material in P1 and P4 was the same as the material used in B3, B4 and B6. The 1/2 inch diameter headed studs in P5 and P6 were produced by machining from one inch diameter bar stock. The 3/4 inch diameter headed studs in P3 were produced by the usual heading process.

One row of connectors was attached to each flange of the steel member. There were two studs per row in specimens P1 and P3 through P6 and one 4" length of channel section per row in specimen P2.

All pushout specimens were cast in an inverted position from that of testing since it was felt that there was a possibility of voids forming in the concrete on the underside of the connectors. These voids probably would not affect the ultimate strength of the connectors, however, they would have an effect on the load slip characteristics of the specimens. An examination of the concrete around the connectors after testing showed that no voids formed on either side of the connectors. Hence, if pushout specimens are cast in a vertical position it is immaterial whether they are inverted or not assuming adequate vibration of the concrete around the connectors is accomplished.

4. TEST PROCEDURE

4.1 Beam Tests

The specimens were simply supported over a span of 15 feet and loaded with two point loads spaced symmetrically with respect to the center of the beam. Two types of load application were used in order to obtain data for evaluating the effect of friction under the loading points on the shear resistance. For beams B2, B3, B5, and B6 the load was applied to the top of the concrete slab as shown in Fig. 3 and is designated as loading type A in Table 1. For beams B1 and B4 the load was introduced directly into the steel beam by essentially hanging the loads from the steel beam and is designated as loading type B. The manner in which these hanging loads were produced can be seen in Fig. 4. Fabricated tee sections were connected to both sides of the web symmetrically with respect to the center line of the beam. Tie rods were pin connected to these tee sections and fixed to the test bed. Equal lifting loads were applied to both ends of the beams by the jacks shown in Fig. 4.

Load was applied to the specimens by means of hydraulic jacks. An Amsler pendulum dynamometer was used to apply and measure the pressure in the jacks.

In testing, the ultimate load at which crushing of the concrete slab will occur can be predicted quite accurately. By stopping the tests short of this load, the loading positions can be changed to produce greater shearing forces for the same ultimate moment - in other words by changing the spread distance "2b" of the two concentrated loads. (See sketch Table 1). Thus a single beam specimen can be used for several load tests and connector failure can be insured.

The beams were designed so that crushing of the concrete (M_p) and connector failure would occur simultaneously with a load spacing "2b" of 36 inches, as was used in the second test of each specimen. The first test of each specimen was conducted with a smaller load spacing "2b" of 18 inches which caused less severe shears and in which failure by crushing of the concrete was expected if the test were carried to completion. The load spacings in the third test of each specimen were such that connector failure would occur prior to reaching the ultimate moment.

Strains in the concrete slab were used to determine the point of which each of the first two tests should be stopped.

A previous test[5] indicated that crushing of the concrete occurred when the strains in the slab approached 0.003 in/in.. When the strains in the slab reached approximately 0.00275 in/in. in the current tests and the load approached the predicted ultimate load the test was stopped if the slip measurements did not indicate that connector failure was impending.

In summary, the first load spacing and test for each beam was made less severe in case the actual connector strength was less than assumed; the second load spacing and test was programmed for balanced flexure and shear strengths; and the third spacing was made severe enough so shear connector failure could be assured thus obtaining the shear value for the connectors in a beam.

The above procedure was followed for both load applied to the top of the slab and for the hanging loads. The load spacings were designated as follows:

Spacing S1 . . . . 2b = 18"

Spacing S2 . . . . 2b = 36"

Spacing S3 . . . . 2b = 56"

Spacing S4 . . . . 2b = 60"

Spacing S5 . . . . 2b = 76"

Since the material properties for all the specimens were the same, the value of the theoretical plastic moment

M_p and hence the applied maximum load P_p, for any given load spacing, assuming adequate shear connection, should be the same for all specimens except for B1 and B4 due to holes in the web. Thus the specimens are grouped according to the load spacing in Table VII.

The load was applied to the specimens in various increments up to approximately $P_p/1.85$* was applied 10 times to determine the cumulative effect of repetitive loading on the specimen. After 10 repetitions the load was again increased in increments up to the yield load. After exceeding the yield load a deflection criterion was used to determine load increments. These increments were chosen so that the increase in deflection produced by each load increment was equal to the measured deflection of the specimen at the yield load. If connector failure was not indicated as the load approached P_p, the load was released and the load spacing 2b increased. A second test was then conducted. This process was followed until connector failure occurred. In test B3-S2 a load equal to $P_p/1.85$ was applied to the specimen for a period of 63 hours. The purpose of this loading was to

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* A load value of $P_p/1.85$ was selected because this was expected to be in the order of magnitude of a working load for the beam. If a load factor or safety factor other than 1.85 were selected, the results could be expected to be of the same character though not numerically equal.

determine the effect of creep on the composite section. A summary of the type of loading used for each specimen and the various load spacings used is presented in Table 1.

The instrumentation used for the beam specimens was of two general types, those measurements aimed at determining the behavior of the specimen as a unit and those aimed at determining the behavior of the shear connection. The first type included strain measurements across the width of the slab and in the steel beam and centerline and quarter point deflections. They provided an indication of the behavior of the composite section as a beam. The second type included measurements of the slip, or relative horizontal displacement between the slab and beam, and the vertical separation between the two. These slip and uplift measurements were taken at various locations along the entire length of the member. The instrumentation and gage locations are shown in Fig. 5.

The measurements mentioned above were recorded at each increment of load application. After exceeding the yield load, the load was released at various intervals along the loading curve in order to determine residual deformations.

4.2 Pushout Tests

The test set up for the pushout specimens is shown in Fig. 6. A piece of 1/2" thick plywood was used as a base plate to protect the platen of the testing machine. A spherical seat was used under the crosshead of the machine at the top of the specimen. Load was applied to the steel section by means of a 300,000 lb hydraulic testing machine. The load was applied in small increments until the increase in slip between the slabs and the steel section became large. The specimen was then loaded so as to produce small increments of slip. The load was allowed to stabilize before any readings were taken. This fact if of importance since the speed of testing has a considerable influence on the strength of the specimen.

The slip between the slabs and the steel section was measured at four locations as shown by the dial gages in Fig. 2. The load was released periodically and residual slip measurements taken.

Auxiliary tests included concrete cylinder tests and tests of tensile coupons taken from both the web and flange of the steel section in order to determine the material properties of the composite section. In addition, Rockwell Hardness tests, tension and compression tests, and shear tests were performed on the shear connector material. The results of these auxiliary tests appear in Tables 2 through 6.

5. RESULTS AND DISCUSSION

5.1 Beam Tests

A summary of the results of the beam tests appears in Table 7 and the load-deflection curves are given in Figs. 7 to 18. For purposes of clarity of presentation the results of the tests for each beam are discussed separately. Following this, comparisons are made between the individual beams.

Beam B1 and Beam B2

Beam B1 and Beam B2 were fabricated without shear connectors. The only shear connection provided was the bond between steel and concrete. Tie rods spaced at intervals of approximately two feet along the entire length of the member were provided to prevent bond breakage during handling of the specimens. (See Fig. 1). Prior to testing it was noted that shrinkage had broken the bond from the end of the specimen to the first tie rod, approximately 12 inches on either end of the specimens. Before commencing the test, slip and uplift readings were taken when the specimens were placed on the test bed and after cutting of the tie rods. Small changes were observed in these readings indicating that shrinkage had destroyed the bond along the entire length of the members. As the specimens were loaded slip started immediately and increased progressively. This also indicated that the bond between the concrete and the steel had been destroyed.

The load versus centerline deflection of beam B1 is given in Fig. 7*. The last point plotted on this curve is 20 kips. Just prior to reaching this load the slab completely separated from the steel beam so that at this load of 20 kips there was no interaction. The test was stopped at this point because additional load would have been carried by the steel beam only. Since all the test points lie on a straight line, it is reasonable to conclude that there was no interaction from the beginning of the test. If there had been interaction initially the last test point would not lie on the same straight line since the load deflection characteristics would have changed after loss of interaction. Strain readings in the steel beam indicated that the loading produced a small torsional moment in the beam. This moment could have been induced either by errors in the test setup or shifting of the slab on top of the beam. This torsional moment probably accounts for the fact that the load deflection curve does not follow that predicted for the steel beam alone.

For Beam B2 the ultimate moment was about 7% higher than the predicted ultimate moment computed on the basis of no interaction, (See Fig. 8). Friction between the slab

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* The identifying symbol B1-S1 in the caption for Fig. 7 designates the beam number B1 and the load spacing number S1 as identified in Art. 4.1. A similar system of identification is used for all beam specimens in the Tables and Figures.

and beam can be developed by the weight of the slab and/or under the loading points. The weight of the slab was small compared to the superimposed loading and therefore the friction developed by the slab is negligible. The localized friction under the load points cannot increase the plastic moment since it is not present at the section of maximum moment. This case is analagous to a steel beam provided with cover plates in the vicinity of the load points. It is apparent that such cover plates would have no effect on the moment resistance at the center of the span.

The difference at ultimate between the theoretical curve and the curve of test results in Fig. 8 cannot be attributed to friction. Average values were used for the specimen dimensions and material properties. Small variations of these quantities for B2 could account for the higher ultimate moment. At any rate, the 7% difference is still within the limit of reasonable experimental variations.

Beams B3 and B4

The shear connection in Beams B3 and B4 was 1/2 inch diameter L shaped studs. For B3 the load was applied to the top of the concrete slab whereas in B4 the load was hung from the steel beam.

The shear connection in B3 was designed so that failure of the connectors and crushing of the concrete would occur at the same time under a load spacing of 2b = 36". Ultimate design values of 16 kips per connector used for connector forces were taken from previous tests[5]. The design of the shear connection in B4 was the same as in B3.

In testing B3 no slip occurred until reaching a load of 58.6 kips. At this load designated by the symbol Ⓢ in Fig. 9, a loud cracking sound was heard and slip followed. Since bond is inelastic and the shear connectors are elastic it is reasonable to conclude that bond was present and all the shear force was carried by bond up to this load of 58.6 kips. At this load the bond broke down and the shear force was transferred to the shear connectors with resulting slip along the entire length of the member due to the deformation of the connectors and the concrete.

The bond stress computed at this load of 58.6 kips was greater than 277 psi (bond stress at P_y = 47.1k). Because the section was not elastic at 58.6 kips an exact determination of the bond stress is not possible. This bond stress was quite high and in view of the results of B1 and B2 it is believed that the shear connection served to restrain the shrinkage and prevent bond breakdown. Thus bond cannot be counted on unless effective restraint against shrinkage is provided. Shear connectors will provide this restraint but will not carry any of the shear until the bond has failed.

Thus a design cannot be based on both bond and shear connectors since only one of them can be effective at any given time. If a design is based on bond for horizontal shear resistance, restraint must be provided to prevent bond breakdown due to shrinkage of the concrete. It is conceivable that over a long period of time these shrinkage forces may destroy the bond despite the fact that restraint in the form of shear connectors is provided. Therefore, it is recommended that bond be neglected in the design of composite beams.

Figure 9 shows that after unloading B3 to permit careful examination of the specimen it was able to carry increased load upon reloading with the same load spacing. The load-deflection curve is of the gradually ascending type indicating good plastic behavior even though a computed shear connector force Q = 13.5 kips per connector was sustained at the maximum load attained. Loading was halted at this point only because it was obvious that crushing of the concrete slab was impending without the shear connectors showing signs of failure.

In the test of the same beam with load spacing S2, the load-deflection curve shows the same elastic type behavior at lower loads even though a residual deflection δ_R of 1.22 in. was present at the start of loading. (Fig. 10). The beam was again able to carry load well into the plastic range with even larger connector forces of 14.5 kips per connector without signs of connector failure and would have reached the fully plastic load if continued far enough.

The effect of the long time loading can be seen on the load deflection curve for B3-S2, Fig. 10. No increase in slip was recorded over this 63 hour period of constant load. It is also significant to note that this creep did not affect the ultimate moment of the section. The curve in Fig. 10 is quite similar to the curve in Fig. 9 for B3-S1 where no creep loading was used.

Figure 11 shows the final loading test for Beam B3 in which the shear forces were so severe that connector failure was finally produced at a Q_F force of 15.3 k per connector. The beam was well into the plastic range though not as near to predicted ultimate as in the two prior loadings.

Comparable phases of the loading of Beam B4 with hanging loads are shown in Figs. 12, 13, and 14 covering load, deflection, and computed connector force.

In computing the ultimate moment of the section and the connector forces at the ultimate load for B4 the effect of the holes in the web was considered. These holes reduce the area of the steel beam and hence reduce the tensile force. Consequently, in Table 7 the predicted ultimate moment of B4 (M_p) is 2770 kip-in. The connector forces at failure in B4 based on this ultimate moment are 15.5 kips per connector. This value is very close to the value of 15.3 determined in B3 in which the load was applied to the top of the slab. These two tests indicate that friction developed under the loading points in a beam test is negligible.

Beam B5

Load-deflection curves for the three loadings of Beam B5 are given in Figs. 15, 16, and 17. The shear connectors in this specimen consisted of channel sections and the load was applied to the top of the slab. The measured slips were small up to about 40 kips. At this point a loud noise, the same as that heard in B3, occurred and slip began to increase at a faster rate.

Failure of this specimen was not due to failure of the shear connectors. As the maximum load was reached large slips resulted. Near ultimate the concrete slab cracked on the top surface around one of the connectors near the load point. This cracking was caused by the top flange of the channel connector pushing thru the top of slab as it deformed. Failure of the specimen occurred when it refused to carry more load. A sketch of the deformed shape of the connectors after failure of this specimen and removal of the slab is shown in Fig. 20.

The behavior of this specimen pointed out that the stresses in the concrete in the vicinity of the shear connectors are of importance. It does not seem advisable to use very large, strong shear connectors. In this case local failure of the concrete near the connector will reduce the ultimate carrying capacity of the member before the ultimate strength of the connector is achieved. Of primary importance are the bearing stresses in the concrete around the connectors.

<u>Beam B6</u>

Only one-half as many shear connectors as in B3 were used in this beam. No slip occurred up to about a load of 40 kips. At this load a cracking sound was heard and slip occurred. The final failure of this specimen was due to connector failure. The initial failure or the point at which the load dropped off, however, was not due to connector failure. The measured slips at this maximum load were all below the slips at failure recorded in B3 and B4. Also there was no visible indication of connector failure at this maximum load. The connector force listed in Table 7 for B6 is therefore not the connector force at failure of the connectors but that at the maximum load on the specimen.

The load deflection curve for this specimen shown in Fig. 18 did not approach the ultimate load and drop off prematurely due to connector failure. The load leveled off considerably below the ultimate load prior to connector failure and therefore the method of determining the connector forces at failure used in this report $\left[\frac{M_u}{M_p} \cdot Q_P\right]$ cannot be used. For this reason a separate calculation for the connector forces for B6 is included in part 5 of the Appendix.

5.2 General Results of Beam Tests

Several general observations were made in connection with the six beam tests. The strain measurements taken across the top of the slab at the centerline, Fig. 19, indicated that the full width of the slab was effective as acting with the steel beam.

The manner in which the stud connectors deform can be seen from Figs. 20 and 21. The marking on the top of the slab in Fig. 21 indicates the position of the load point at failure of the specimen. It will be noted that the connectors bend about their base and there is no bending or opening of the hooked portion. From this it can be seen that the hooked part of the stud will be effective in providing a restraint against uplift or separation of the slab up to failure.

In most of the beam tests a load approximately equal to $P_p/1.85$ was applied to the specimen 10 times. This is designated on the load deflection curves as 10xPk. It will be noted that these repetitions had no adverse effect on the specimens.

In B3, B4, B5, and B6 in which connectors were provided the slip behavior up to about 40 kips was the same. Very little, if any slip occurred up to or near this load at which time cracking noises were heard in the beam. After this occurrence slip started to increase. Based on this load of 40 kips the computed bond stress at failure of the bond was 236 psi. There was, however, a variation between the specimens as to how this bond failure occurred. In B3 it was a sudden failure whereas in the other specimens it occurred gradually. It is apparent that bond was present but only approximations can be made as to the magnitude.

Fig. 22 points out the fact that even at high loads the separation between the slabs and the beams was small. The separation occurring in each test is plotted separately as a bar graph at the location of the measurement. The residual uplift from a previous test is not included. In several cases when the load spread was increased the residual uplift decreased as load was applied to the specimen. This was esentially a "negative" uplift. In a virgin test such negative uplift would be impossible and therefore it was not plotted. This negative uplift is indicated by zero's in Fig. 22. The uplift was only partially recoverable, however. When the specimens were unloaded most of this uplift remained as permanent separation. Observation of the slip distribution along the members given in Fig. 23 to 26 indicates that the maximum slip occurred near the load point where the beam section was in the partially plastified state. This result is in agreement with theoretical studies. The slip occurring in each test is plotted separately. The residual slip from a previous test is not included.

In order to compare the load-deflection characteristics for all the tests the non-dimensional graphs of Figs. 27a, b, and c were plotted. The curves shown were drawn using the test results with no interpolation used to obtain smooth curves. The individual points were omitted from the drawing for the sake of clarity. It will be noted that all the curves in Fig. 27a except the one for B2-S1 and B6-S1 are straight lines up to $\frac{M}{M_y} = 1$. This indicates that these beams behaved as if there was complete interaction. For subsequent tests of the same beams in Figs. 27b and 27c the deflection at $\frac{M}{M_y} = 1$ did increase slightly. For instance, comparison of curves for B3-S1 and B3-S3 indicate a slight increase in deflection for B3-S3 at $\frac{M}{M_y} = 1$.

The curve labeled "X" is the curve of a previous composite beam test in which the shear connection was adequate and failure was due to crushing of the concrete. (Ref. 5, Fig. 5, Curve B1-T1). All the tests except B2-S1 and B6-S1 parallel this curve up to the point where they were stopped short of crushing of the concrete or the studs failed. Considerable slip was observed between slab and beam in all the tests as is evident from Figs. 23 through 26. Thus it appears that slip between slab and beam does not reduce the load carrying capacity of a composite beam. Also it does not appreciably alter the load deflection characteristics of the section.

Beam 6 had an extremely weak shear connection and Fig. 27a indicates that the load deflection characteristics were subsequently influenced. The deflection at yield of the section ($\frac{M}{M_y} = 1$) was increased above the theoretical deflection assuming complete interaction ($\delta/\delta_y = 1.5$ vs. $\delta/\delta_y = 1$). The curve does not parallel the curve for specimen "X" but levels off at approximately $M/M_y = 1.2$. Thus an extremely weak shear connection alters the behavior of a composite section.

Because there was no interaction in B1 this beam test was not included in Fig. 27a. Also the limited data obtained would only yield a small portion of such a nondimensional curve.

The connector forces determined in these tests at connector failure are in good agreement with those in another investigation (Ref. 5 - Table 2). There was a considerable difference, however, in the strength of the concrete slab in the two investigations. The average concrete strength in these tests was 3600 psi and in the other investigation was 5500 psi. This would indicate that the connector strength in a beam test is independent of the concrete strength.

5.3 Pushout Tests

A summary of the results of the pushout tests is given in Table 8 and the load slip curves for the various specimens appear in Fig. 28 through 33. The value of slip plotted as the abscissa in these graphs was the average of the two dials located on the slab in which connector failure occurred first. Values of the connector force at failure Q_F given in Table 8 were determined by dividing the maximum load P reached in the test by the total number of connectors in the specimen. The differences in readings of the four slip dials at any given load were small, thus justifying the assumption that each connector carried an equal portion of the total load on the specimen.

As specimens P1, P4, P5, and P6 were loaded to failure there was no cracking noted in either slab. There was, however, a slight separation between the top of the slab and the steel section in tests P1, P5, and P6. All the connectors in these four tests were 1/2 inch in diameter and the appearance of the area around the studs after failure was similar to that for P1 shown in Fig. 34. It is significant to note the crushed zone of concrete in front of the studs and the fact that there are two separate zones, one in front of each stud.

In specimen P2 a considerable amount of cracking of the slab occurred as the load on the specimen reached its maximum value. A large separation of slab and steel section occurred at the top of the slab. The specimen failed not by a shearing of the channel sections but by a pulling away of the slab from the steel beam. The channel connectors after failure are sketched in Fig. 20.

Considerable cracking of the slab also occurred in specimen P3. It will be noted in Fig. 35 that the crushed concrete in front of the connectors forms essentially one zone. A determination of the minimum transverse connector spacing and the minimum edge distance required can be based on the influence of the shear connector on the surrounding concrete by assuming that the shear connector transfers its load to the surrounding concrete in a manner similar to a load on a semi-infinite elastic solid. Theoretical studies along these lines and a comparison of the crushed zones of concrete around the connectors observed in these tests may lead to recommendations concerning the minimum transverse spacing and edge distance for shear connectors.

A comparison of P1 and P4 with P5 and P6 in Table 8 indicates that there was a difference between the 1/2" diameter L shaped stud and a 1/2" diameter headed stud, the 1/2" headed stud being about 12% stronger than the other. The material properties of the studs given in Tables 4, 5, and 6 do not substantiate this difference. The fact that the material in the L studs was harder than that in the headed studs is pointed out in Table 4. Tables 5 and 6 indicate that the L stud material was stronger in both tension and double shear than the headed stud material. From these differences in material properties the 1/2" L studs in P1 and P4 should have been stronger than the 1/2" headed studs in P5 and P6.

Further tests are in progress to determine the reasons for the difference between the L stud and the headed studs. In these tests the material properties of the studs will be kept constant.

A comparison of the results of P3 with those of P1, P4, and P5, P6 is of interest. It will be noted in Table 8 that the computed shear stress at failure on the 3/4 inch stud was below that of any of the1/2 inch studs. Considering P3 vs. P4 there is approximately a 10% decrease in the shear stress for P3 over P4. The results of the double shear tests on the stud material, given in Table 9, indicate that this difference is not due to the material properties of the studs. The 3/4 inch material was somewhat stronger in double shear than the 1/2 inch material. The pushout tests show that the ultimate connector force is not proportional to the shear area of the connector. It is believed that the bearing area of the connector is also of considerable importance. There may be a size effect involved in the behavior of studs embedded in concrete. Additional tests are planned to determine the behavior of 3/4 inch diameter studs in beam specimens.

5.4 Comparison of Beam Tests and Pushout Tests

In the past the results of pushout tests have been used to evaluate the deformation characteristics of shear connectors and establish values of connector forces to be used for design purposes.[9] In Table 9 an attempt was made to determine whether a correlation exists between beam and pushout tests.

It was observed in this series of tests that the maximum slips at failure recorded in the beam tests were different from those observed in the corresponding pushout tests. The ultimate connector forces were also considerably different between the two types of tests. This leads to the conclusion that the behavior of a shear connector in a pushout specimen is different from that in a beam specimen.

The results of these tests are too limited to establish the correct correlation between pushout tests and beam tests. However, it can be seen from Table 9 that the connector force in all the beams having an adequate number of shear connectors exceeded the connector force in the comparable pushout specimen by 39% or more.

6. CONCLUSIONS

The following conclusions are drawn from the test results:

1. Shrinkage forces will destroy the bond between the concrete slab and the steel beam. Some type of restraint is required to resist these shrinkage forces if bond is to remain intact. (B1 and B2)

2. The shear force and consequently the interaction created by friction is negligible (B2, B3, B4)

3. An insufficient number of shear connectors reduces the ultimate moment capacity of the composite section and also alters the deflection characteristics. (Fig. 18)

4. The 1/2 inch L shaped studs provided sufficient tie down action so that uplift or separation of slab and beam was small prior to failure of the connectors. (B3, B4, B6 max. uplift - 0.04 in.).

5. An estimate of the bond strength achieved in these tests was 236 psi. (B3, B4, B5, B6).

6. Creep or long time loading has no noticeable effect on the ultimate carrying capacity of a member.

7. The strength of a shear connector obtained in a pushout specimen is different from that in a beam specimen. In beams with adequate shear connectors the connector force at failure was at least 39% greater than the connector force in a comparable pushout specimen.

8. Shear connector failure does not occur suddenly. Considerable deformation of the connector, and consequently large slips result prior to connector failure, if a ductile connector is used.

9. The connectors used in the beam tests, 1/2 inch diameter L shaped studs, permit a redistribution of the horizontal shear forces. The ultimate strength of these

connectors based on the results of the beam tests is approximately 15 kips per connector. (Table 7)

10. Considerable slip between slab and beam of a composite section does not alter the load-deflection characteristics significantly nor substantially reduce the ultimate carrying capacity of the composite beam.

7. DESIGN RECOMMENDATIONS

It was stated in Art. 1 that the design of composite beams can be based on three possible approaches. The results of B1 and B2 have indicated that a rational design in which interaction is induced by bond between the slab and steel beam is not feasible since some restraint is required to prevent bond failure due to shrinkage. The results of B6 have shown that a weak shear connection alters the behavior of the composite section and prevents achieving the ultimate moment of the section. Further study is required before any design recommendations can be made for beams with weak shear connections.

Two alternative design procedures exist for beams with full composite action namely, an elastic approach or a design based on the ultimate load capacity of the section. The economies and advantages of plastic design in steel are well known. If applied to composite beams, plastic design is also advantageous and economical.

In building design the depth of the steel members is such that the neutral axis is usually in the slab at the ultimate load. In this case the steel beam is completely yielded in tension as opposed to an elastic design in which only the extreme fibers of the section reach design stress. In addition the problems of buckling encountered in plastic design in steel are not present in composite beams. Because the top flange of the steel beam is anchored to the slab by the shear connectors lateral buckling is prevented. The problem of local buckling is also eliminated since the entire steel beam is in tension at ultimate.

The results of B3, B4, and B5 have demonstrated that plastic design of composite beams is feasible. Theoretical analysis and comparison with previous tests [5] have shown that excess deflections do not result from such a design approach. It is possible to predict the ultimate load of a composite section quite accurately. A design of the shear connectors by the method used in this report using the recommended values for connector forces will assure that this ultimate load can be reached.

In view of the above considerations it is recommended that the design of composite beams be based on the ultimate carrying capacity of the section. Further tests are presently in progress to determine whether this analysis can be applied to continuous beams and thus result in plastic design of continuous composite beams.

The design recommendations which follow are based on the results of the tests in this report. These recommendations stem directly from the conclusions listed previously.

The composite section should be designed considering equilibrium of the internal forces at the section of maximum moment in the simple span. The working load multiplied by a load factor of 1.85 should be used for the design load.

The location of the neutral axis at ultimate is determined by assuming a linear distribution of strain through the depth of the composite section. A stress distribution is then computed from the assumed strain distribution and an equilibrium check of the internal forces is then made. If equilibrium is satisfied the assumed strain distribution is correct, if not another trial is made until the internal forces at the section are in equilibrium. The following strains should be used as limiting values

$$\varepsilon_{st} \text{ (steel)} = 15 \times 10^{-3} \text{ in/in}$$

$$\varepsilon_{yield} \text{ (steel)} = 1.1 \times 10^{-3} \text{ in/in}$$

$$\varepsilon_{ult.} \text{ (conc.)} = 3 \times 10^{-3} \text{ in/in}$$

and the yield stress for the steel taken as 33 ksi. Present criteria[2] should be used for determing the effective width of the concrete slab. The ultimate moment of the section can then be computed from the stress distribution at the critical section.

The shear connection used should consist of a ductile type connector so that a redistribution of shear forces is possible at ultimate. The design of these shear connectors should consider equilibrium of the slab from the section of ultimate moment to the section of zero moment. The number of connectors is then chosen assuming that each carries an equal share of the compressive force in the slab. Recommendations concerning the distribution and spacing of these connectors along the length of the member will be made in another report. A nominal amount of connectors should be provided over sections of constant moment to provide a tie between slab and beam.

The following value should be used for the ultimate connector force for 1/2 inch diameter L shaped studs:

$$Q_p = 15 \text{ kips/connector}$$

If the composite section is designed on the basis of an allowable maximum stress (elastic design) it would be in order to modify the allowable stresses presently used in elastic design in steel. The shear connection would then be designed by the conventional method $\left(\frac{VQ}{I}\right)$ with allowable connector forces used which would be compatible with the allowable steel stresses.

The present AISC Building Code permits a working or allowable stress of 20,000 psi in the steel member for simple beams when an elastic design is followed. Assuming a yield stress of 33,000 psi for A7 steel and an average shape factor of 1.12 for wide flange shapes, an elastic design of a simple beam results in a reserve capacity or factor of safety against ultimate (neglecting strain hardening) of 1.85. This corresponds to the load factor presently used in plastic design in steel. Many years of practice have shown that this procedure leads to safe designs. It would therefore seem reasonable that a similar approach would be possible for composite beams.

The shape factor or ratio of yield moment to ultimate moment for a composite section is greater than that for a wide flange shape. For the sections tested, which had the same proportions as those encountered in building construction, the ratio of $\frac{M_p}{M_y}$ was 1.5. Since a portion of a composite beam consists of a concrete slab it may be argued that the close quality control that is present in the manufacture of steel wide flange shapes is not always present with concrete. Thus a value of reserve strength against ultimate higher than 1.85 might be reasonable. Assuming that an increase of this value of 1.85 to 2.0 were used the allowable design stresses in the steel section could be increased to 25,000 psi. The reserve strength of the composite beam would then be of the same magnitude as that for a steel beam.

In order to provide essentially a balanced design the shear connectors would therefore be designed with the same reserve strength of 2.0 or $\frac{15}{2.0} = 7.5$ kips per connector. It is therefore recommended that if an elastic design is used for the composite section the allowable stresses in the steel section be increased to 25,000 psi and a value of 7.5 kips per connector be used for an allowable connector force for 1/2 inch diameter L shaped studs. These studs would then be spaced in accordance with the shear diagram by using the formula $\frac{VQ}{I}$ to determine the shear flow.

Concrete stresses in accordance with the present ACI building code should be used in such an elastic design.

8. ACKNOWLEDGEMENTS

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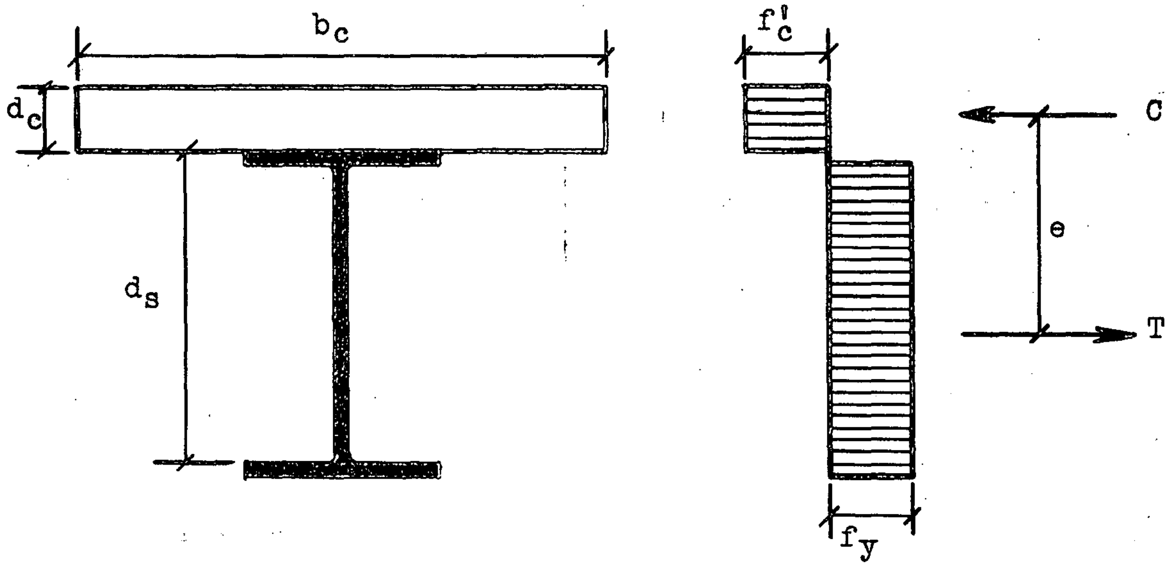
9. NOMENCLATURE



A_s = steel area

A_{web} = area of web of steel beam

A_{flg} = area of one flange of steel beam

a_{st} = distance from neutral axis of composite section to extreme fiber of steel in tension

b = distance from center line of beam to point of load

b_c = effective width of concrete slab

C = total compressive force = $f'_c b_c d_p$

d_c = depth of concrete slab

d_p = depth of compressive stress block at M_p

d_s = depth of steel section

e = distance between resultant compression and tension forces at M_p

f'_c = cylinder strength of concrete at 28 days

f_y = yield stress of steel beam

$f_{y(flg)}$ = yield stress of flange of steel beam

$f_{y(web)}$ = yield stress of web of steel beam

I = moment of inertia of composite section, concrete transformed to equivalent steel area

I_s = moment of inertia of steel section

L_s = shear span = distance between sections at which plastic moment and zero moment occur

m = statical moment of transformed compressive concrete area about the neutral axis of the composite section

M_p = theoretical plastic moment of composite section

M_u = experimentally observed ultimate moment

M_y = theoretical yield moment

n = $\dfrac{E_{steel}}{E_{concrete}}$

P = externally applied load

P_p = externally applied load at M_p

Q = connector force

Q_F = connector force at failure of connectors

s = connector spacing along longitudinal axis of beam

$\circledS$ = load at which slip first occurred

T = total tensile force = $f_y \cdot A_s$

δ = deflection of beam in inches

δ_r = residual deflection of beam in inches

10. APPENDIX

10.1 Section Properties

A. Beam Specimens

a. Concrete Slab

$b_c = 48$ in.

$d_c = 4$ in.

$f'_c = 3600$ psi*

reinforcement - mesh 6"x6"x1/4" placed at mid depth of slab

* A total of 18 cylinders were tested at various ages. The value of f'_c used in the calculations was an average of all the cylinders tested.

b. Steel Beam (12WF27)

All the steel beams were from the same rolling. The cross sectional dimensions and the weight of each specimen were recorded. Measured values were all very close to the handbook properties so the handbook dimensions were used in the calculations.

$A_s = 7.97 \text{ in}^2$

$d_s = 11.95$ in.

$I_s = 204.1 \text{ in}^4$

f_y** = 39.0 ksi (flange)
44.0 ksi (web)

** Coupons were taken from both the web and flange of the steel beams. The respective static yield stresses were used in computing the T force as shown on page

c. Connectors

(1) Studs - B3, B4, B6

diameter = 1/2 in.

height = 2.25 in.

area = 0.196 in.^{2}

(2) Channels (3 ⊔ 4.1)-B5

height - 3 in.

width - 4 in.

weld - 3/16 in. fillet across toe and heel

d. Composite Section

$n = 10$

$a_{st} = 11.60$ in.

$I = 587.7$ in^4

m = (innerface of slab) = 45.1 in^3

B. <u>Pushout Specimens</u>

a. Concrete Slab

28" x 20" x 4" - See Fig. 2

f'_c = 3600 psi (same as for beam specimens)

reinforcement - mesh 6" x 6" x 1/4" placed

1" from outer face of slab

b. Steel Section - 8WF31

c. Connectors

P1, P4 - 1/2" dia L shaped studs

P2 - Channel section (3⊔4.1)

P3 - 3/4" dia headed studs

P5, P6 - 1/2" dia headed studs

10.2 Specimen Design

The slab thickness for the beam specimens was set at 4" because this is the slab thickness usually used in floor slabs in buildings.

The slab width of 4' feet satisfies one of the two criterion for the effective width of T-beams.(2)

Values of f'_c = 3000 psi, f_y = 38 ksi, and connector forces of 16 kips/ connector were assumed and used to determine the number and spacing of the connectors. The procedure used considers equilibrium of the concrete slab as a free body between sections of zero moment and full plastic moment. The design calculations are not included but they were essentially the same as those which follow under Art. 10.3C except a value for Q was assumed and values of s or connector spacings determined. In Art. 10.3C the material properties used (f'_c, f_y) were those obtained from coupon and cylinder tests.

10.3 Predicted Quantities

A. Calculation of Yield Moment

$$\sigma = \frac{Mc}{I} = \frac{Ma_{st}}{I}$$

$$M_y = \frac{f_y I}{c}$$

where:

$f_y = 39$ ksi

$c = 11.60$ in.

$I = 587.7 \text{ in}^4$

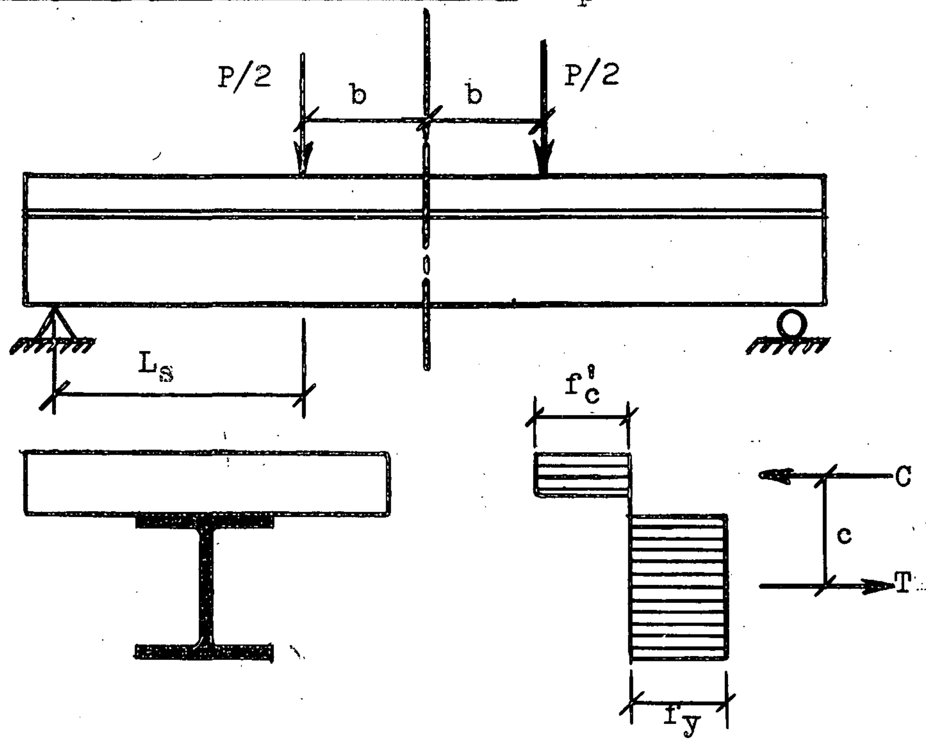
$M_y = 1975$ k-in.

The specimens were constructed in such a manner that the dead load of the slab was carried by the formwork. Therefore, the dead load of the slab was carried by the composite section after removal of the forms. In testing the moment to be applied to the specimen to produce yielding in the steel section should therefore be:

$$\begin{aligned} M &= M_y - M_{D.L.(slab)} \\ &= 1975 - 68 \\ &= 1907 \text{ k-in.} \end{aligned}$$

This value of 1907 k-in. will be referred to us the yield moment of the section.

B. Calculation of the Plastic Moment (M_p)



The proportions of the composite are such that the neutral axis is located in the slab at ultimate. The steel section is completely yielded in tension and the concrete is assumed to have no tensile resistance. The internal couple method is used in computing the plastic moment.

The total tensile force T developed by the steel section is:

$$T = f_{y(flg)} \cdot A_{flg} + f_{y(web)} \cdot A_{web}$$
$$= 39 \cdot 5.29 + 44 \cdot 2.68$$
$$= 324 \text{ k}$$

For longitudinal equilibrium, a compressive force equal in magnitude to this tensile force in the steel is required. It is assumed that this compressive force is provided by an area of concrete fully stressed to the cylinder strength f'_c. The depth of penetration of this compressive area into the slab is:

$$d_p = \frac{T}{b_c f'_c}$$

$$= \frac{324}{48 \cdot 3.6}$$

$$= 1.87 \text{ in.}$$

The moment arm between the tensile and compressive forces is:

$$e = \frac{d_s}{2} + d_c - \frac{d_p}{2}$$

$$= 5.98 + 4 - \frac{1.87}{2}$$

$$= 9.05 \text{ in.}$$

The plastic moment of the composite section is the moment produced by this couple of tensile and compressive forces:

$$M_p = Te = Ce$$

$$= 324 \cdot 9.05$$

$$= 2930 \text{ k-in.}$$

The plastic moment which can be reached in testing will be reduced by the dead load moment of the slab.

$$M_p = 2930 - 68$$
$$= 2862 \text{ k-in.}$$

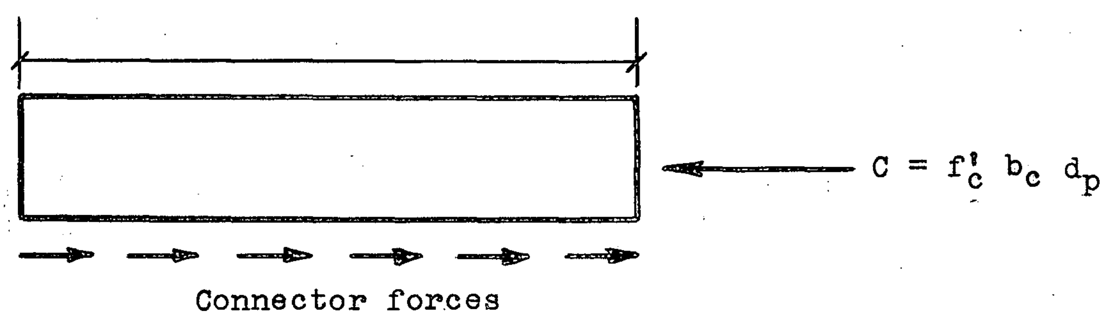
This value of 2862 k-in. will be referred to as the plastic moment of the section.

In beam B4 several holes were drilled in the web of the steel section to provide for the loading fixtures. The area of the web was therefore reduced and the tensile force which the section could develop was reduced. Using this new tensile force and following the same procedure as above the values for B4 are:

$$M_y = 1832 \text{ k-in.}$$
$$M_p = 2770 \text{ k-in.}$$

C. Calculation of Connector Forces

The connector forces are computed by taking a free body of the slab between the section of full plastic moment and zero moment. (length = L_s)



By assuming that the connector forces are equal over the length L_s the connector forces are computed by dividing the C force by the total number of connectors in the length L_s.

The shear stress in the connector is computed by dividing the connector force by the shear area of a connector.

The above procedure leads to the following results:

<u>Example</u>

B3 - S1

$C = 324$ k

no. of connectors over length L_s equals 22

$$Q_p = \frac{324}{22}$$

$$= 14.7 \text{ k}$$

Beam	S1 (2b = 18")		S2 (2b = 36")		S3, 4, or 5	
	Force per connector Q_p (kips)	τ (ksi)	Force per connector Q_p (kips)	τ (ksi)	Force per connector Q_p (kips)	τ (ksi)
B3	14.7	75.0	16.2	82.6	18.0 (2b=56)	91.9
B4	13.7	69.9	15.1	77.1	16.8 (2b=60)	85.4
B5	64.9	-	81.0	-	108 (2b=76)	-
B6	29.4	150.0	-	-	-	-

10.4 Deflection Calculations (Theoretical)

1. Due to Bending

$$\delta_B = \frac{Pa}{24EI}\ (3L^2-4a^2)$$

where

$L = 15'-00"$

$E = 30 \times 10^3$ ksi

$I = 587.7\ in^4$

$a = L/2 - b$

2. Due to Shear

$$\delta_S = \frac{\tau a}{G} = \frac{Pa}{2A_wG}$$

where

$A_w = 2.68\ in^2$ (web area of steel beam)

3. Total Deflection

$$\delta = \delta_B + \delta_S$$

		2b=18"	2b=36"	2b=56"
Load (P)		40	40	40
Deflection due to Bending δ_B (in.)		0.272	0.260	0.040
Deflection due to Shear δ_S (in.)		0.052	0.046	0.040
Total Deflection $\delta_B + \delta_S$	in.	0.324	0.306	0.280

10.5 Analysis of Test Results

A. Calculation of Q_F

The values for the connector forces (Q_F or Q_u) at failure in the beam specimens were computed by multiplying the connector forces at the plastic moment by the ratio of the ultimate moment reached in testing to the calculated plastic moment $\frac{M_u}{M_p} Q_p = Q_F$.

Example

B3-S1

$M_u = 2638$ k-in.

$M_p = 2862$ k-in.

$Q_p = 14.7^k$

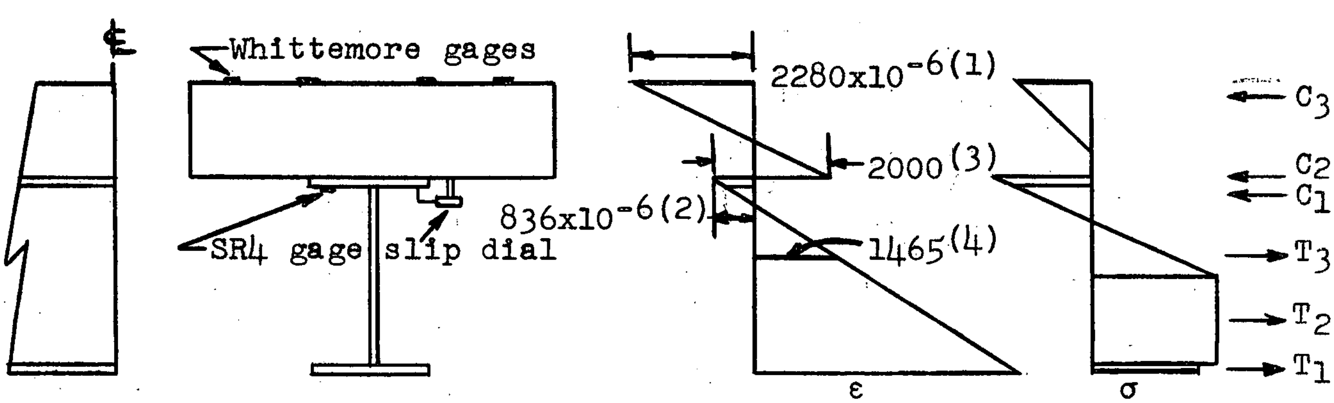
$$Q_F = \frac{2638}{2862} \cdot 14.7$$

$Q_F = 13.5^k$

These connector forces are listed in Table 7 under the column "Connector Force".

B. Calculation of Maximum Connector Force B6

The connector forces are computed by utilizing the measured strain and slip readings. The computed values must be considered approximate since a straight line was assumed. This is obviously an assumption for the cracked concrete slab. (Continued on next page)



(1) Measured strain in concrete - Whittemore gage

(2) Measured strain in steel beam - SR4 gage

(3) Measured slip - Ames dial

(4) Computed

Note:

a) SR4 gage on bottom flange can't be used since bottom flange has yielded.

b) Whittemore readings on under side of slab can't be used since concrete slab is cracked.

The several compressive and tensile forces in the Figure above are computed by converting the average strains to stress and then multiplying by the appropriate areas.

$$T_1 = \sigma_{y\ flg.} A_{flg} = 39 \cdot 2.64 = 103^k$$

$$T_2 = \sigma_{y\ web}\ A_2 = 44 \cdot (0.24)\ (8.12) = 85.7^k$$

$$T_3 = \varepsilon_{ave.}\ E\ A_3 = 732 \times 10^{-6} \cdot 30 \times 10^{-3} \cdot (0.24)\ (1.93) = 10.2^k$$

$$C_1 = \varepsilon_{ave.}\ E\ A = 418 \times 10^{-6} \cdot 30 \times 10^{-3} \cdot (0.24)\ (1.1) = 3.31^k$$

$$C_2 = \varepsilon_{ave.}\ E\ A_{flg.} = 988 \times 10^{-6} \cdot 30 \times 10^{3} \cdot 2.64 = 78.3^k$$

The final compressive force of the concrete slab is obtained by equilibrium of the section.

$$C_3 = T_1 + T_2 + T_3 - C_1 + C_2 = 117.3^k$$

The moment computed from the measured strains is compared with the moment from the beam test.

Computed moment using T and C forces = 2227 k-in.

Actual Moment $(\frac{P_u}{2} 81)$ = 2340 k-in.

% error = 5%

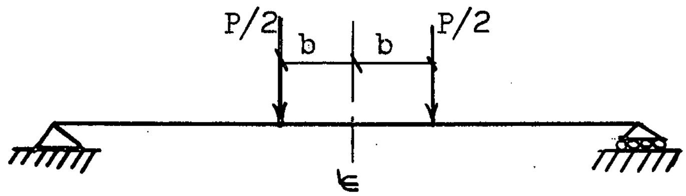
$$Q = \frac{C}{\text{no. of connectors}} \quad \text{where } C = C_3 \text{ above}$$

$$Q_u = \frac{117.3}{11} = 10.7^k \pm 5\%$$

Since the moment computed differed from the measured moment by 5%, the approximate connector force may also be in error.

TABLE 1

Designation of Beam Specimens



Specimen	Connector Type	Connector Spacing c (in.)	Type of Loading	Test No.	Load Spacing 2b (in.)	Test Designation
B1	Bond	-	(B)	1	18	B1-S1
B2	Bond	-	(A)	1	18	B2-S1
B3	1/2" dia.L studs	2at7.5	(A)	1	18	B3-S1
			(A)	2	36	B3-S2
			(A)	3	56	B3-S3
B4	1/2" dia.L studs	2at7.5	(B)	1	18	B4-S1
			(B)	2	36	B4-S2
			(B)	3	60	B4-S4
B5	channel section (3⊔4.1)	section 4" wide at 20"	(A)	1	18	B5-S1
			(A)	2	36	B5-S2
			(A)	3	76	B5-S5
B6	1/2" dia.L studs	1at7.5	(A)	1	18	B6-S1

(A) Top loading - load applied to top of slab

(B) Hanging loads - load applied to steel beam

TABLE 2

Cylinder Strengths of Concrete in Beam Slabs and Pushout Specimens

Cylinder No.	Age at Test (days)	Strength (psi)
1		3654
2		3636
3	31	3326
4		3539
5		3490
6		3618
	Ave	3544
7	33	3583
8	35	3715
9	40	3565
10	46	3512
11	46	3627
12	49	3539
13	53	3610
14	53	3689
15	53	3795
16	61	3777
17	95	3547
18	95	3592
	Average of all cylinders	3600 psi

TABLE 3

Static Yield Strength of Material in 12WF27

Coupon No.	Material	Location of Coupon	Static Yield Stress (ksi)	Ultimate Stress (ksi)	Modulus of Elasticity E (ksi)
1		↑	39.25	-	29.5×10^3
2			38.70	68.30	29.7
3	ASTM A-7		39.00	67.50	30.0
4	Structural	Flange	38.10	66.10	30.2
5			38.40	67.50	31.1
6			39.40	68.10	30.5
7		↓ Ave.	38.95		
8		Web	44.50	69.60	-
9	ASTM A-7	Web	45.00	69.60	30.4
10	Structural	Web	43.20	65.90	30.4
		Ave.	44.23		

Average values used in calculations

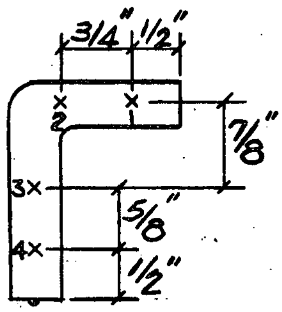
f_y = 39.0 ksi (flange)

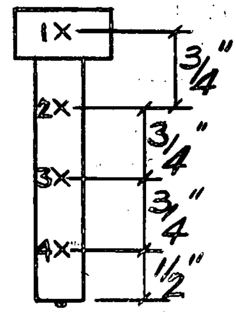
f_y = 44.0 ksi (web)

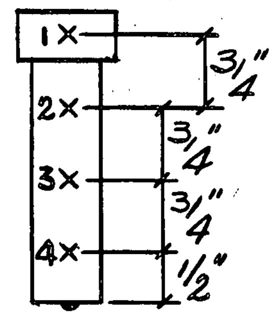
TABLE 4

Rockwell Hardness Tests of Stud Material

Location of Readings







Type of Stud	Point 1	Point 2	Point 3	Point 4	Average
(1)-1/2" dia. L shaped stud	B84 B86	B85 B84	B77 B82	B87 B84	B83.6
(2)-1/2" dia. headed stud	B84 B83	B76 B69	B78 B76	B76 B78	B75.5*
(3)-3/4" dia. headed	B92 B97	B80 B87	B87 B85	B85 B87	B87.5

Note: At each point one reading on either side of the stud was taken.

* Average of points 2, 3, 4 only

TABLE 5

Coupon Tests of Connector Material

Specimen	Connector Material	Type of Coupon	Yield stress (ksi)	Ultimate Strength (ksi)	Modulus of Elasticity E (ksi)
1	1/2" dia. L studs	Compression	57.1	-	$30.0x10^3$
2	"	Compression	56.5	-	30.0
3	1/2" dia. headed studs	Compression	52.0	-	28.4
4	3/4" dia. headed studs	Compression	73.0	-	32.4
5	"	Compression	70.6	-	31.5
6	1/2" dia. L stud	Tension	56.0	66.1	28.6
7	"	Tension	57.0	67.4	30.3
8	Channel	Tensile coupon cut from weld of channel parallel to direction of rolling	36.7	61.0	28.8
9	Channel	"	37.3	61.5	31.0
10	Channel	Tensile coupon cut from web of channel perpendicular to direction of rolling	42.9	64.4	-
11	Channel	"	43.5	64.5	-
12	Channel	"	55.0	64.1	-

Note: Specimens 1-5, 8-12, yield stress determined by 0.2% offset method.
Specimens 6, 7 yield stress equal static yield stress.

TABLE 6

Double Shear Tests of Connector Material

Specimen No.	Material*	Stud Type	Ultimate Shear Load (lbs)	Ultimate shear stress (psi)
1	C1010-C1017	1/2" L	17,550	44,700
2	"	1/2" L	17,650	45,100
3	-	1/2" headed	16,200	41,400
4	-	1/2" headed	15,700	40,000
5	C1015-C1017	3/4" headed	42,700	48,500
6	"	3/4" headed	42,100	47,700

* Material designations are those of the American Iron and Steel Institute.

The specified properties of the stud material are as follows:

1/2" L

Tensile strength - 72,000 psi min.

Yield strength - 61,000 psi min.

Elongation - 20% (2" gage length)

3/4" headed

Tensile strength - 65,000 psi min.

Table 7

Summary of Beam Test Results

Specimen	Test	Load Spacing 2b (in.)	Failure Type	C_L Moment M (k-in.) M_p	M_u	Connector Force Q (kips)	Max. End Slip at P_u (in.)	Residual End Slip (in.)
B1	B1-S1	18	(A)	1560 (1)	-	-	-	-
B2	B2-S1	18	(A)	1560 (1)	1678	-	0.335	0.223
B3	B3-S1	18	(B)	2862	2632	13.5	0.040	0.030
B4	B4-S1	18	(B)	2770 (2)	2495	12.5	0.015	0.004
B5	B5-S1	18	(B)	2862	2619	59.3	0.029	0.022
B6	B6-S1	18	(C)	2862	2340	10.7±5%	0.120	-
B3	B3-S2	36	(B)	2862	2560	14.5	0.077	0.059
B4	B4-S2	36	(B)	2770 (2)	2470	13.6	0.020	0.017
B5	B5-S2	36	(B)	2862	2682	75.9	0.046	0.074
B3	B3-S3	56	(C)	2862	2438	15.3	0.092	0.170*
B4	B4-S4	60	(C)	2770 (2)	2538	15.5	0.126	0.189*
B5	B5-S5	76	(D)	2862	2342	88.5	0.207	-

Failure Type: (A) Test stopped

(B) Test stopped short of crushing of slab

(C) Shearing of studs

(D) Failure to carry additional load

Note: (1) Assuming no interaction

(2) Effect of holes in the web considered

* After connector failure

TABLE 8

Summary of Pushout Test Results

Specimen	Connector Type	Ultimate Connector Force Q_F (kips)	Shear Stress* (ksi)	Type of Failure	Remarks
P1	1/2" dia. L shaped studs	11.0	56.1	Shearing of studs	No cracks in slab
P2	Channel section (3⊔4.1) 4" length	47.5	--	Separation of slabs and beam stub	Large cracks in slab
P3	3/4" dia. headed stud	21.2	48.1	Shearing of studs	Large cracks in slab
P4	1/2" dia. L shaped studs	10.4	53.0	Shearing of studs	No cracks in slab
P5	1/2" dia. headed stud	12.1	61.7	Shearing of studs	No cracks in slab
P6	1/2" dia. headed stud	12.1	61.7	Shearing of studs	No cracks in slab

* Computed on the basis of a uniform distribution of shear stress on the cross section of the connector

TABLE 9

Comparison of Beam Tests and Pushout Tests

Specimen	Connector Force Q_F (kips)	Manner of Failure	$\frac{Q_{Beam}}{Q_{Pushout}}$
B3	B3 15.3	B3 - shearing of studs	
	P1 11.0	P1 - shearing of studs	Q_{B3}/Q_{P1} = 1.39
	P4 10.4	P4 - shearing of studs	Q_{B3}/Q_{P4} = 1.47
B4	B4 15.9	B4 - shearing of studs	
	P1 11.0	P1 - shearing of studs	Q_{B4}/Q_{P1} = 1.45
	P4 10.4	P4 - shearing of studs	Q_{B4}/Q_{P4} = 1.53
B5	B5 88.5	B5 - concrete failure	
	P2 47.5	P5 - concrete failure	Q_{B5}/Q_{P2} = 1.86
B6	B6 10.7	B6 - shearing of studs	
	P1 11.0	P1 - shearing of studs	Q_{B6}/Q_{P1} = 0.974
	P4 10.4	P4 - shearing of studs	Q_{B6}/Q_{P4} = 1.03

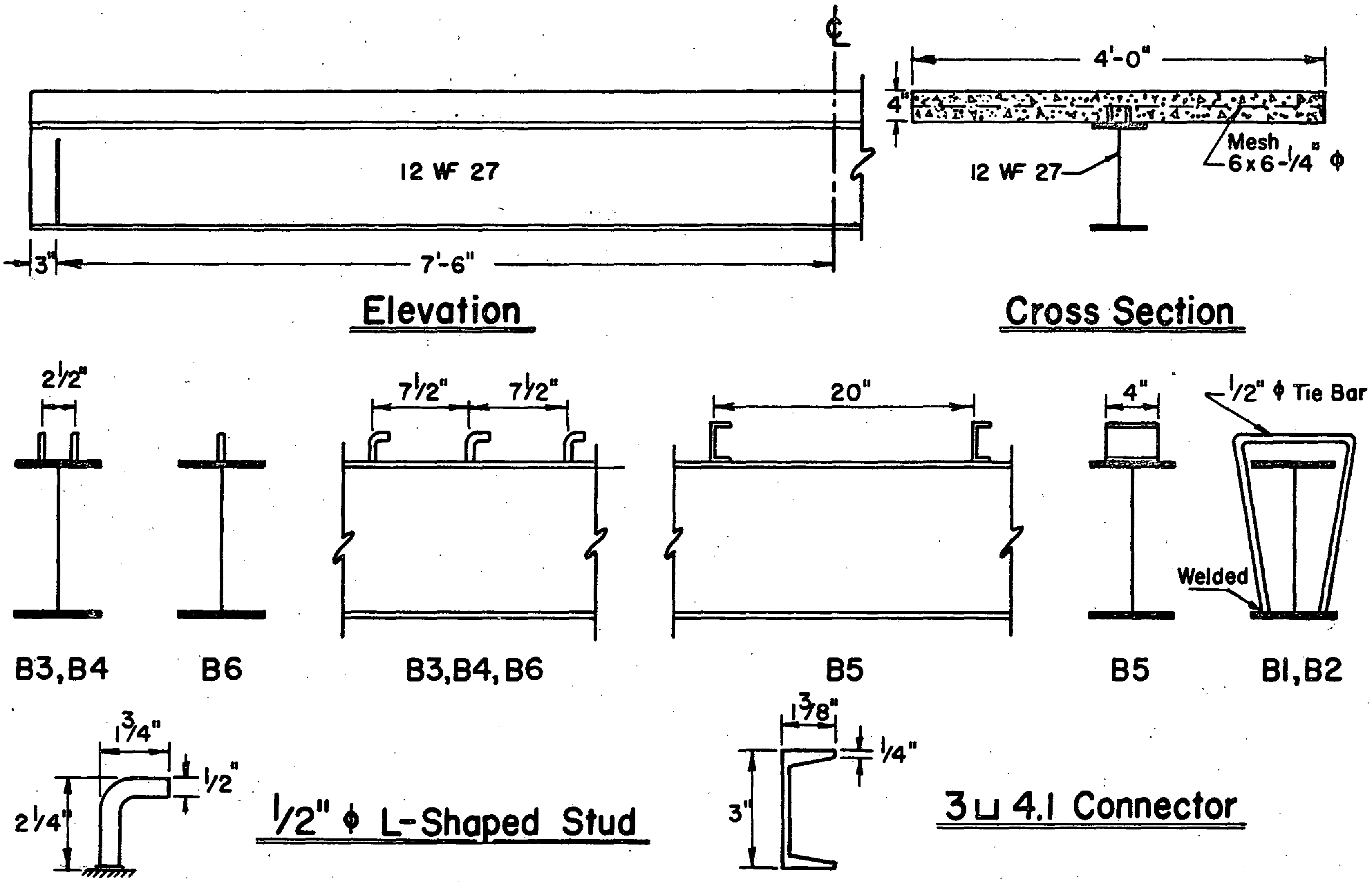


Fig. 1 - Dimensions of Beam Specimens

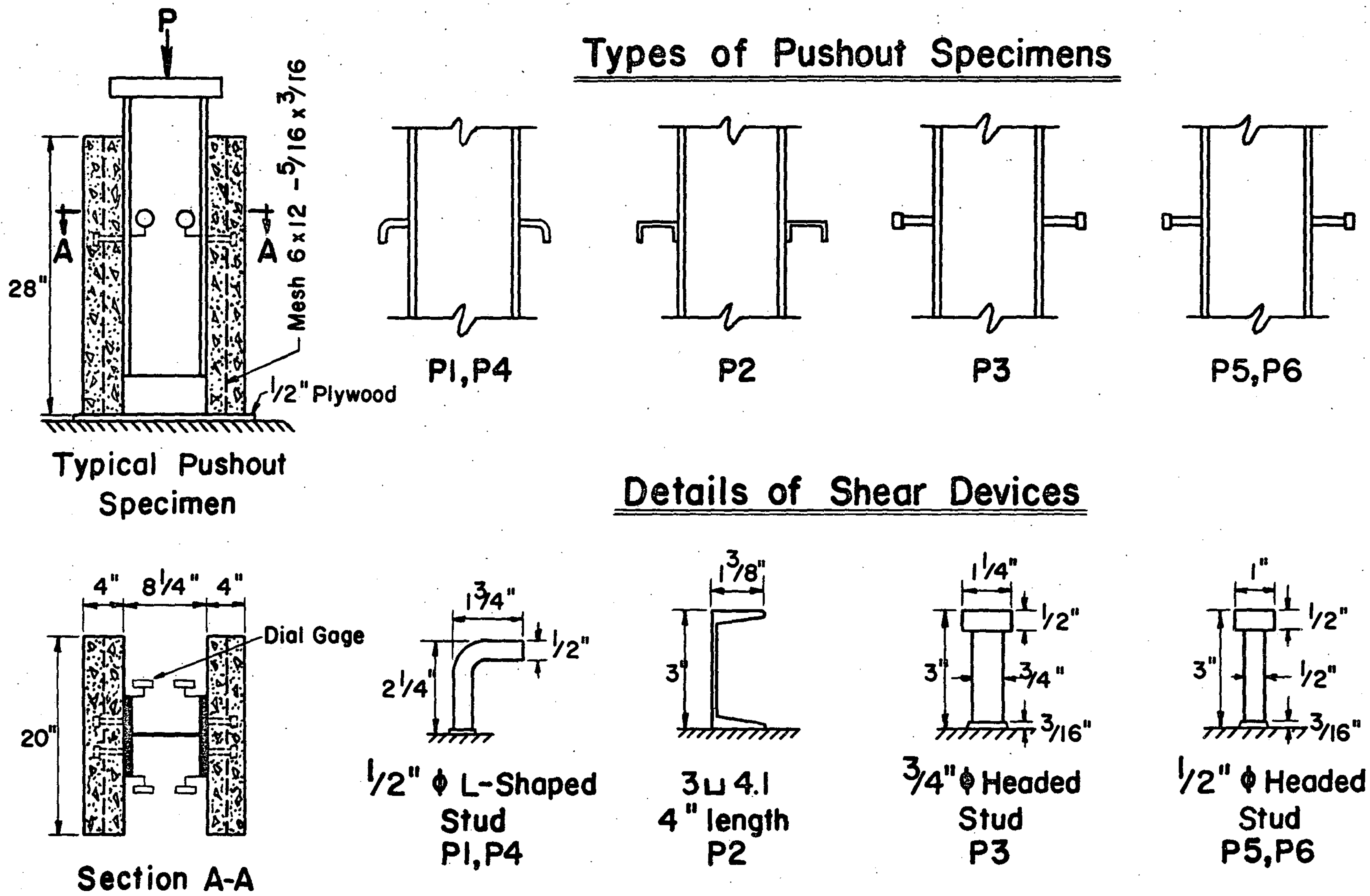


Fig. 2 - Dimensions of Pushout Specimens

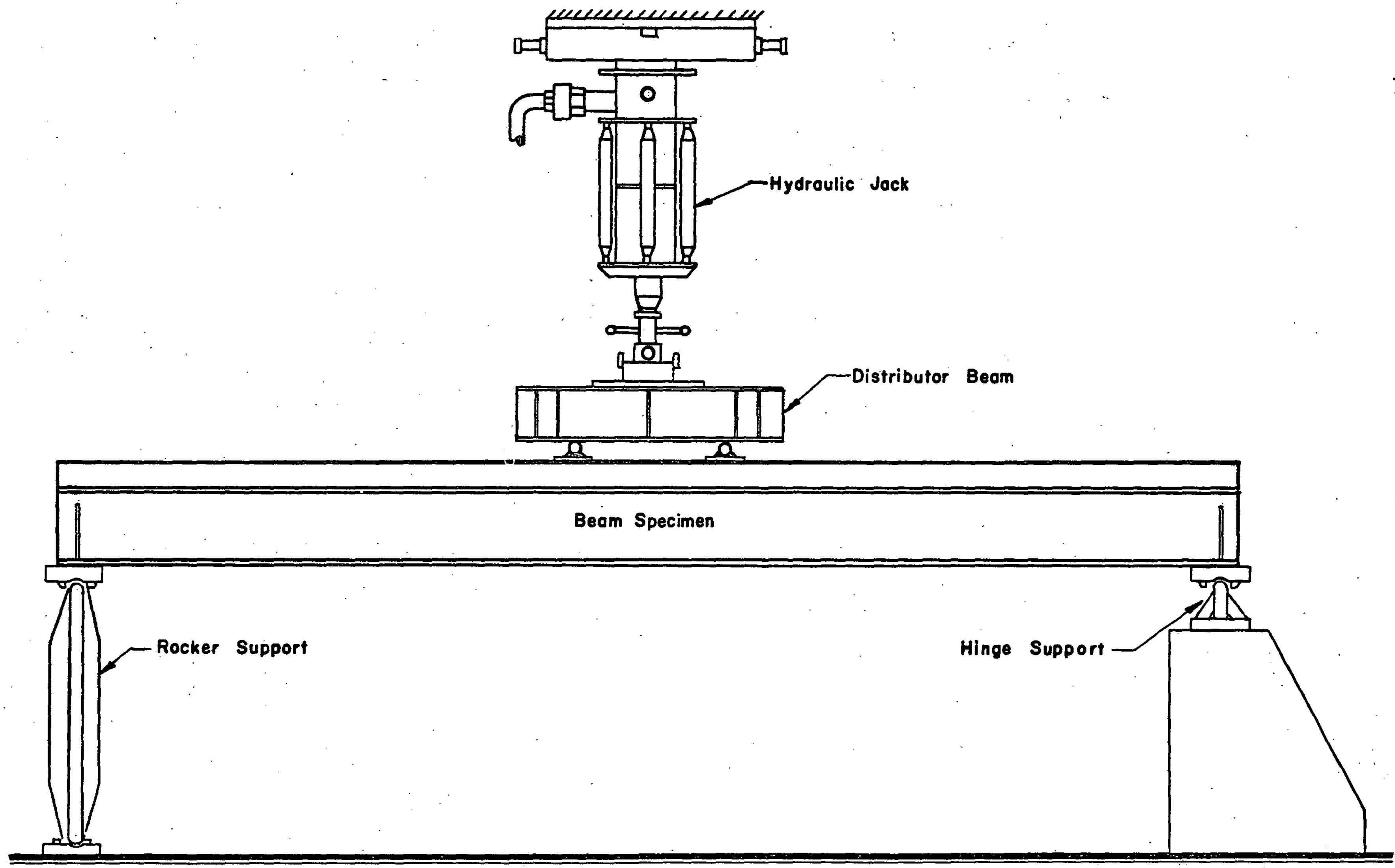


Fig. 3 - Test Setup for Beams with Top Loading

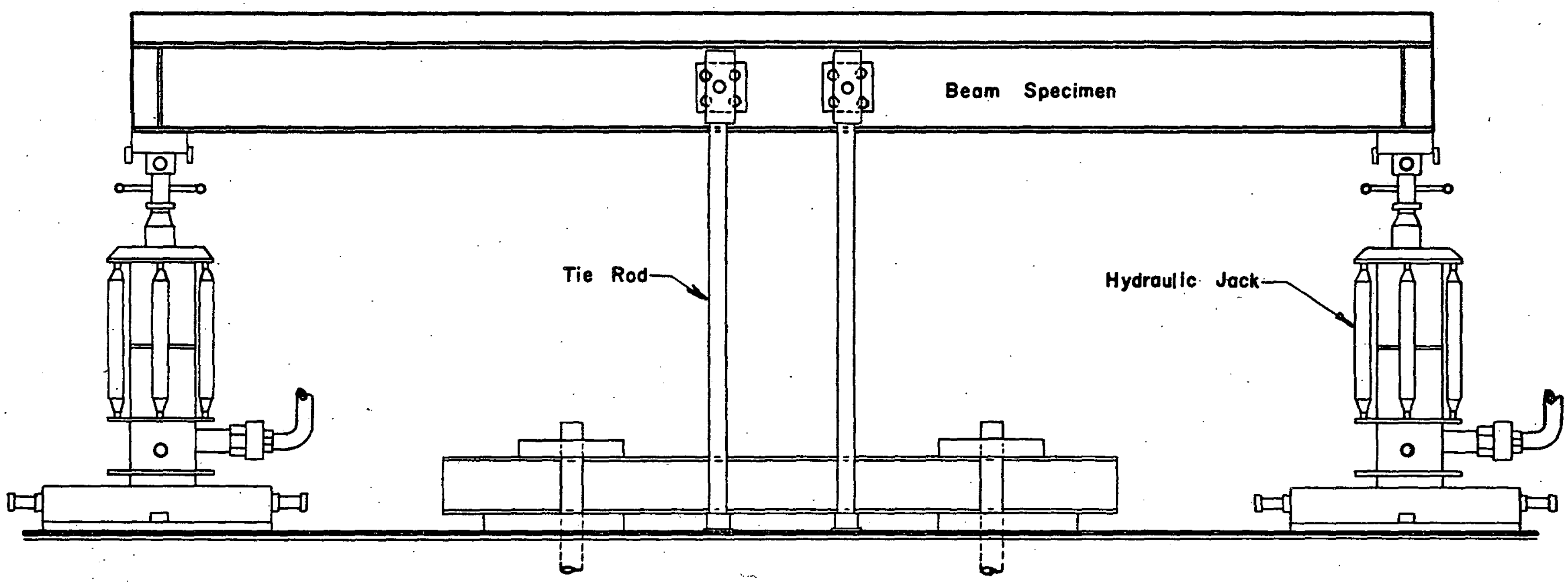


Fig. 4 - Test Setup for Beams with Hanging Loads

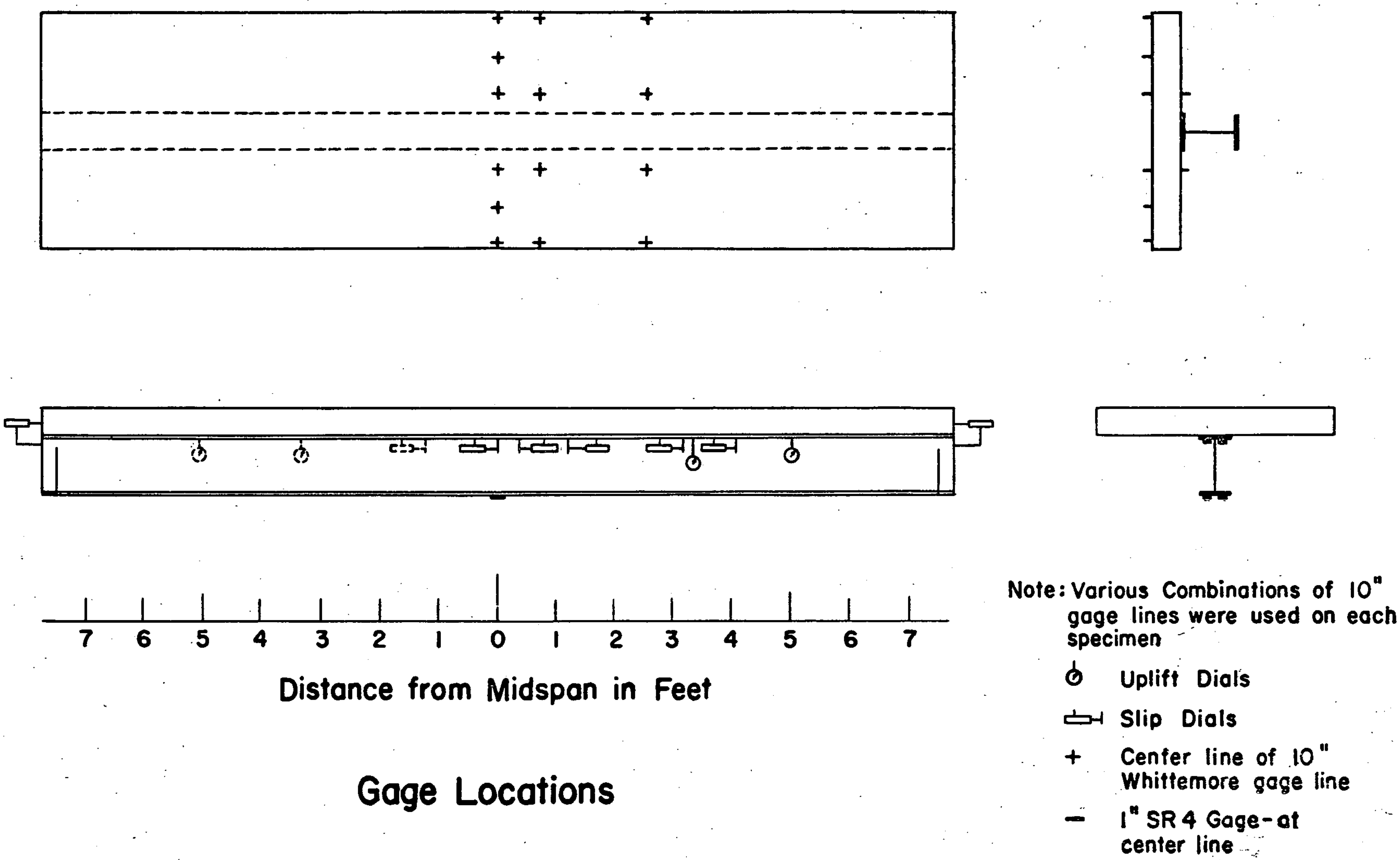


Fig. 5 - Arrangement of Recording Gages for Beam Specimens

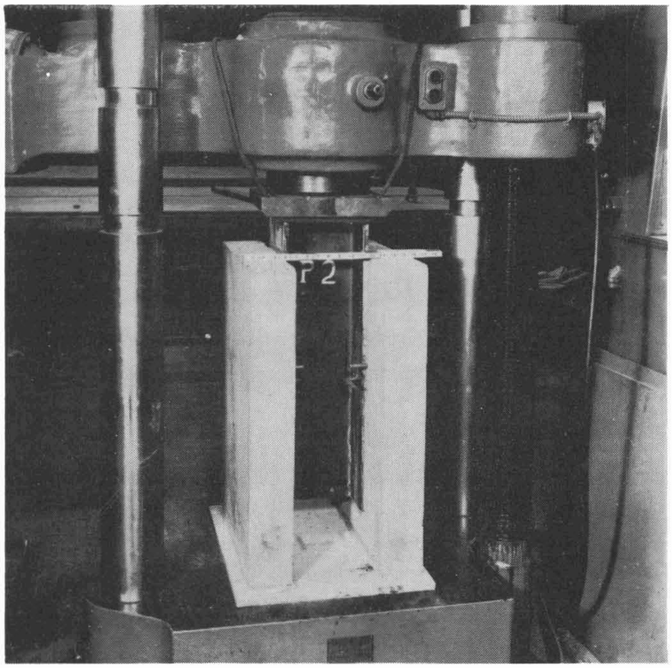


Fig. 6 - Test of Pushout Specimen

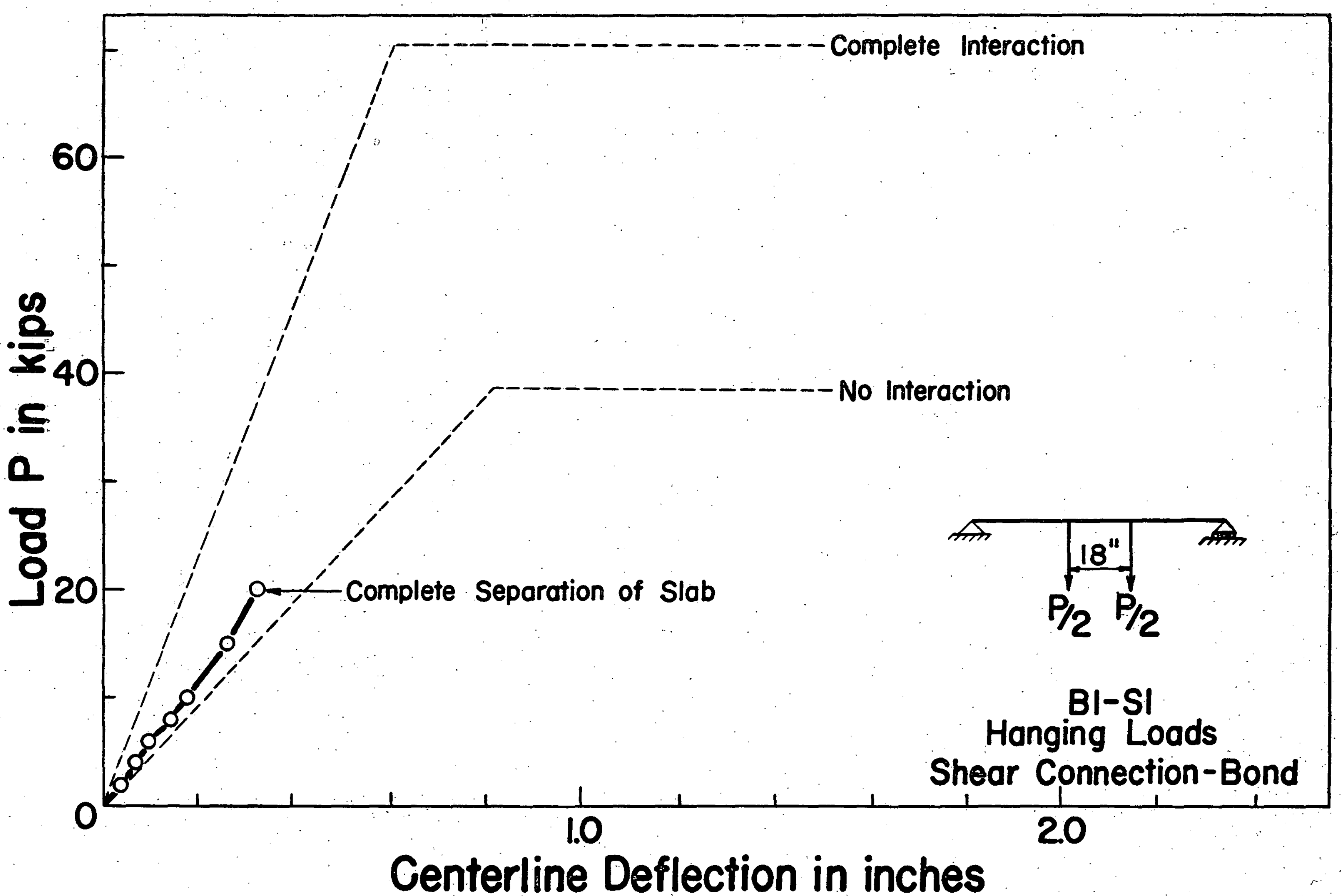


Fig. 7 - Load-Deflection Curve for Beam B1-S1 Without Shear Connectors

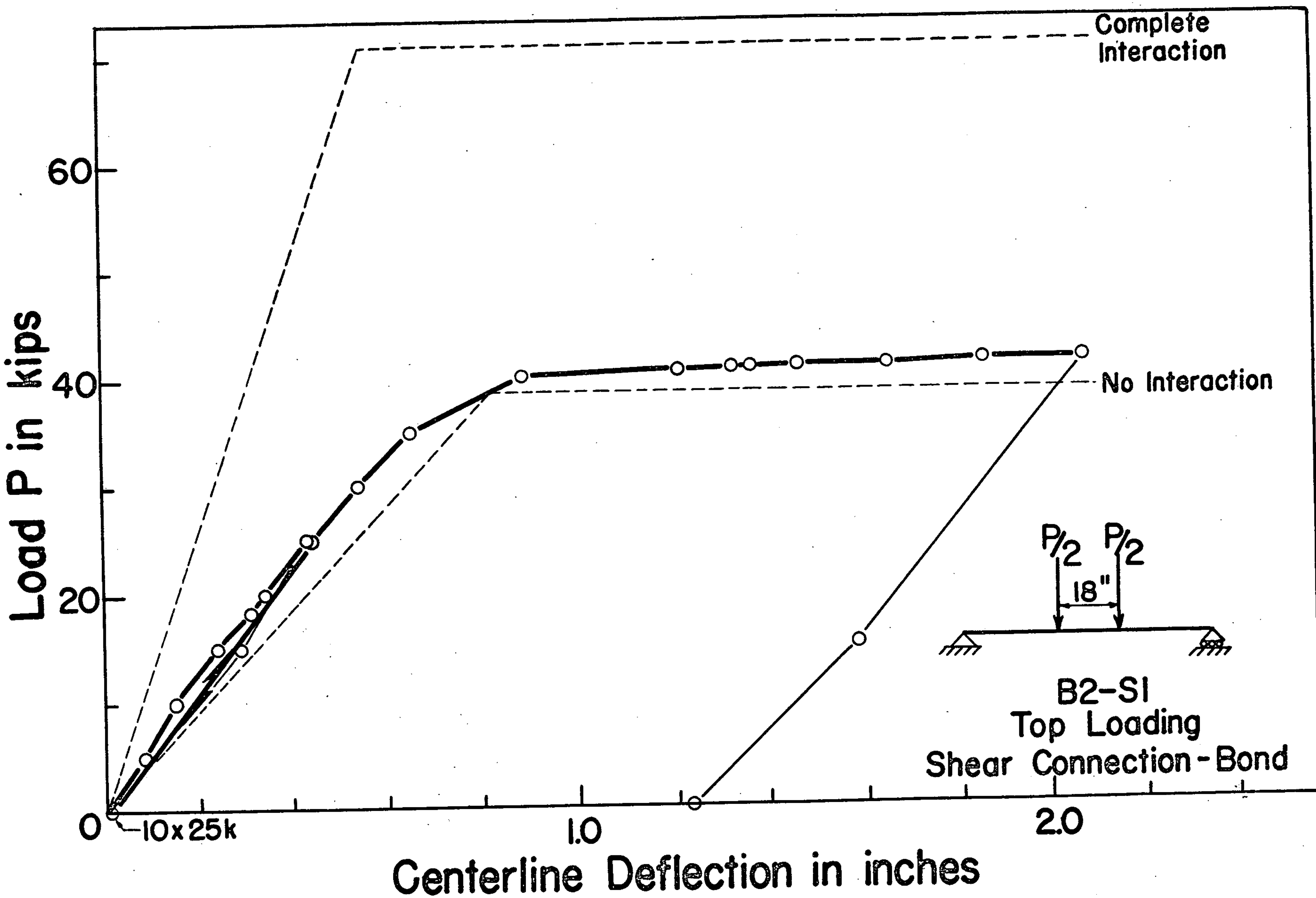


Fig. 8 - Load-Deflection Curve for Beam B2-S1 Without Shear Connectors

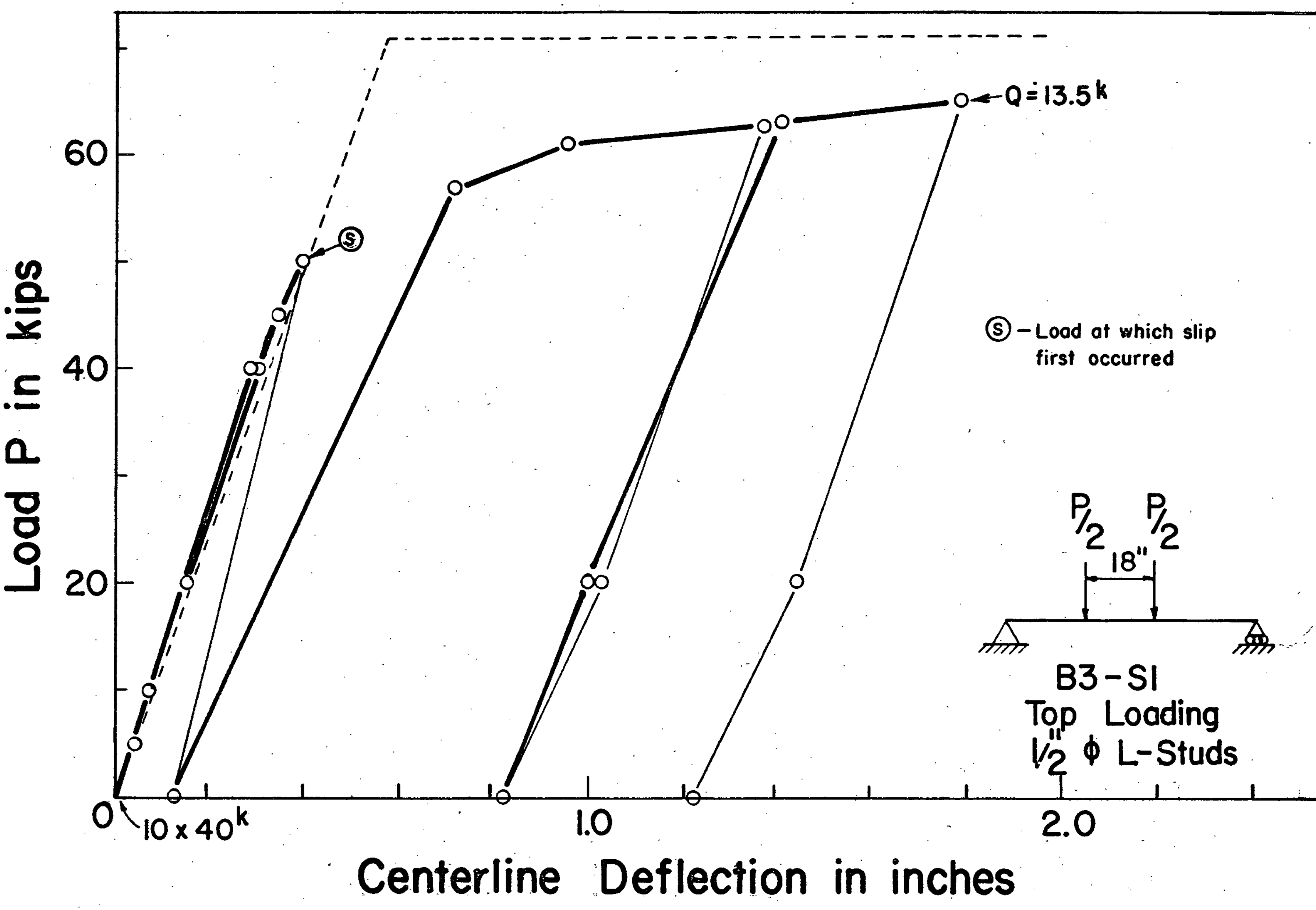


Fig. 9 - Load-Deflection Curve for First Test of Beam B3 with 1/2" L-studs

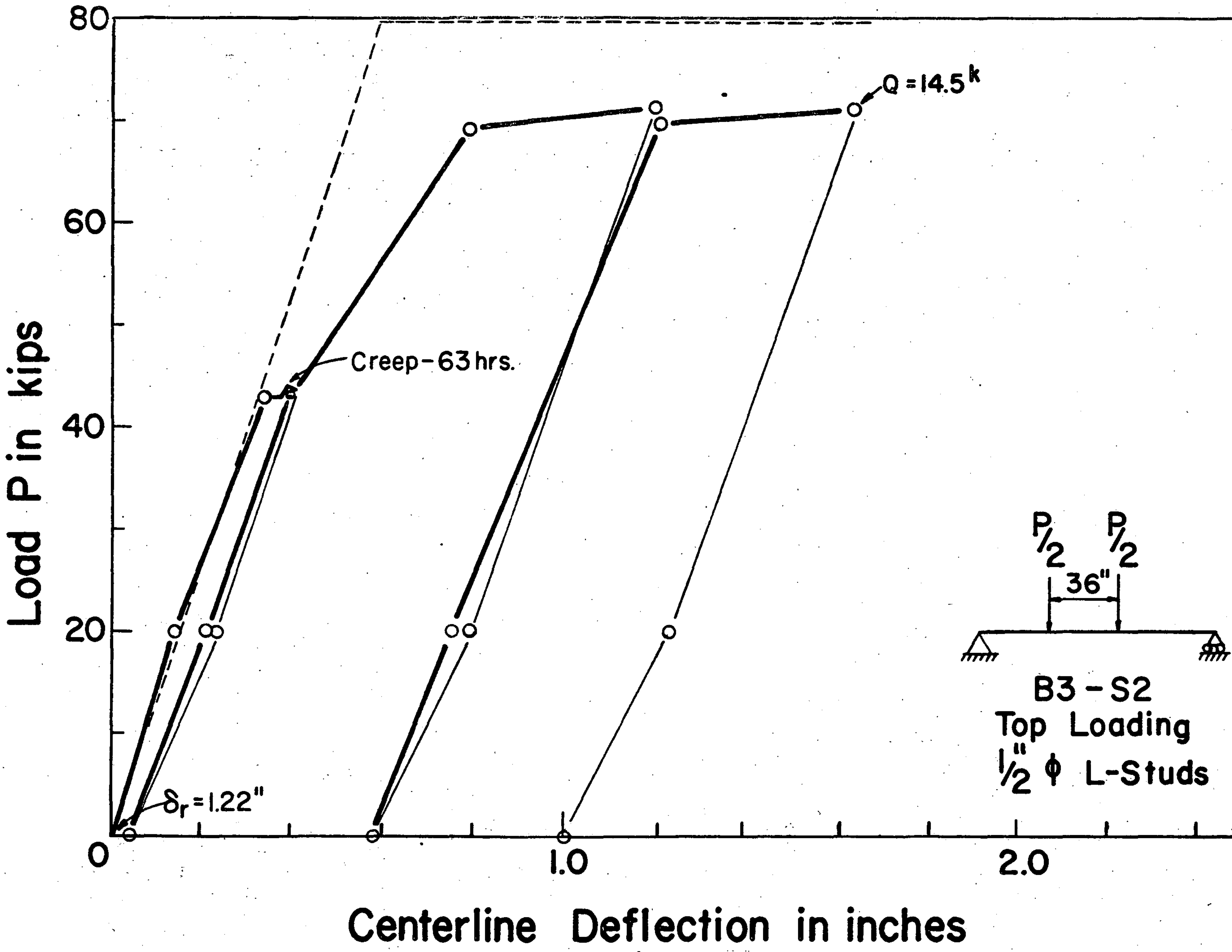


Fig. 10 - Load-Deflection Curve for Second Test of Beam B3 with 1/2" L-studs

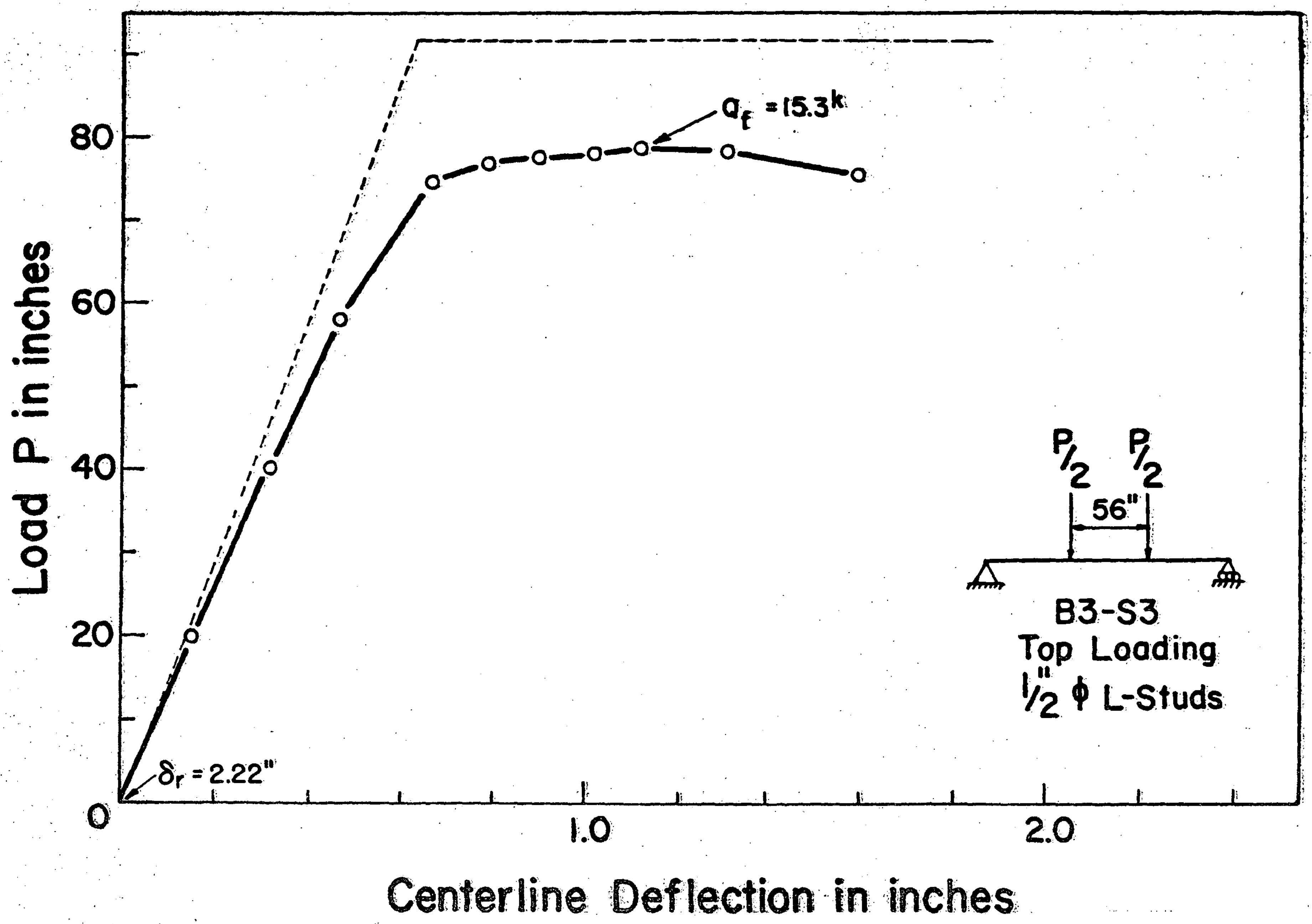


Fig. 11 - Load-Deflection Curve for Final Test of Beam B3 with 1/2" L-studs

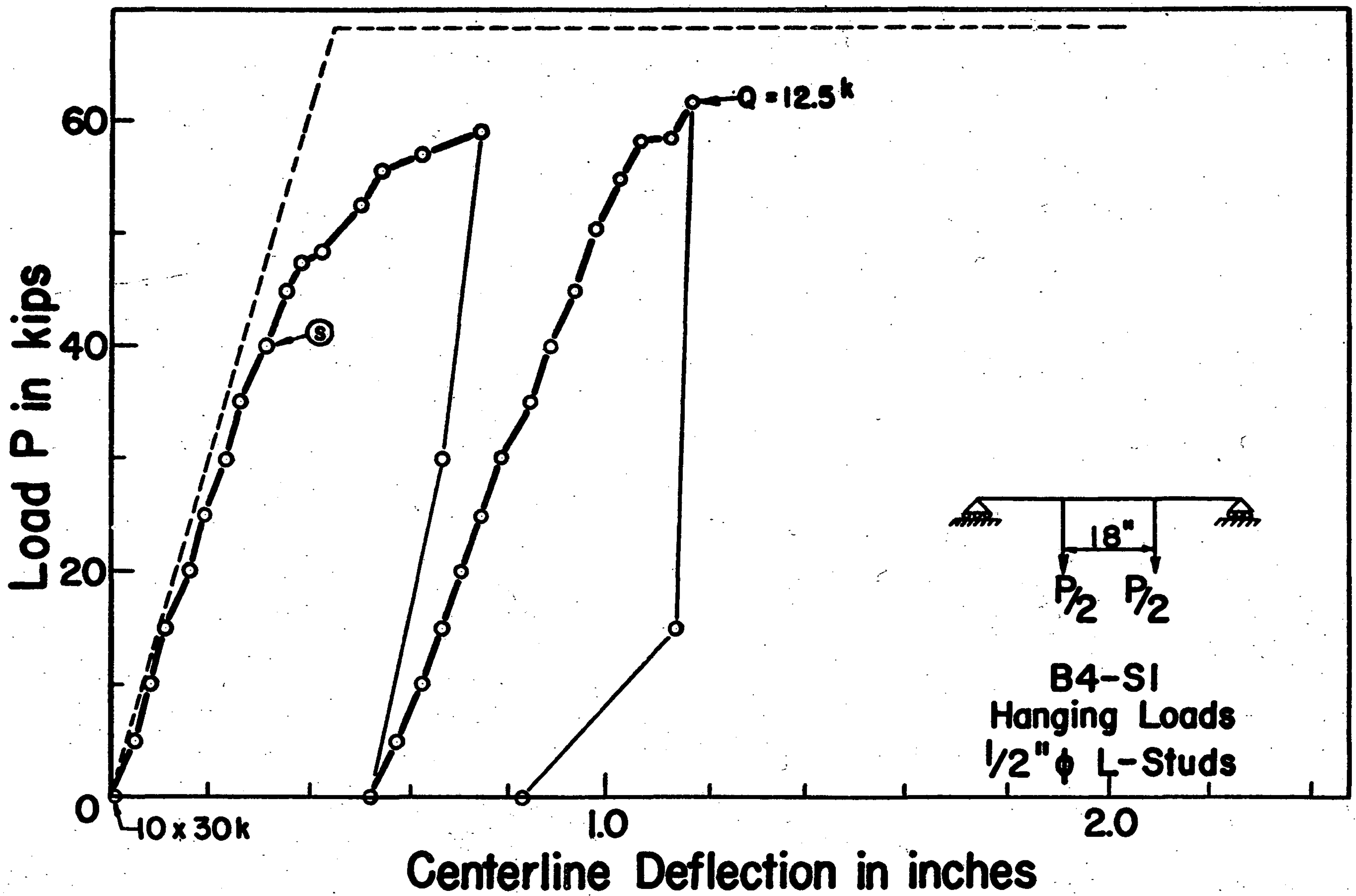


Fig. 12 - Load-Deflection Curve for First Test of Beam B4 with 1/2" L-studs

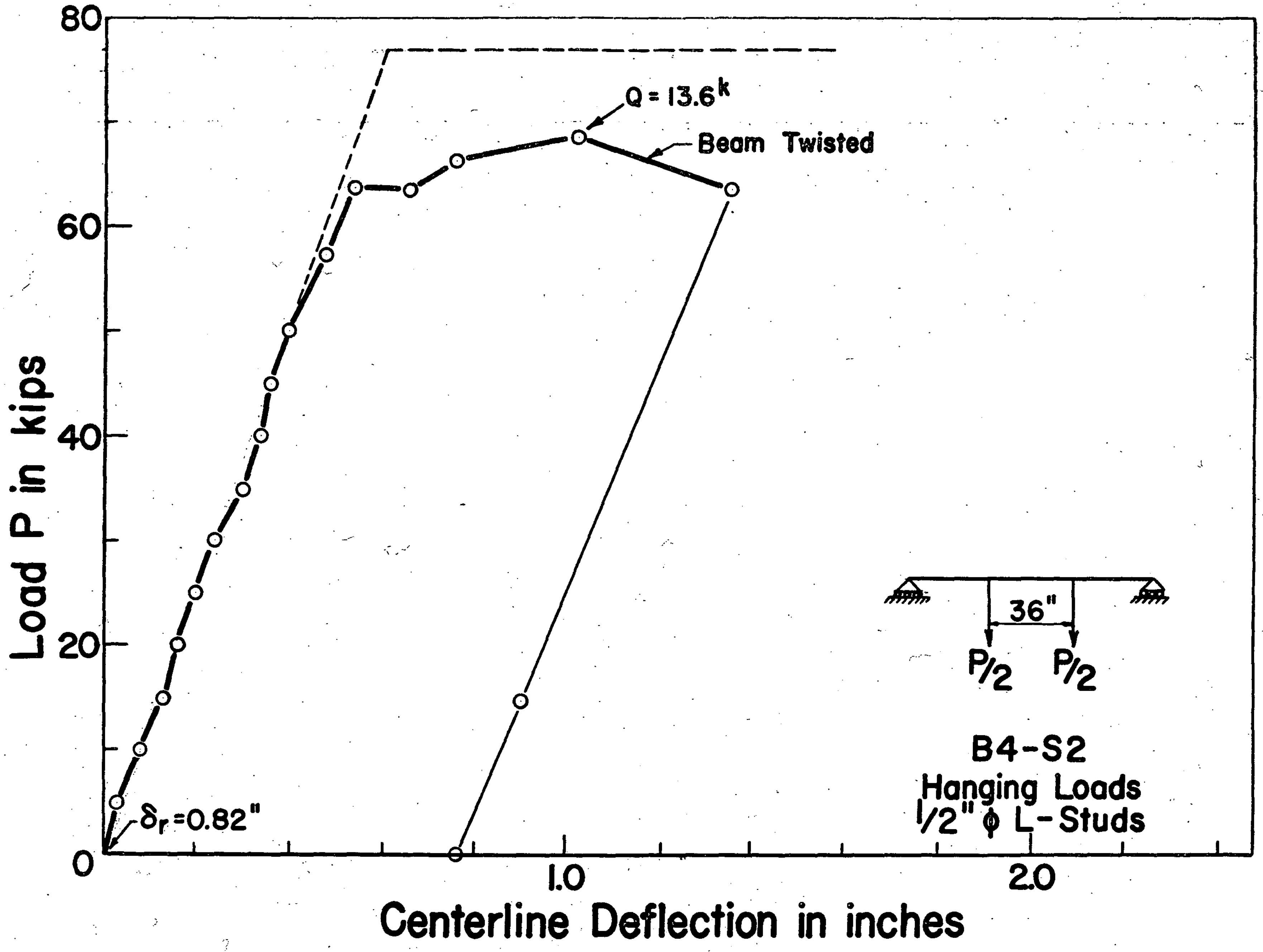


Fig. 13 - Load-Deflection Curve for Second Test of Beam B4 with 1/2" L-studs

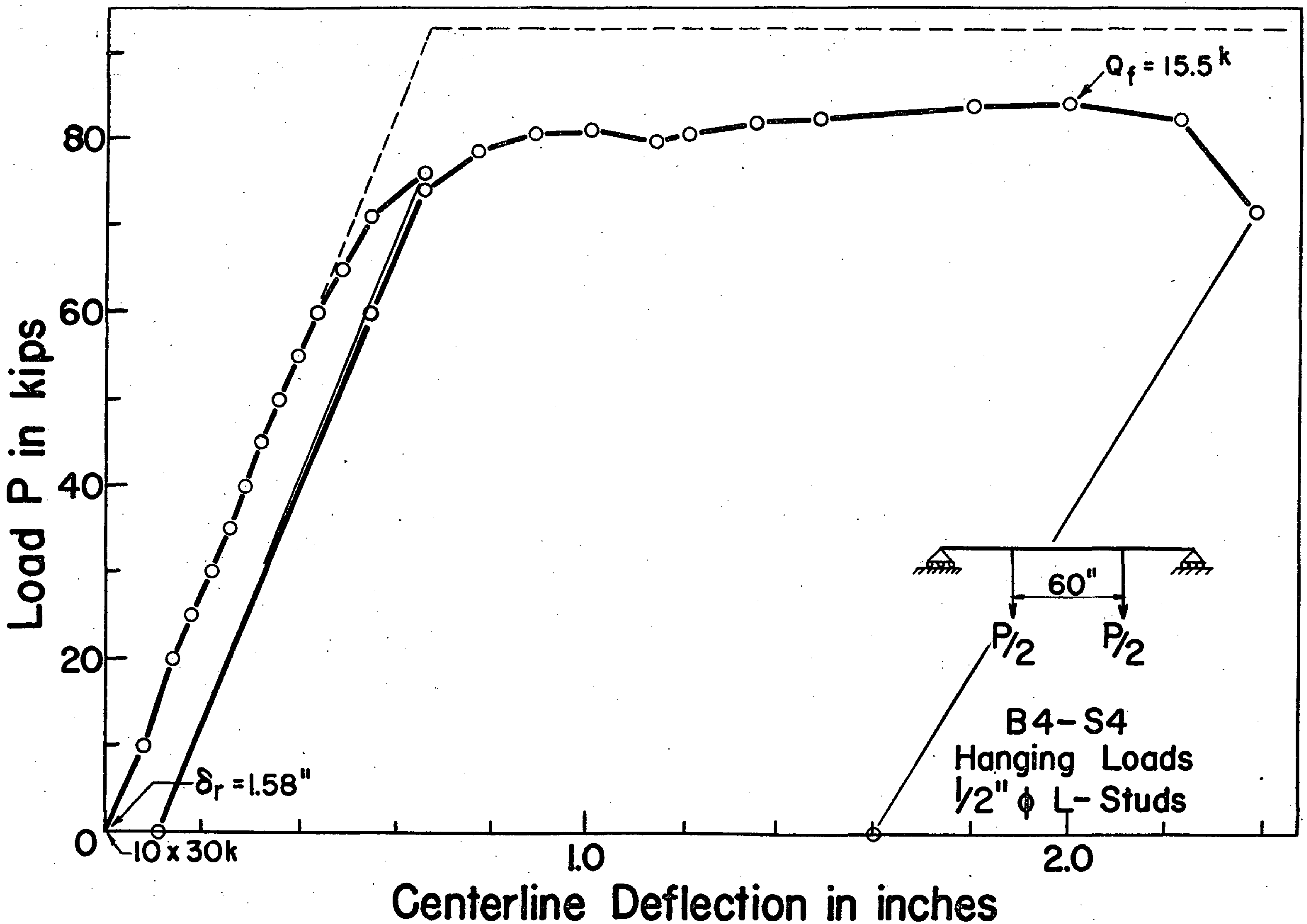


Fig. 14 - Load-Deflection Curve for Final Test of Beam B4 with 1/2" L-studs

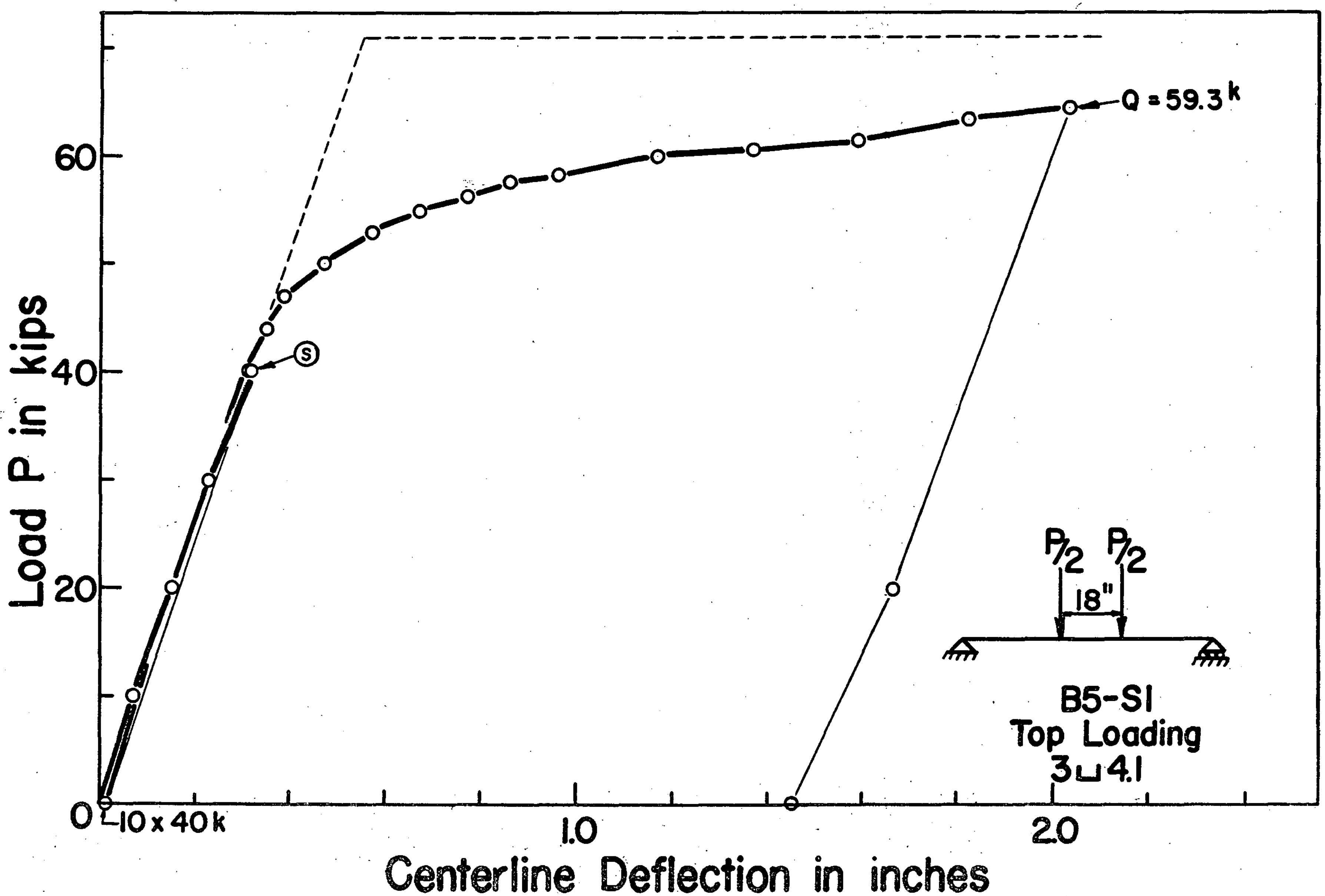


Fig. 15 - Load-Deflection Curve for First Test of Beam B5 with Channel Shear Connectors

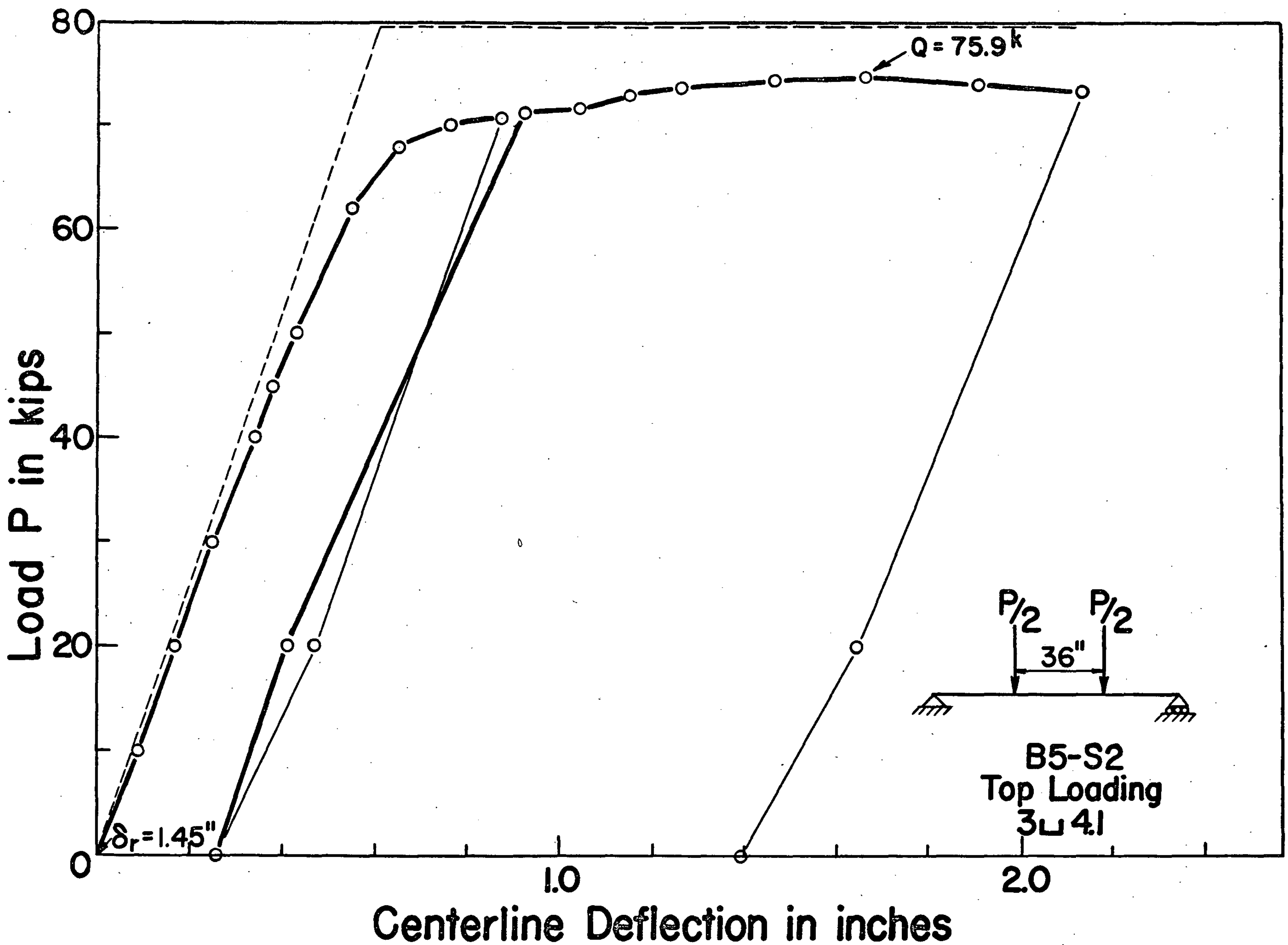


Fig. 16 - Load-Deflection Curve for Second Test of Beam B5 with Channel Shear Connectors

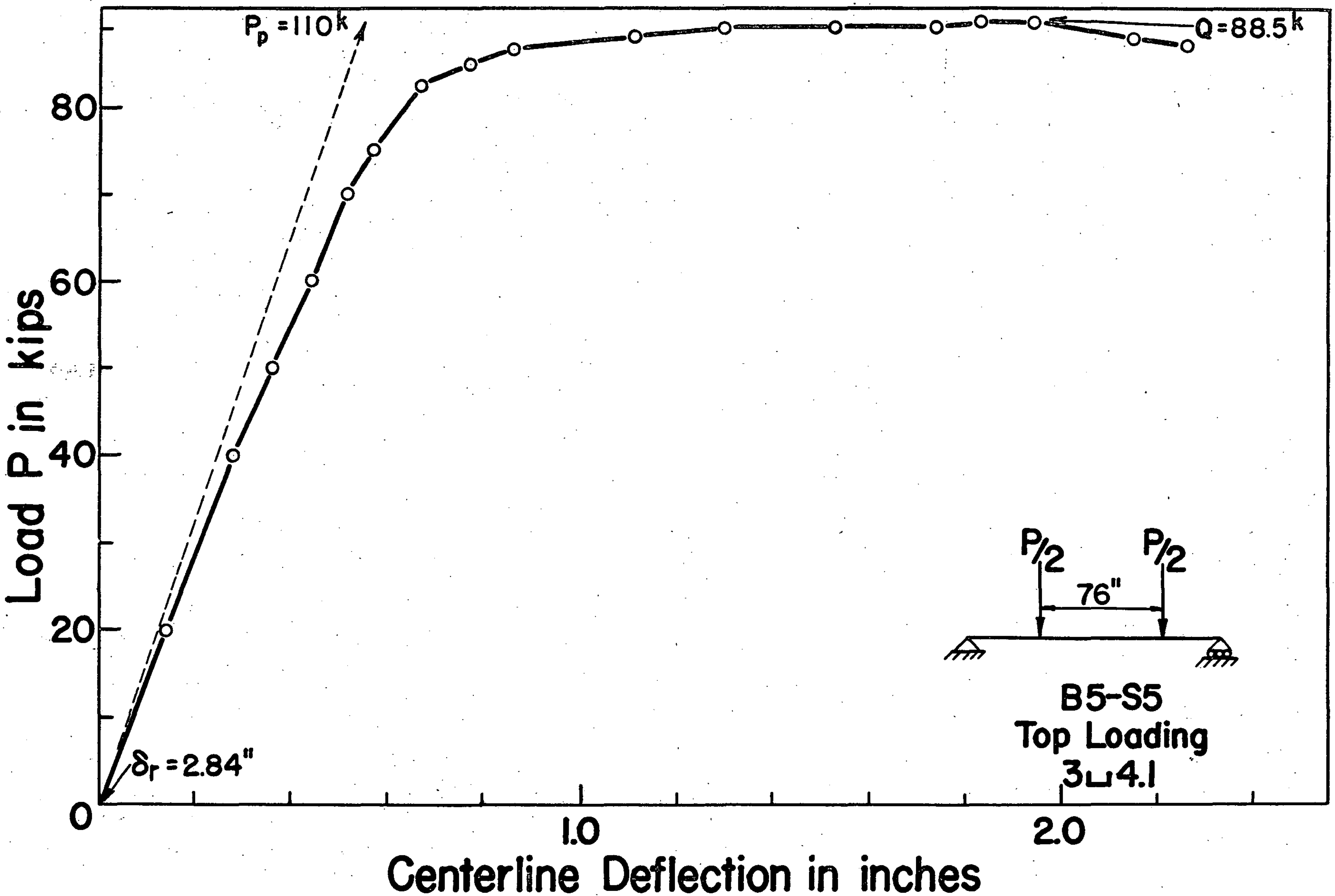


Fig. 17 - Load-Deflection Curve for Final Test of Beam B5 with Channel Shear Connectors

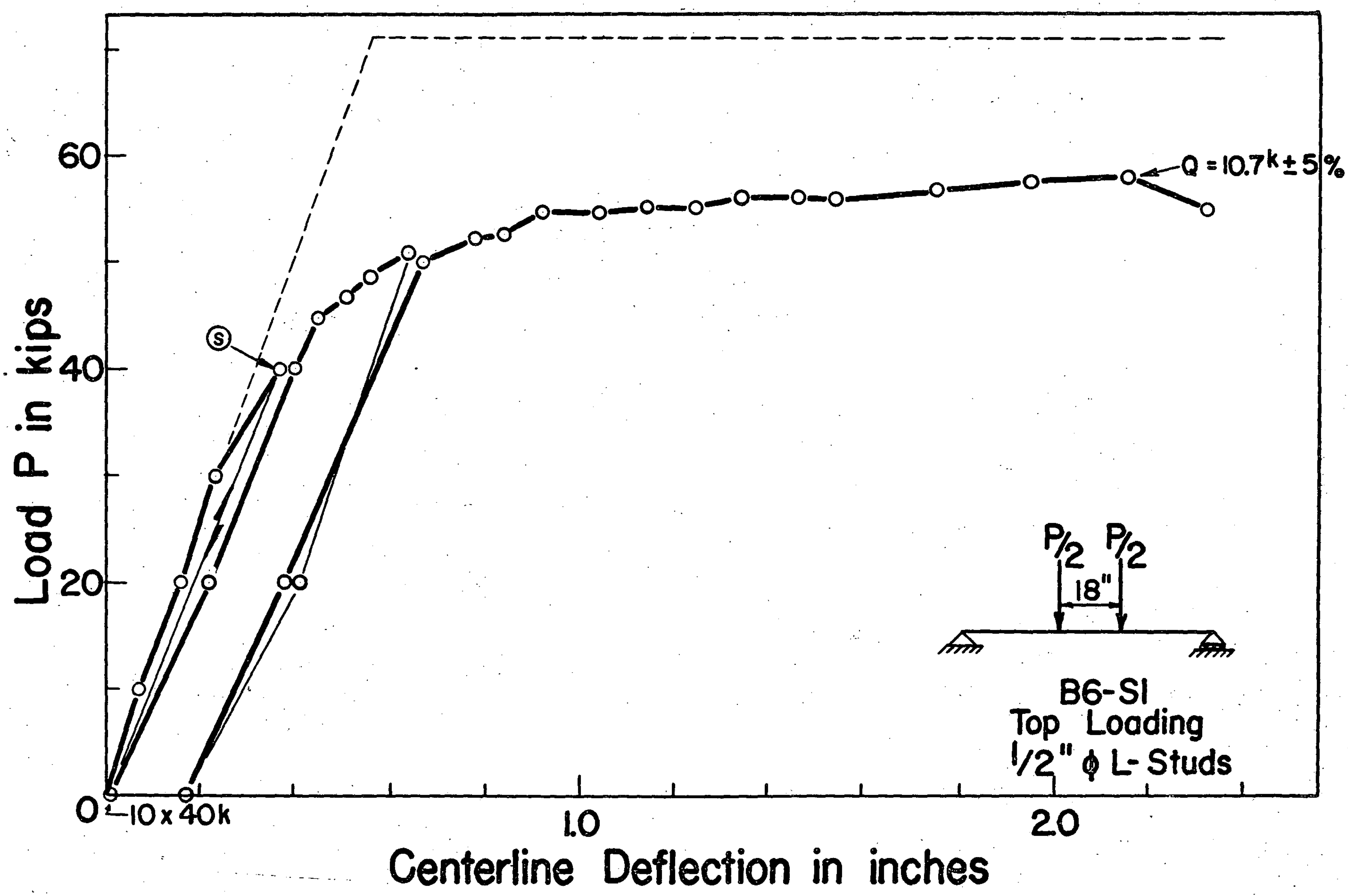


Fig. 18 - Load-Deflection Curve for Test of Beam B6 with 1/2" L-studs

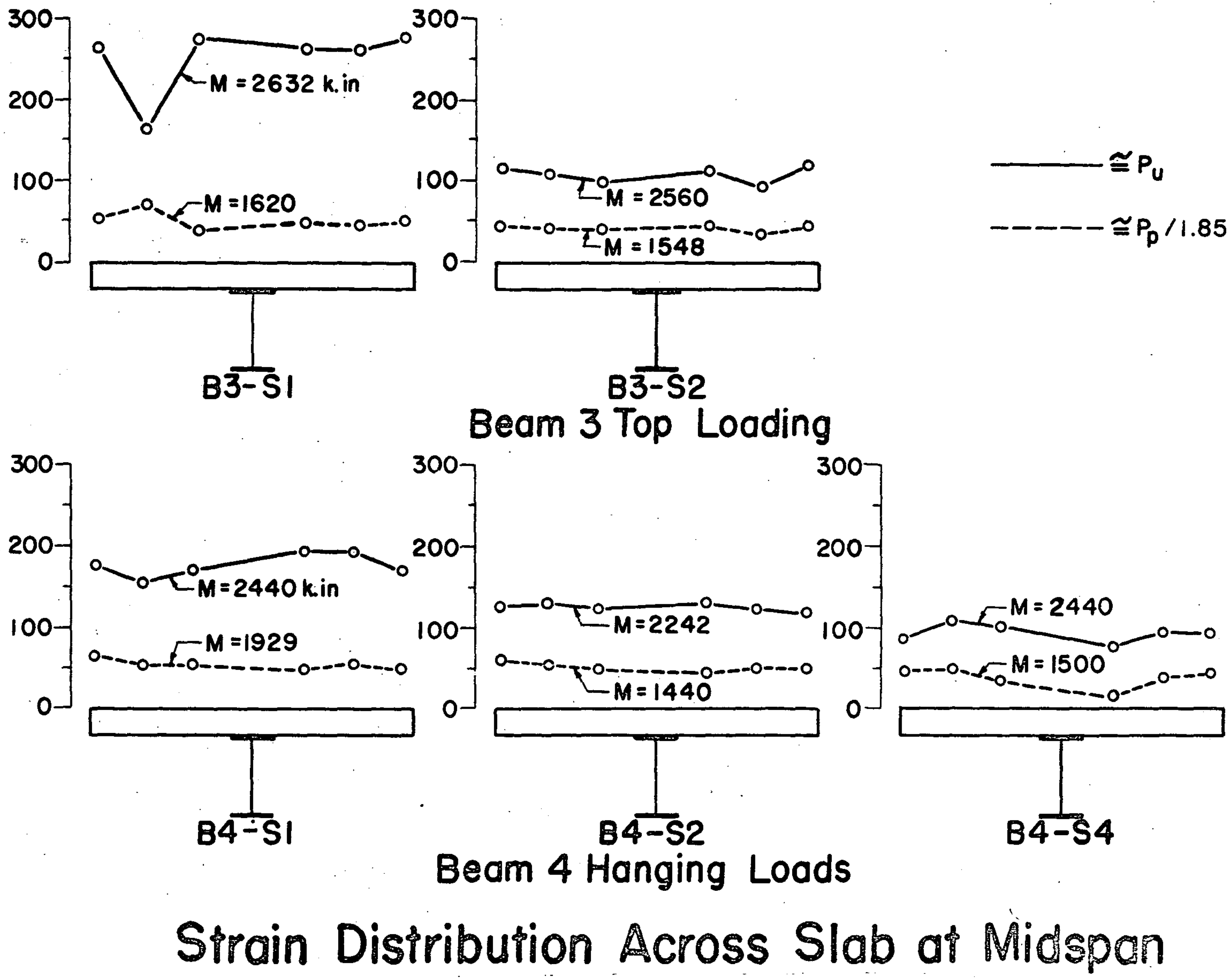
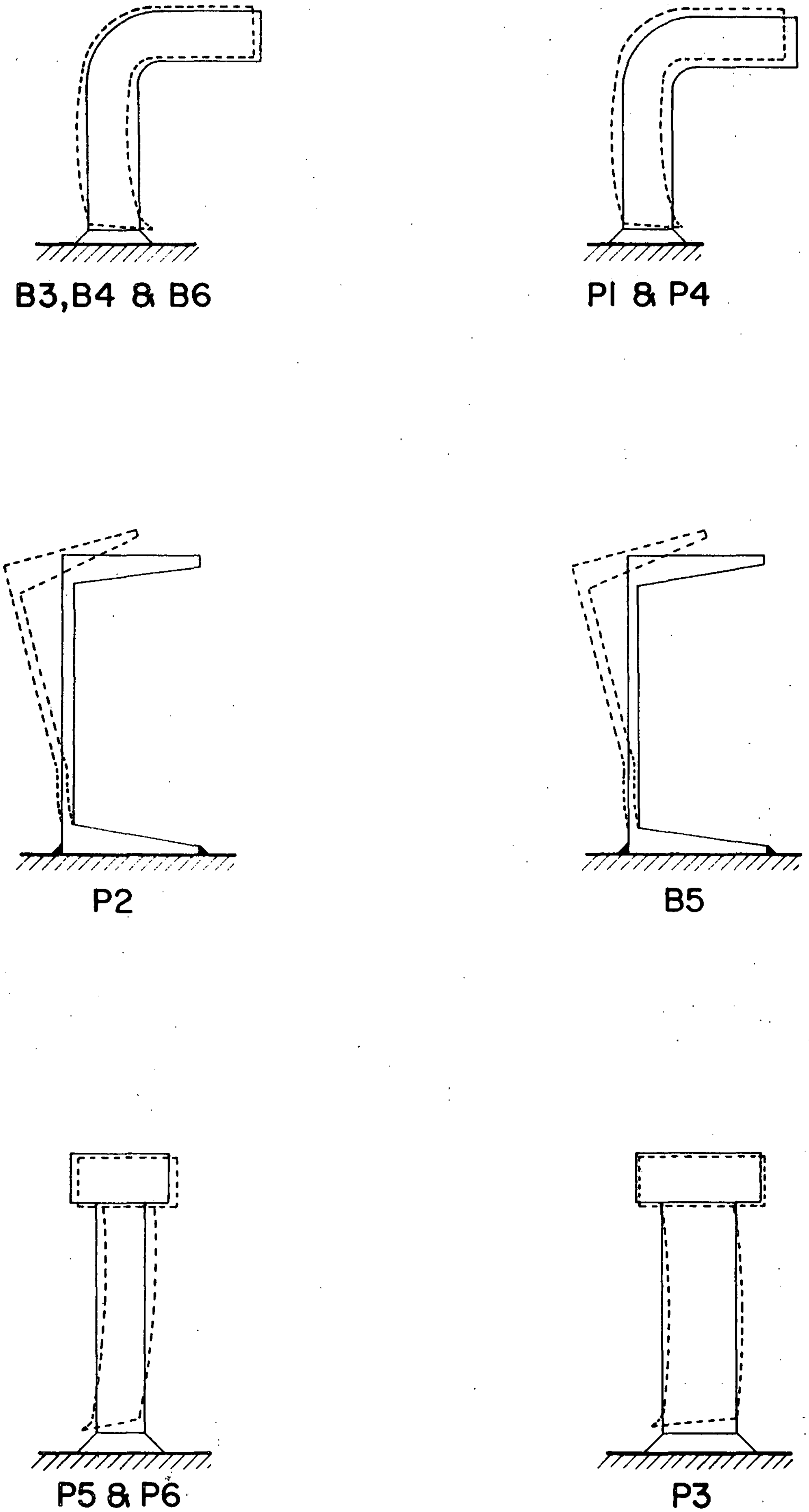


Fig. 19 - Strain Distribution Across Slab



Typical Connector Failures

Fig. 20 - Deformed Shape of Connectors After Failure

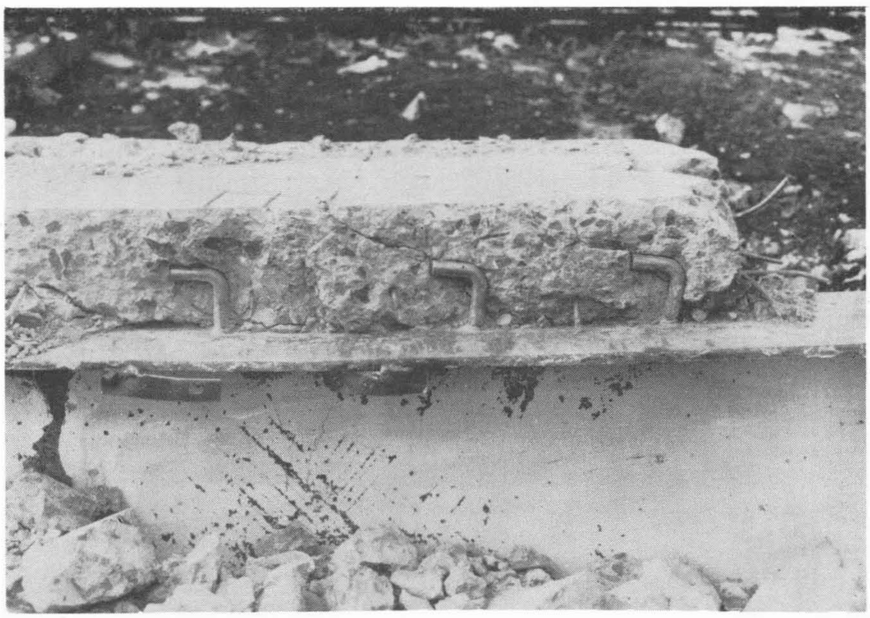


Fig. 21 - Deformed Studs in Beam Specimen B 6

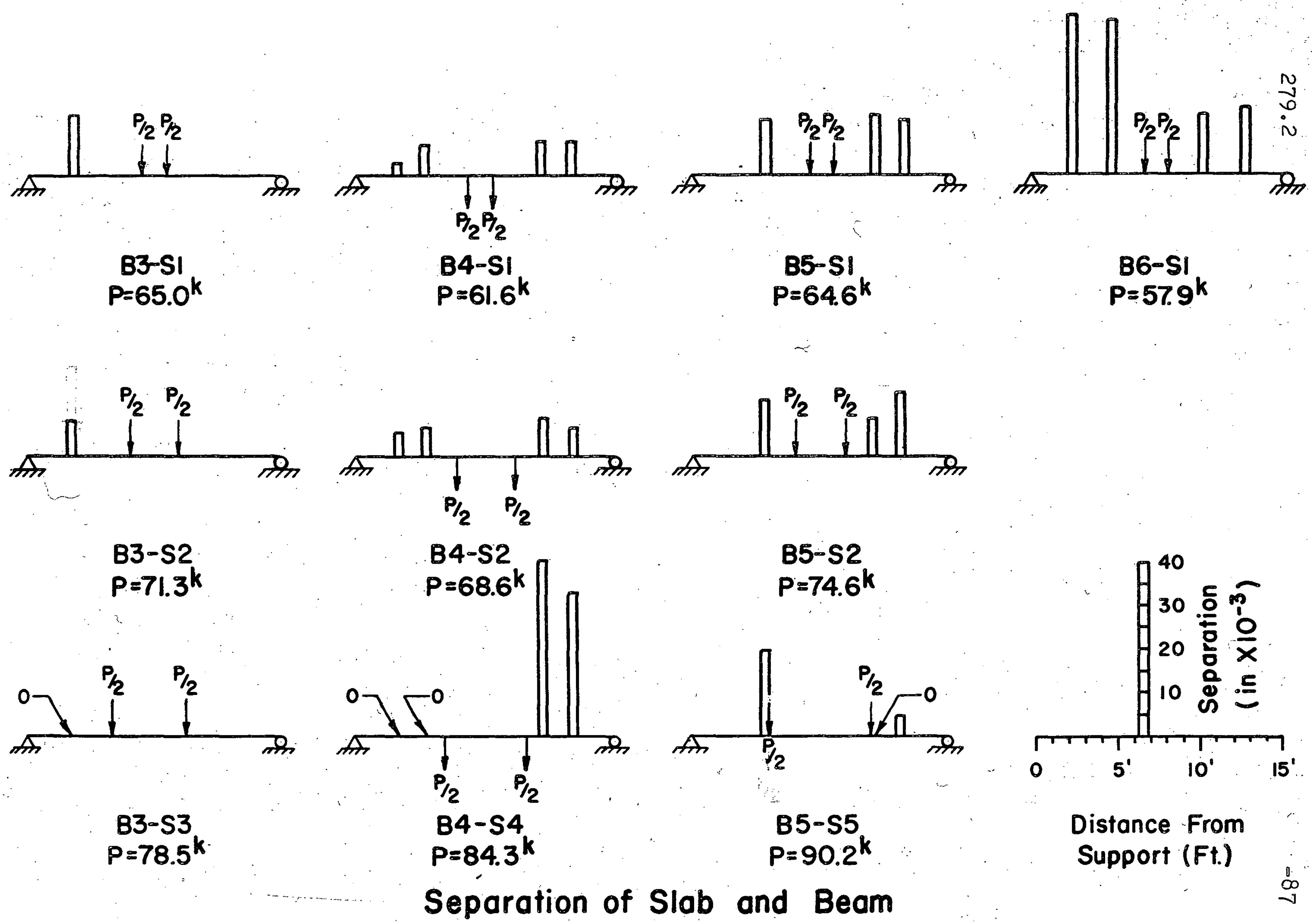


Fig. 22 - Separation of Slab and Beam

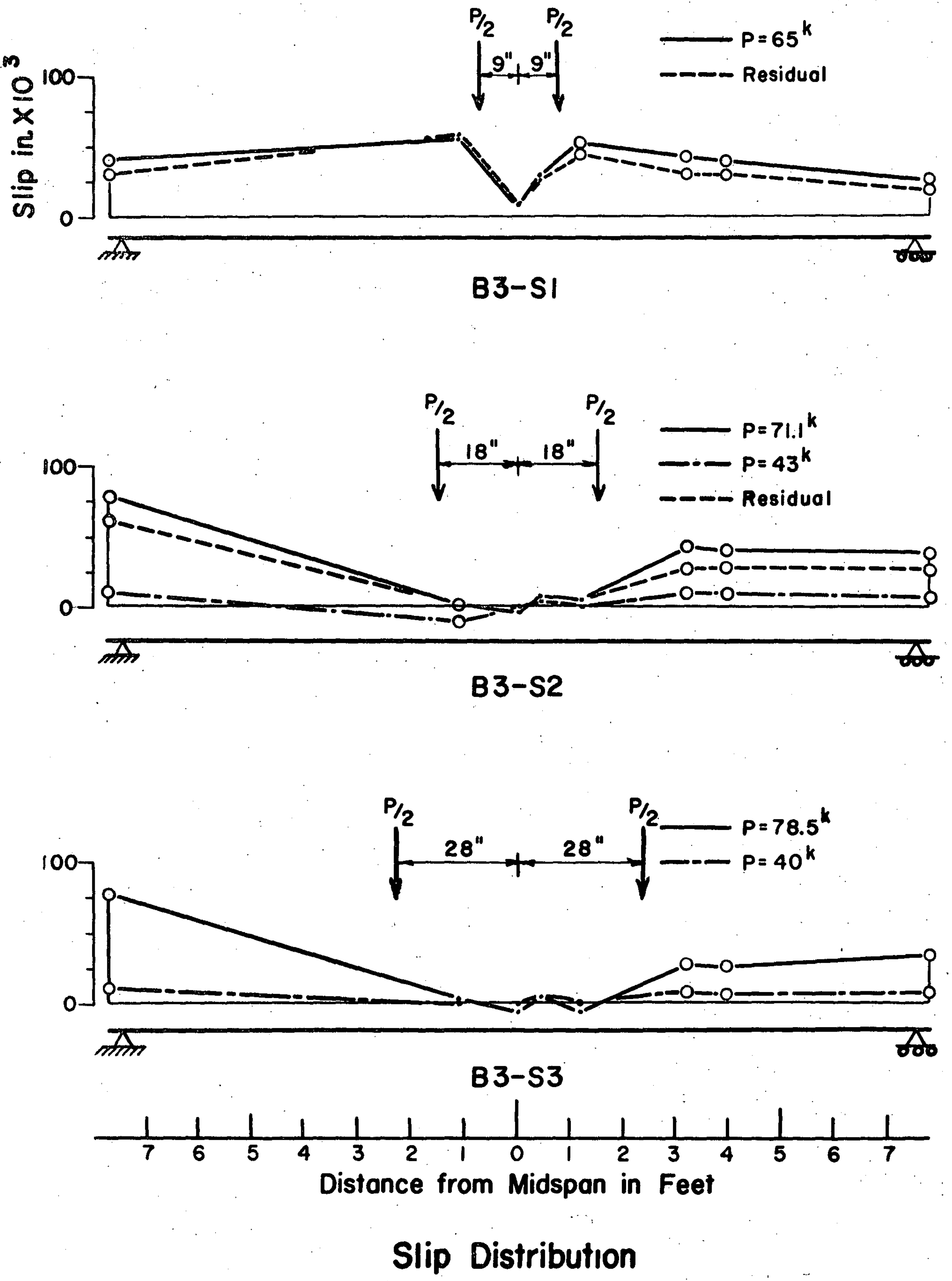


Fig. 23 - Slip Distribution Along Beam B3

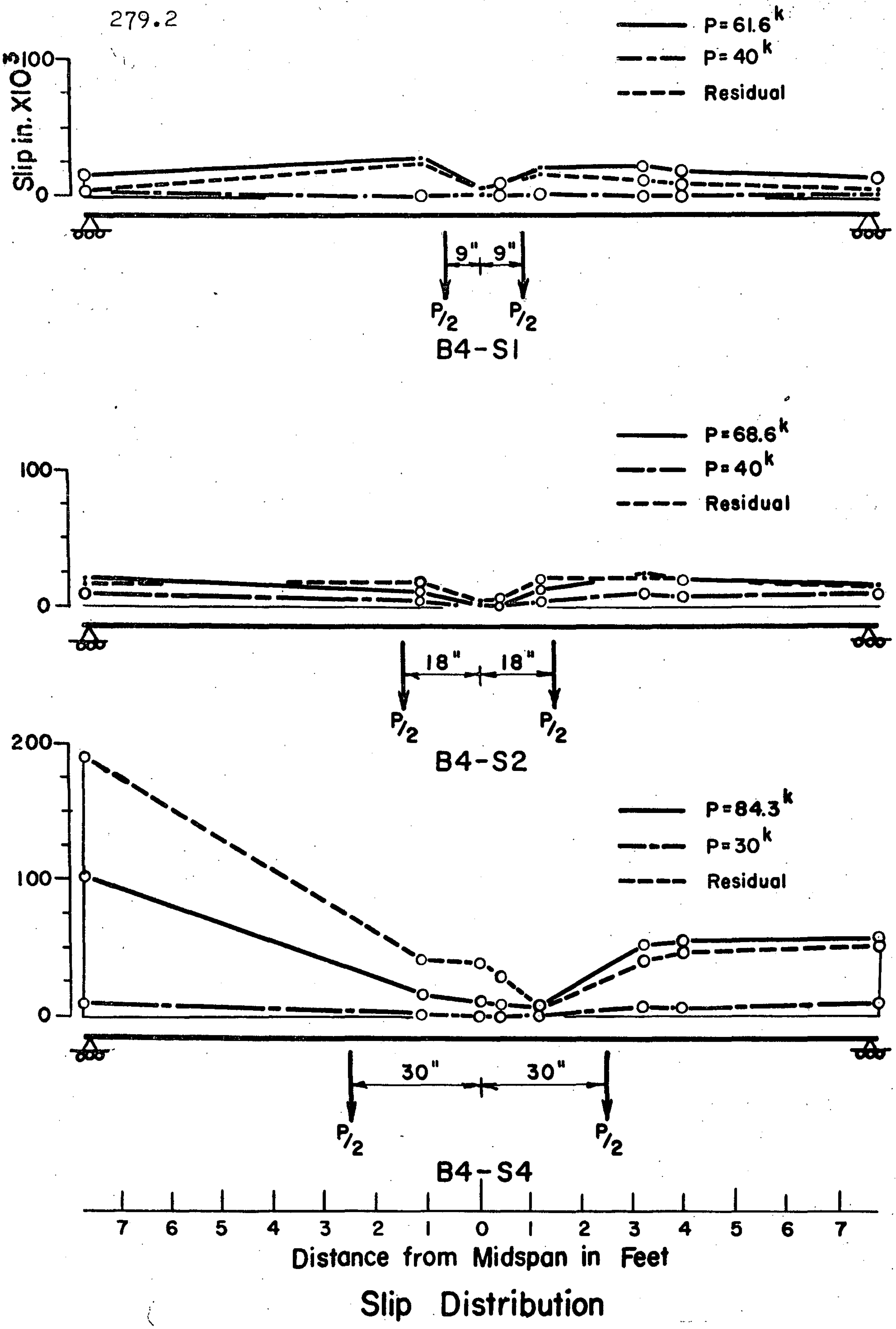
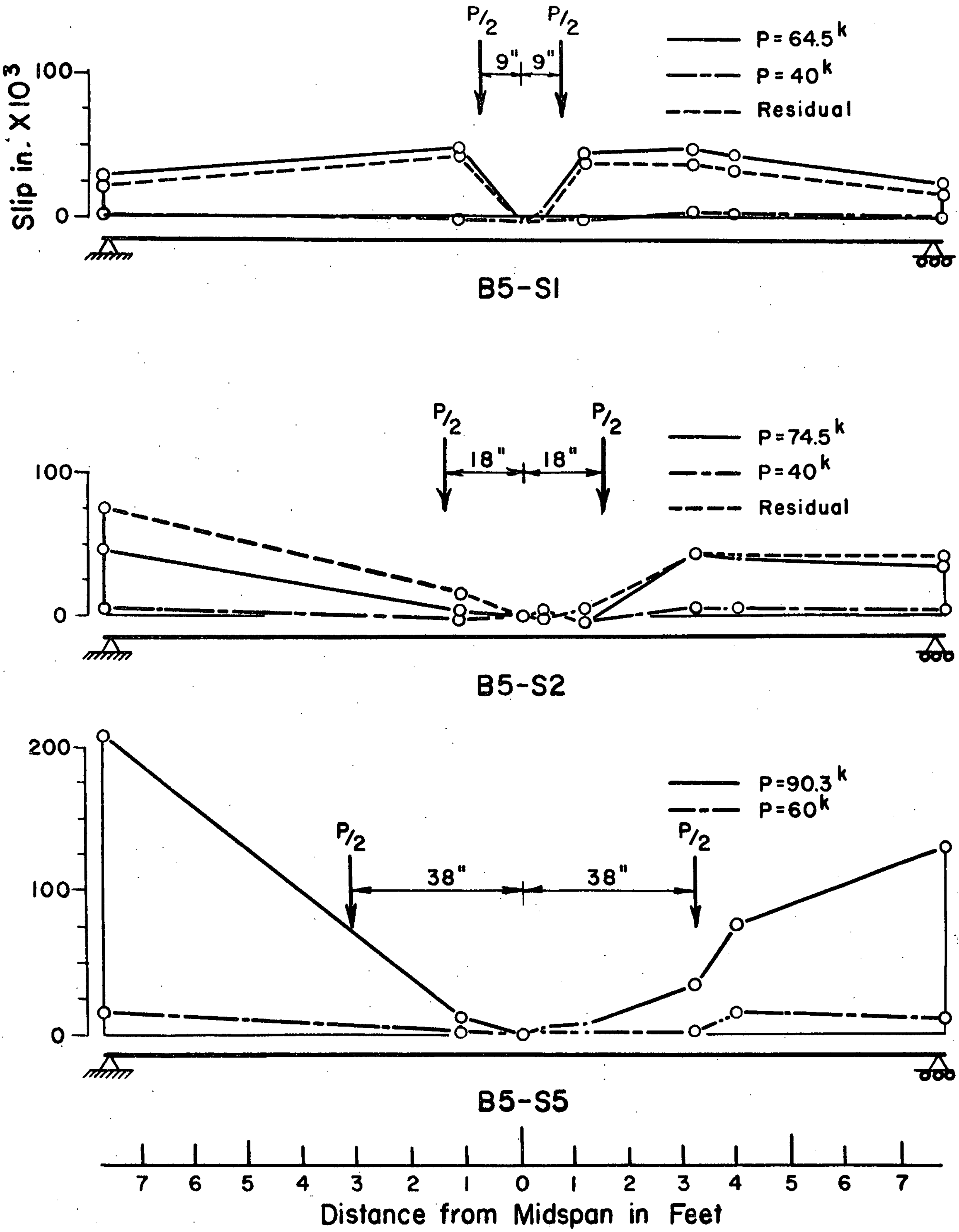


Fig. 24 - Slip Distribution Along Beam B4



Slip Distribution

Fig. 25 - Slip Distribution Along Beam B5

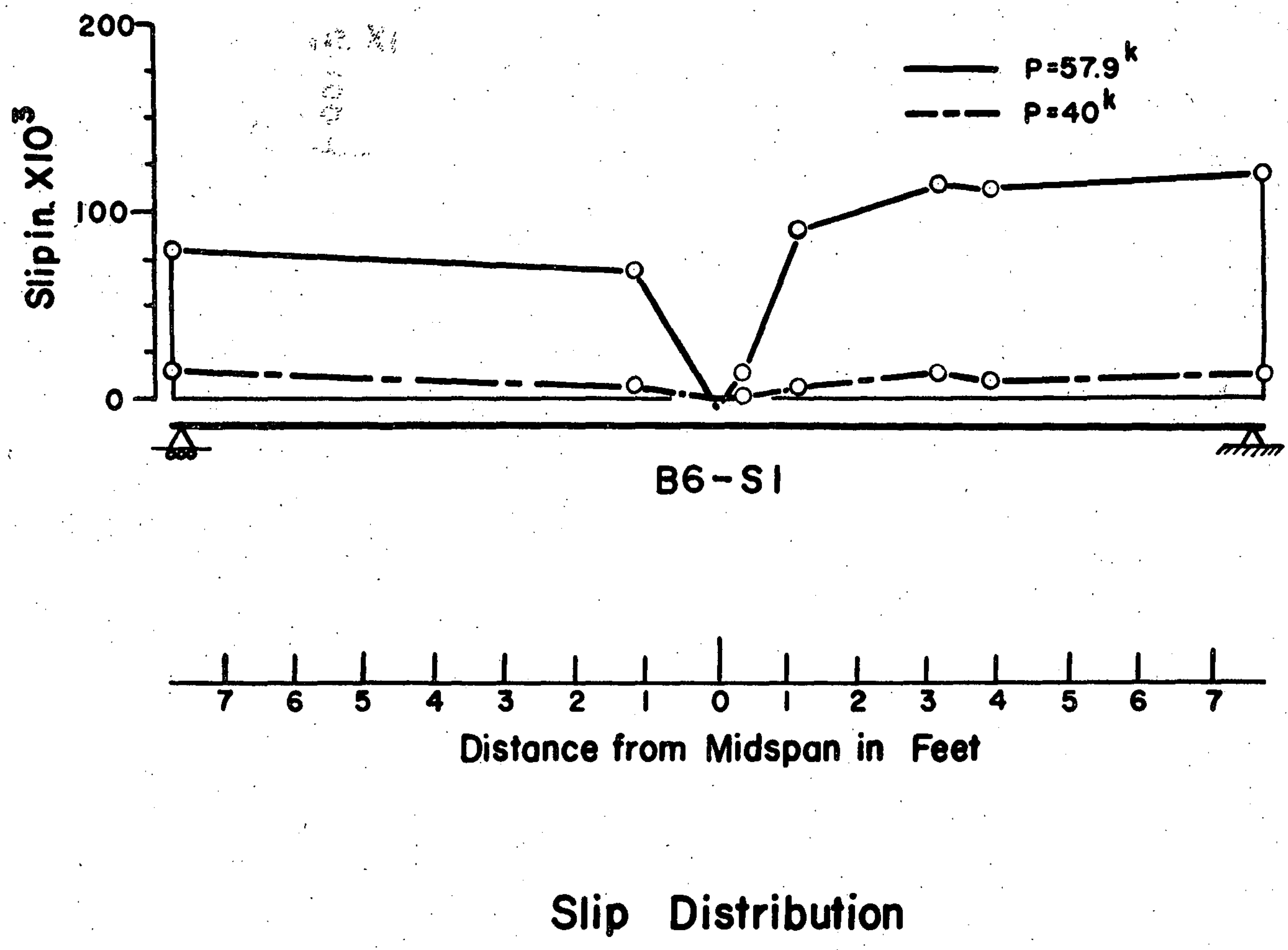


Fig. 26 - Slip Distribution Along Beam B6

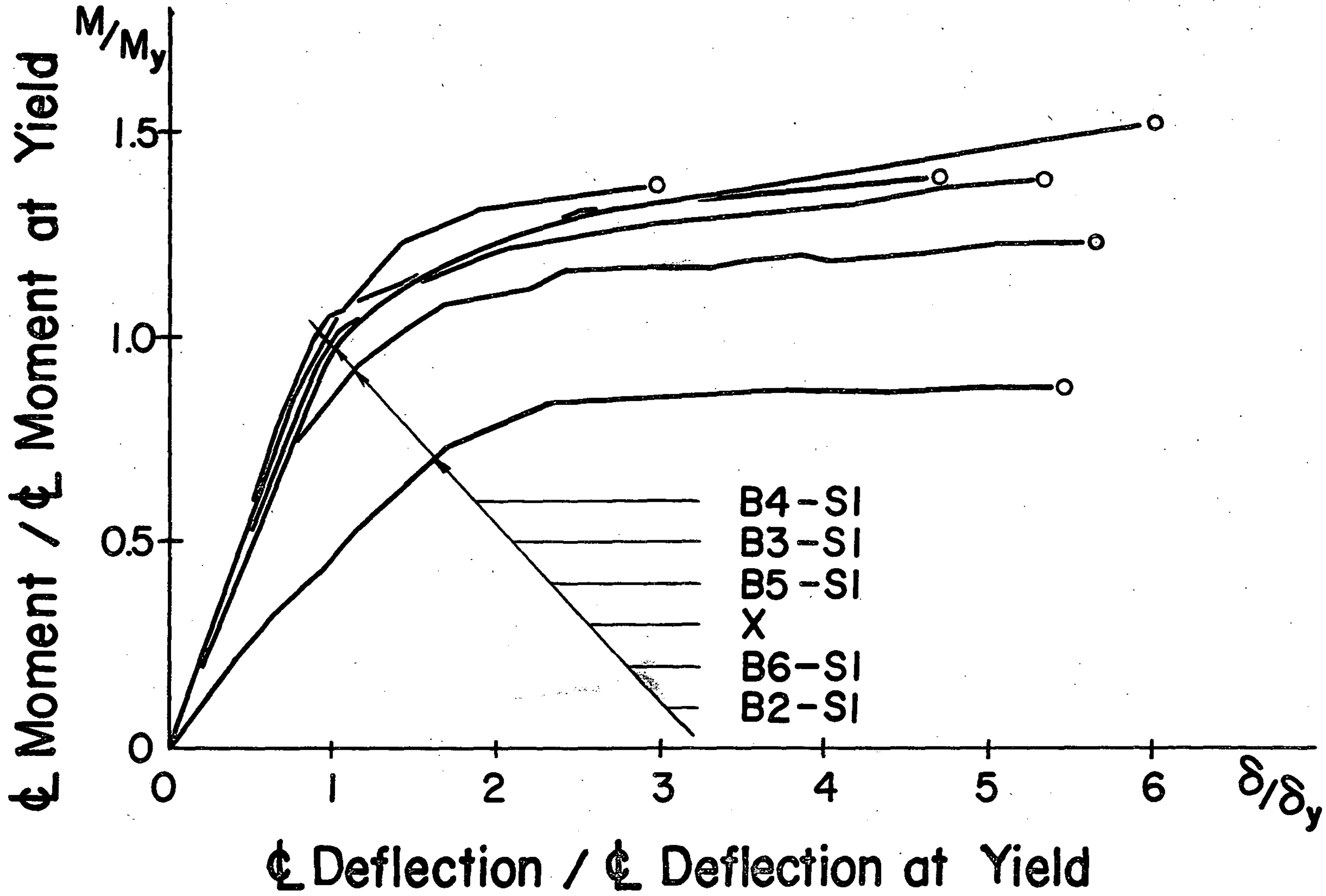


Fig. 27a - Comparison of Beam Tests with First Load Spreading

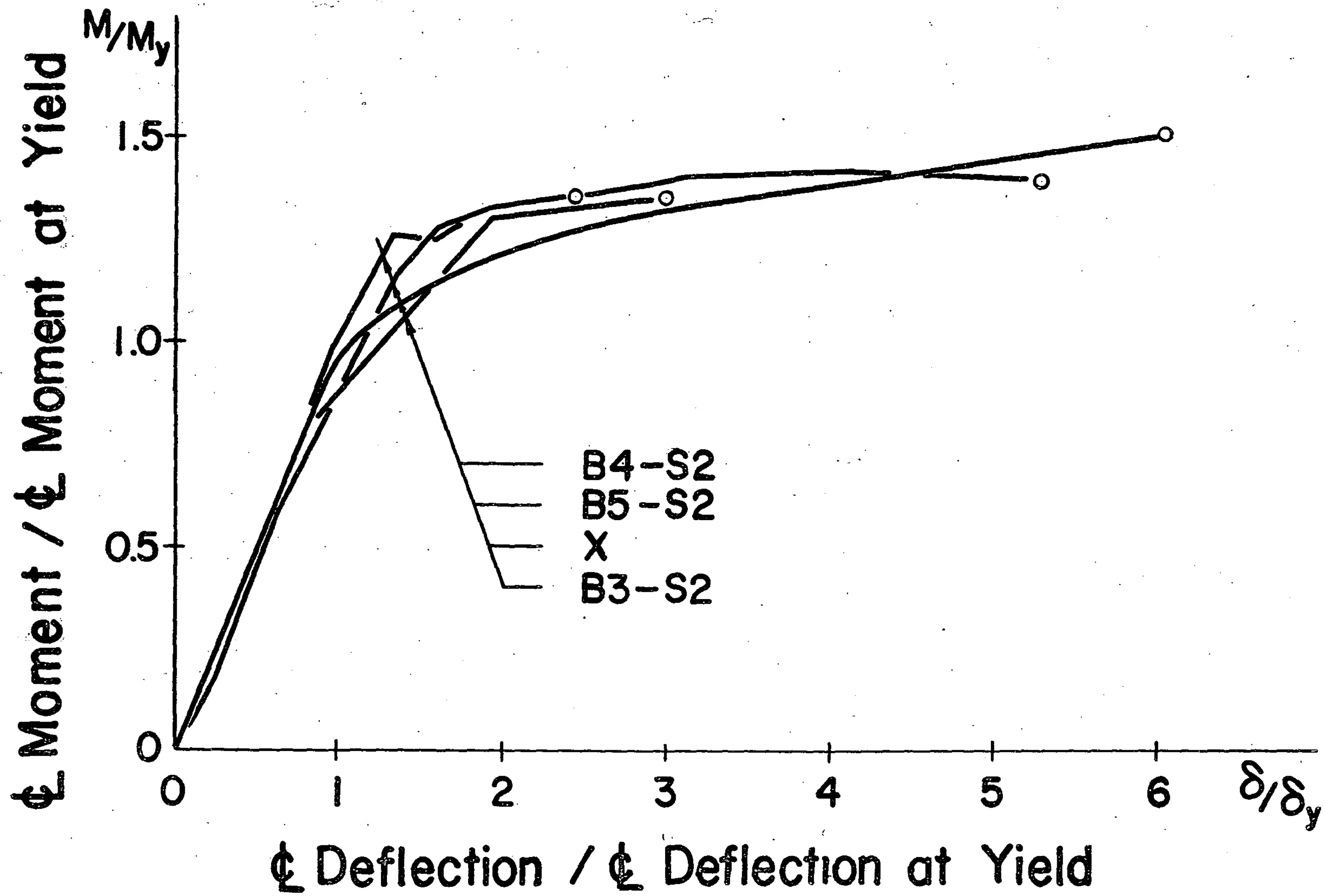


Fig. 27b - Comparison of Beam Tests with Load Spreading
Giving Balanced Shear and Moment

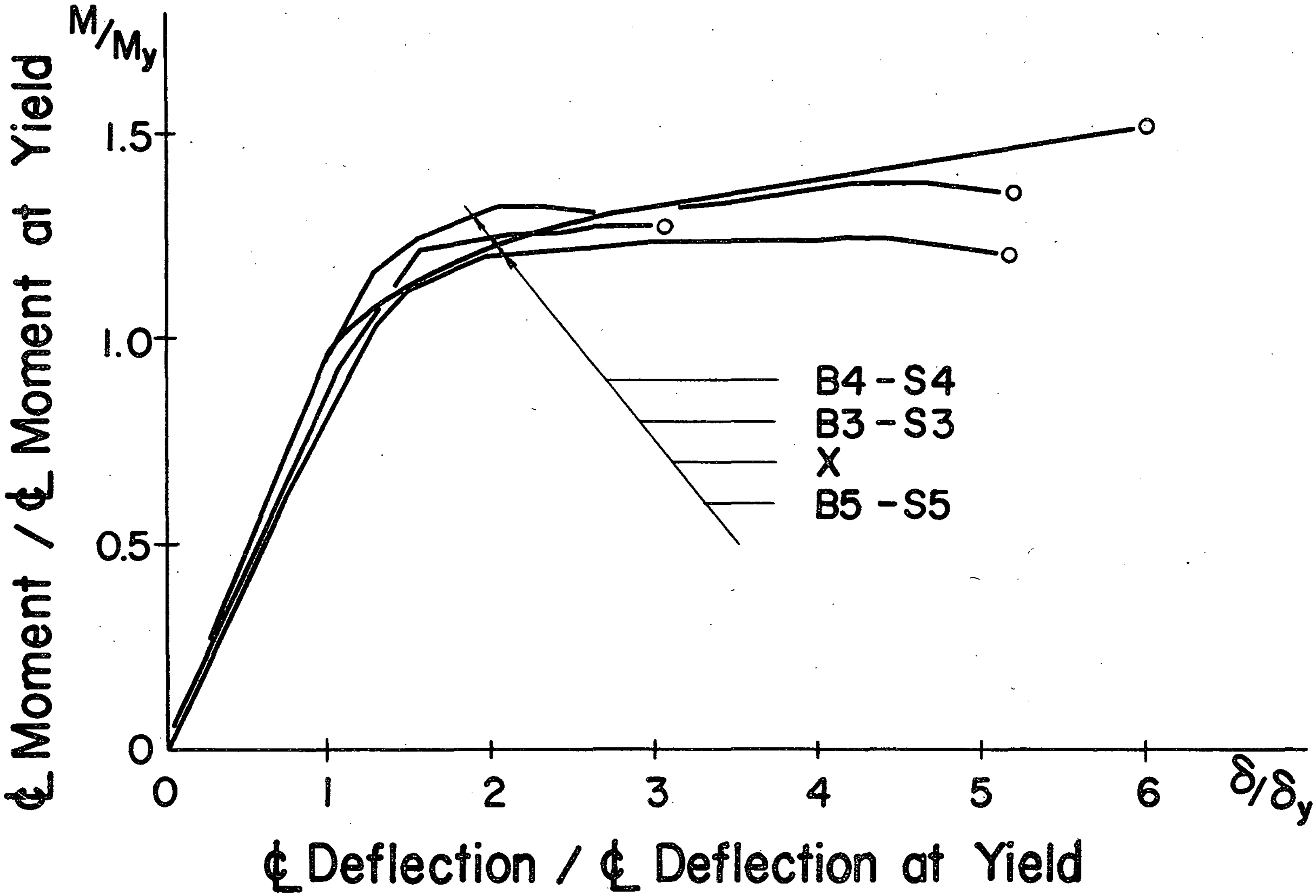


Fig. 27c - Comparison of Beam Tests with Load Spreading Giving Excess Shear

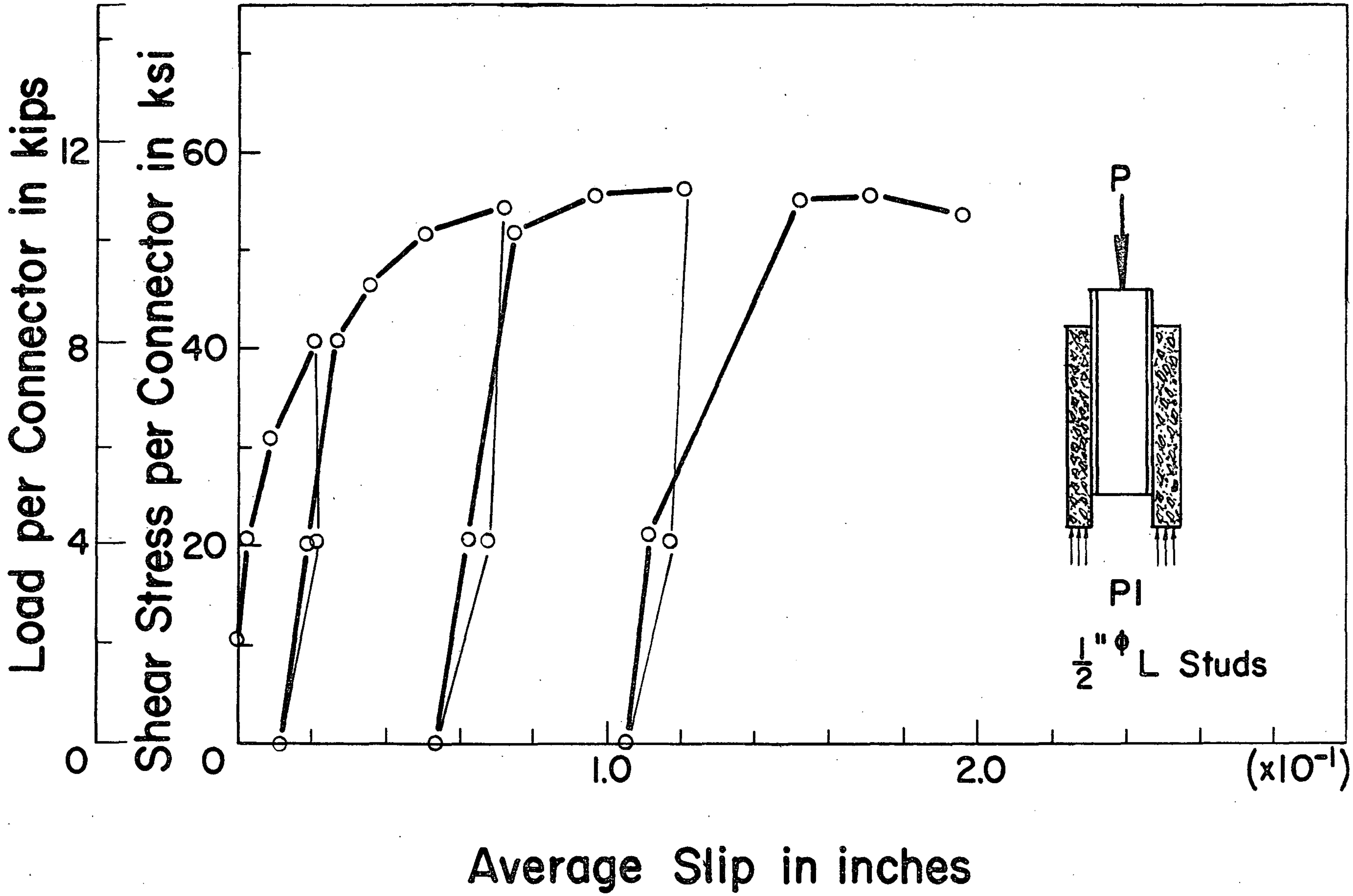


Fig. 28 - Load-Slip Curve for Pushout Specimen P1

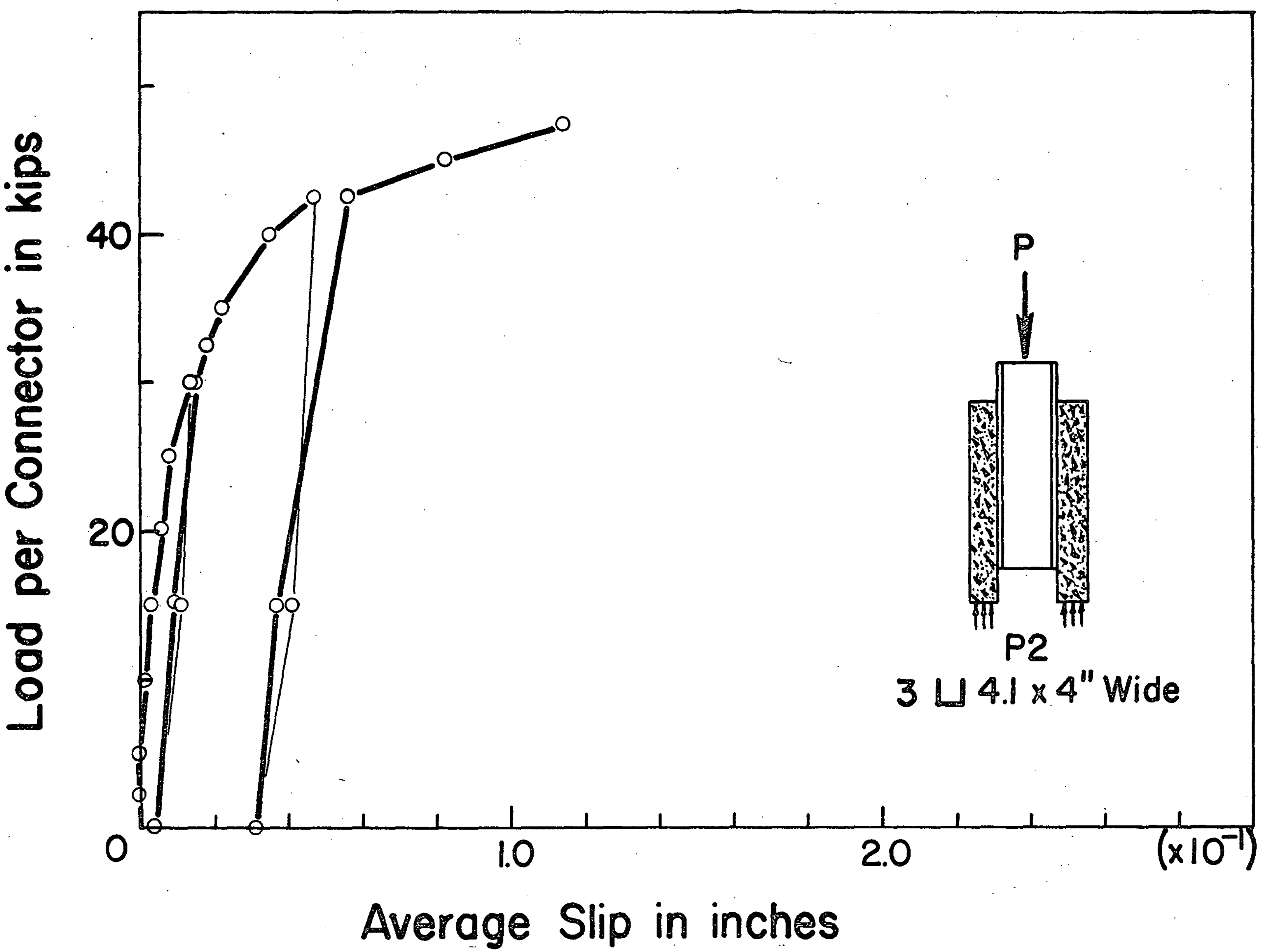


Fig. 29 - Load-Slip Curve for Pushout Specimen P2

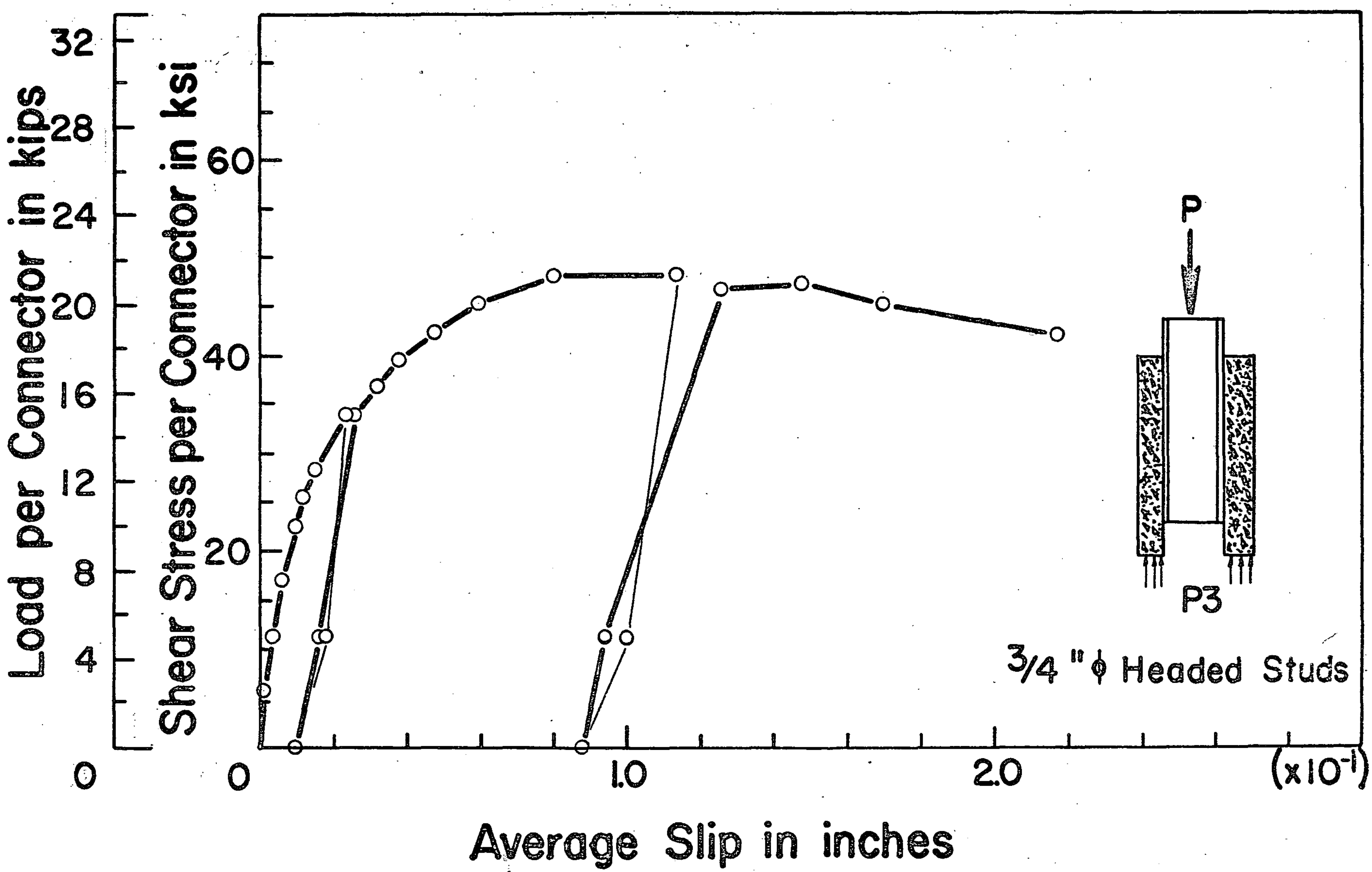


Fig. 30 - Load-Slip Curve for Pushout Specimen P3

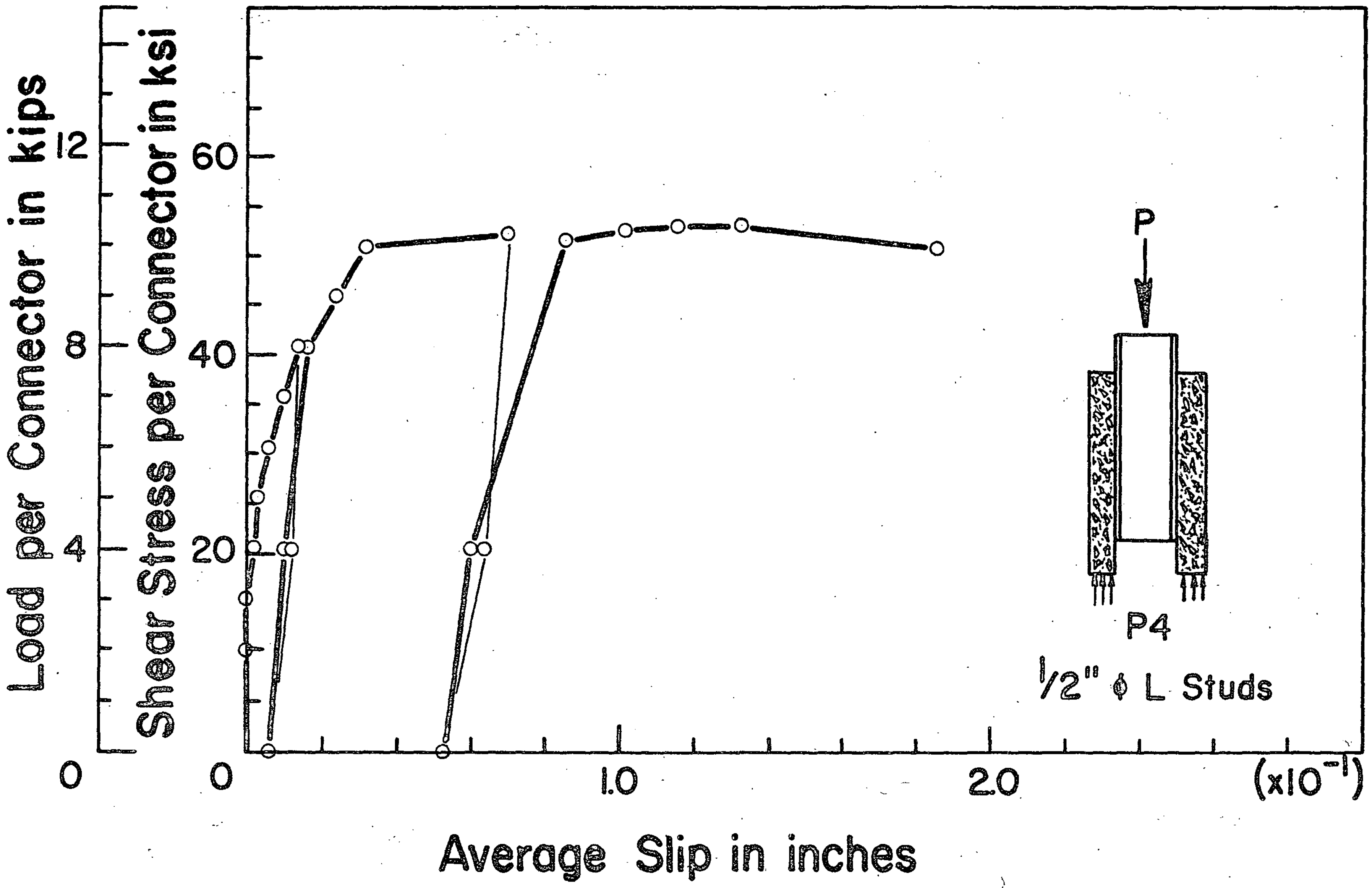


Fig. 31 - Load-Slip Curve for Pushout Specimen P4

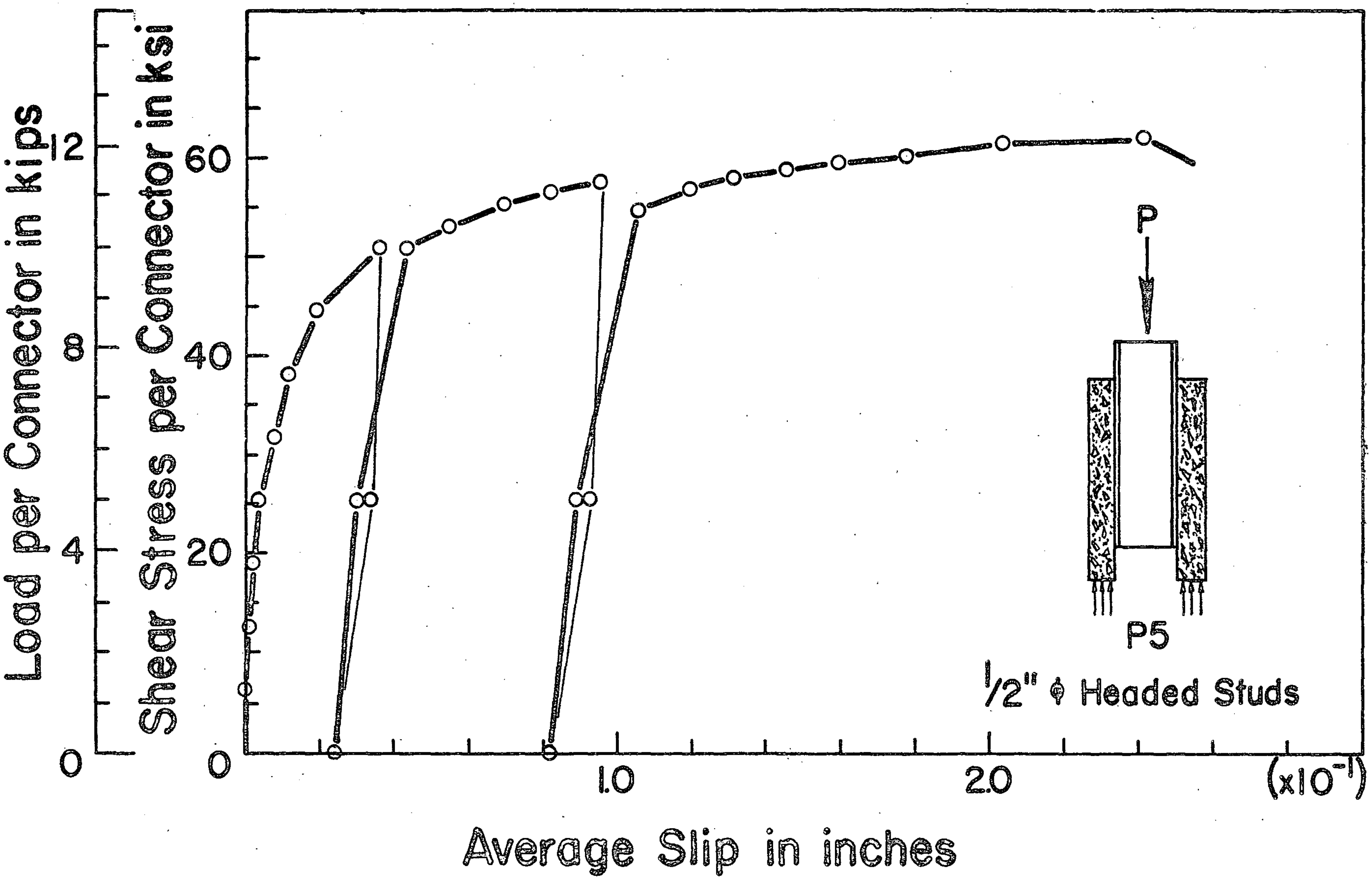


Fig. 32 - Load-Slip Curve for Pushout Specimen P5

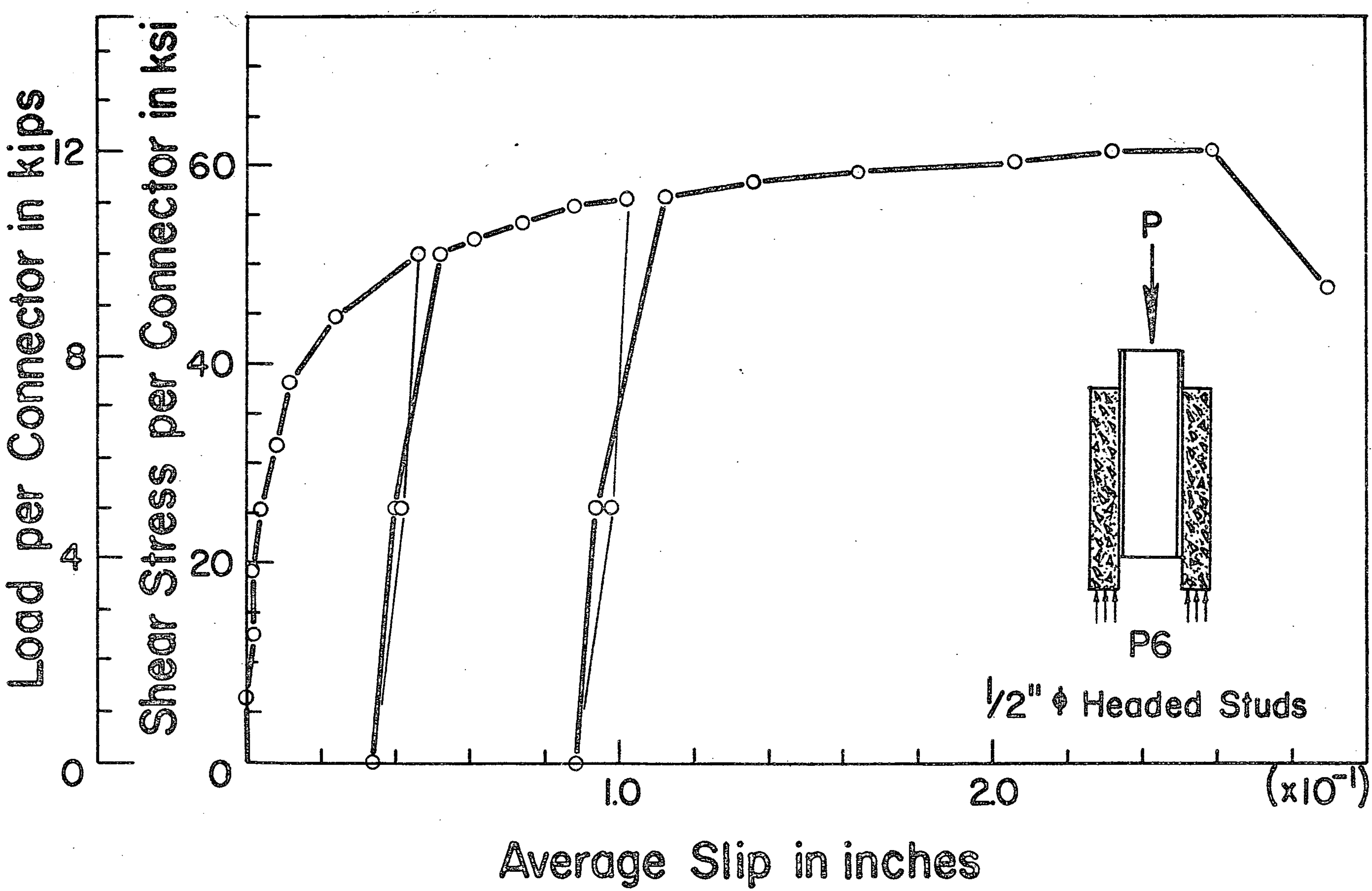


Fig. 33 - Load-Slip Curve for Pushout Specimen P6

Fig. 34 - Crushed Concrete Around 1/2" L-Studs in Pushout Specimen P1.

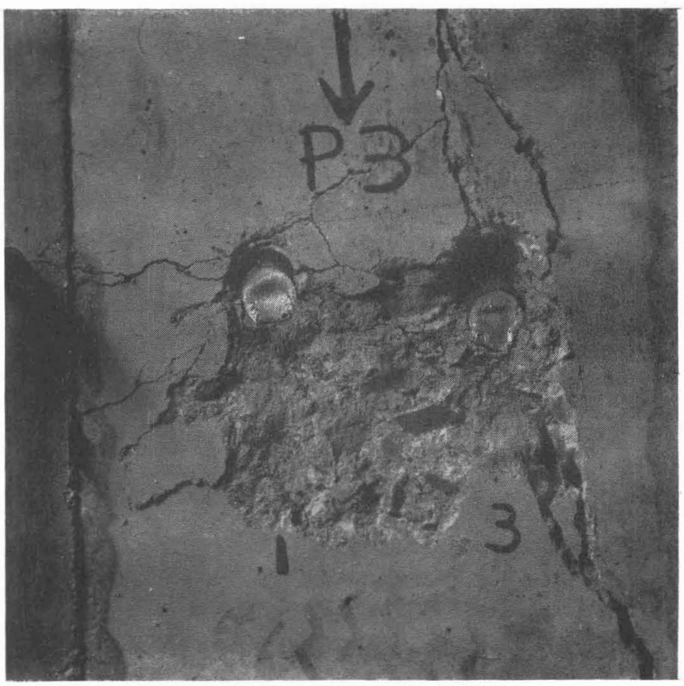


Fig. 35 - Crushed Concrete Around 3/4" Headed Studs in Pushout Specimen P3.

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