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Tests of composite beams for buildings, January 1960

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COMPOSITE DESIGN FOR BUILDINGS

Progress Report No. 2

TESTS OF COMPOSITE BEAMS FOR BUILDINGS

by

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Report on a research project sponsored
by the American Institute of Steel Construction

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ABSTRACT

This report is a supplement to Progress Report No. 1 "TESTS OF COMPOSITE BEAMS FOR BUILDINGS". Additional tests reported herein were designed to answer questions arising from the results of the tests in Progress Report No. 1. Also it was intended to obtain information concerning the distribution and spacing of shear devices along a beam and the feasibility of combining composite construction and plastic design for continuous beams.

Three composite beams were tested in order to compare the behavior of 1/2" and 3/4" diameter headed studs and 1/2" diameter L shaped studs. These tests were included after comparison of the results of the pushout tests on these three types of studs reported in Progress Report No. 1. Three composite beams were tested to determine the effect of distribution and spacing of shear devices along a beam loaded in such a manner that the shear diagram varied along the length of the member. A continuous beam was tested to establish the feasibility of designing continuous composite beams by means of plastic design.

Information concerning the behavior of composite beams and welded studs 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 background material for this investigation was discussed in the Introduction of Progress Report No. 1. In order to eliminate undue repetition this material will not be discussed here.

This report describes a series of tests designed to provide information on the following problems:

- (1) Strength of stud shear connectors in a beam specimen.
- (2) Influence of slip on the load deflection curve of a composite beam.
- (3) Distribution and spacing of shear devices along a beam.
- (4) The feasibility of designing continuous composite beams on an ultimate basis.

Six simple span isolated composite beam specimens, a continuous beam specimen, and three pushout specimens were tested and are described in this report. Conclusions and design recommendations based on the results of these tests are also included herein.

2. GENERAL DESCRIPTION OF TEST SERIES

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 which might be encountered in ordinary buildings.

Three beams, B7, B8, and B9 were included to obtain data on three different types of studs. One-half inch diameter L studs were used in beam B7, one-half inch diameter headed studs in B8, and three-quarter inch diameter headed studs in B9. These three beams were exactly alike in all other respects and therefore a comparison of the results of these tests is in effect a comparison of the behavior of the three types of shear connectors.

Beams B10, B11, and B12 were included to obtain data on the effect of connector spacing on the behavior of the composite section. These three beams were subjected to loads (five equal loads spaced at the sixth points of the beam) which produced a varying shear diagram. The total number of shear connectors provided in each beam was the same. However, in beams B10 and B11, a uniform connector spacing along the length of the beam was used, whereas in beam B12 the shear connectors were spaced according to the proportions of the shear diagram.

Beam B13 was a continuous beam and was designed plastically in order to evaluate the behavior of a continuous composite beam.

Three pushout specimens P7, P8, and P9 were included in this series of tests. The three types of studs, 1/2 in. diameter straight, 1/2 in. diameter L, and 3/4 in. diameter straight studs were used in these specimens. By comparing the performance of these pushout specimens the relative strengths and behavior of the three types of studs can be evaluated and since the same types of studs were used in beams B7, B8, and B9 a comparison between beam and pushout test can also be obtained.

3. DESIGN AND FABRICATION OF TEST SPECIMENS

3.1 Beam Specimens

All the specimens were designed on an ultimate basis in order to evaluate the feasibility of designing composite beams on this basis. Information concerning the elastic behavior of the composite section and consequently the feasibility of an elastic design could 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 horizontal shearing forces is possible. According to this assumption each shear connector is carrying the same shear force at ultimate load.

Design values for the connector forces which would permit the section to develop the ultimate moment prior to connector failure were obtained from the previous tests reported in Progress Report 1 (1). A value of 16 kips per connector was used for the 1/2 in. L studs. Due to the fact that there was no beam test data available for 3/4" diameter headed studs it was necessary to extrapolate the data from Progress Report 1 for these studs. It was assumed that the ultimate force which a connector can develop is proportional to the cross sectional area of the stud. By multiplying the ultimate connector force for a 1/2" diameter stud by the ratio of the area of a 3/4" to the area of a 1/2" diameter

stud an ultimate connector force of 36 kips was obtained for a $3/4$ " stud. 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 $1/4$ in. diameter rods placed one inch below the top of the slab. Additional reinforcing in the form of $5/8$ in. diameter bars placed in the transverse direction on 6 in. centers was used. This additional transverse reinforcing was provided only in the vicinity of the ultimate moment and its purpose was to prevent longitudinal cracking of the slab by the transverse bending moments which develop in the slab near the plastic moment as a result of the large deformations occurring at this point. One-half in. diameter L shaped studs, one-half in. straight studs, and $3/4$ in. diameter straight studs were used in the various beams for the shear connection. Figs. 1, 2, and 3 give the specimen dimensions and the connector spacings.

The stud shear connectors were of the solid flux type attached to the steel beams by a conventional stud welding process. The $1/2$ in. L and $3/4$ in. straight studs were manufactured by ordinary methods of stud production. It was desirable that the $1/2$ in. L studs and $1/2$ in. headed studs be manufactured from identical material. However, studs of these two types from the same material were not

available from the manufacturer. Due to the high cost of producing studs in small quantities an alternate method of obtaining 1/2 in. straight studs was used. Instead of using the conventional heading technique, an enlarged head was welded to a straight 1/2 in. bar of the same material as used for the 1/2" L studs. Since most of the deformation due to load of a stud takes place near the base of the connector this weld should not alter the behavior of these studs from those which might be produced by the conventional heading technique.

The steel beams for beams B7, B8, and B9 were from the same rolling and the concrete for these three specimens was from one mix. The steel beams for beams B10, B11, B12, and B13 were also from one rolling but not the same as that for B7, B8, and B9. Again, the concrete for beams B10, B11, B12, and B13 was from one mix. All concrete used was of the commercial ready-mix type with a maximum aggregate size of 3/4 in. The material used for the 1/2 in. L and headed studs in all the beams (B7 through B13) was from the same bar stock. By keeping the physical properties of the materials constant the only variable was the type shear connection or connector spacing and comparison of the test results was facilitated.

3.2 Pushout Specimens

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 used for these tests.

The pushout specimens (P7, P8, P9) were cast from the same mix as the beam specimens B7, B8, and B9. The types of connector used in these pushout specimens were the same as those in the respective beam tests of the same number. (P7-B7, P8-B8, P9-B9). The dimensions of the pushout specimens and the connectors are given in Fig. 4.

All pushout specimens were cast in an inverted position from that of testing in order to eliminate the possibility of voids forming in the concrete on the underside of the connectors.

4. TEST PROCEDURE

4.1 Beam Tests

Essentially three types of beam tests were used and they will be discussed individually. Beams B7, B8, and B9 can be grouped in the first category, Beams B10, B11, and B12 in the second and Beam B13 in the third category.

4.1a Beams B7, B8, B9

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. Load was applied to the top of the slab in all cases as shown in Fig. 5.

Load was applied to the specimens by means of a hydraulic jack. 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.

Beams B7 and B8 (1/2 inch L and 1/2 inch straight studs) 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 these two specimens. 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 for a third test were such that connector failure would occur prior to reaching the ultimate moment.

Beam B3 which was included in Progress Report 1 and beam B7 were essentially the same. Since B3 was tested using the full range of the three load spacings and connector failure occurred under the third load spacing prior to reaching the ultimate moment, it was decided to carry the second test of beam B7 to failure. Under this test, connector

failure and crushing of the concrete should have occurred simultaneously i.e., this test provided a balanced design for shear connectors and moment capacity.

No previous beam tests had been conducted on 1/2 inch straight studs and therefore connector failure was desired so as to evaluate their strength in a beam. For this reason all three load spacings were used for beam B8 in order to insure connector failure.

At the outset of these tests the strength of a 3/4 in. straight stud in a beam specimen was not known. In designing specimen B9 it was assumed that the strength of various studs is proportional to the shear area of the stud. Knowing the strength of a 1/2 inch stud from previous tests (Progress Report 1) an estimate of the strength of a 3/4 inch stud was obtained by multiplying this strength by the ratio of the area of a 3/4 to the area of a 1/2 inch stud (See Appendix). Three load spacings were chosen for this beam in the same manner as described above so that connector failure would occur under the third load spacing.

Strains in the concrete slab were used to determine the point at which each of the first two tests should be stopped. A previous test ⁽²⁾ indicated that crushing of the concrete occurred when the strains in the slab reached approximately 0.0039 in/in. In the current tests, when the strains in the slab reached approximately 0.00275 in/in, the test was

stopped if the slip measurements did not indicate that connector failure was impending.

The load was applied to the specimens in various increments up to approximately $P_p/1.85^*$. This load was then 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.

The instrumentation used for these three 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

* A load value of $P_p/1.85$ was selected because this was expected to be 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.

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. 8.

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.

4.1b Beams B10, B11, B12

The specimens were simply supported over a span of 15 feet and loaded with five concentrated loads equally spaced along the length of the member as shown in Fig. 6. This loading produced moment and shear diagrams closely approximating those for a beam subjected to a uniformly distributed load.

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

In these three beam tests the method of increasing the load spacing in order to guarantee connector failure as described for beams B7, B8 and B9 was not feasible. For this reason the beams were designed so that shear connector failure and crushing of the concrete would occur simultaneously under the loading shown in Fig. 6.

The loading procedure followed for these three specimens was essentially the same as that for B7, B8, and B9. The only difference in this case was the fact that the load spacing was not changed.

Instrumentation was also the same except for changes in the location of the slip and uplift dials.

4.1c Beam B13

This specimen was a 30 foot, two span continuous beam. The test setup is shown in Fig. 7. In order to determine the effect of various loading conditions for a continuous beam, a loading procedure was used in which alternate spans were loaded. After completing these preliminary tests, both spans were loaded in order to determine the maximum carrying capacity of the section. An outline of the loading procedure used is given in Table 7.

The load was applied to this specimen and measured in the same manner as for the other specimens.

The instrumentation for this specimen included strain readings in the steel beam and concrete slab, deflections, and slip and uplift readings. In addition, the plastic hinge rotation or slope of the beam over the center support was determined by means of level bars located over the center support. In order to check the load application to the specimen, dynamometers as shown in Fig. 7 were used to measure the center reaction. These readings provided a check as to whether the loading was applied properly.

4.2 Pushout Tests

The test setup for the pushout specimens is shown in Fig. 9. 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 is 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. 4. 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, tension tests and shear tests were performed on the shear connector material. The results of these auxiliary tests appear in Tables 3 through 6.

5. RESULTS AND DISCUSSION

5.1 Beam Tests

A summary of the results of the beam tests appears in Tables 8, 9, and 10 and the load-deflection curves are given in Figs. 10 to 21. 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.

Beams B7, B8, B9

These three beams were similar in every detail except for the shear connection. One half inch L, 1/2" headed, and 3/4" headed studs were used in Beams B7, B8, and B9 respectively. A summary of the pertinent results is given in Table 8.

The load deflection curves for B7, B8, and B9 given in Figs. 10 through 17 are all of the gradually ascending type indicating good plastic behavior even though the computed connector forces are large. The graphs for the second and third tests of each specimen show the same elastic type behavior at lower loads as the initial tests despite the presence of large residual deflections from these first tests. All the beams were again able to carry load well into the plastic range with even larger connector forces than before.

The failure of beam B7 was a flexure failure due to crushing of the concrete slab near midspan. Connector failure ensued immediately after this crushing of the slab. Beam B8 failed by shearing of the connectors and beam B9 failed due to inability to carry additional load. In the case of Beam B9 with $3/4$ in headed studs, localized cracking around the connectors was noted near the ends of the specimen prior to failure as shown in Fig. 33.

The slip distributions plotted in figs. 27, 28, and 29 for the three specimens indicate that somewhat larger slips occurred in beam B9 with $3/4$ " studs than in the other two beams with $1/2$ " studs. The separation between slab and beam for the three beam tests is plotted in Fig. 25. This figure indicates that the separation was somewhat lower for the beams with headed studs than for beam B7 with L studs. Despite this difference all the separations recorded were small.

A comparison of the load deflection characteristics of the three specimens is provided in Figs. 34a, 34b, and 34c. On the basis of the comparison made in these three figures the behavior of all three beams was quite similar. Thus it would appear that the type of stud used, either 1/2"L, 1/2" headed, or 3/4" headed does not influence the overall behavior of a beam specimen.

Beams B10, B11, B12

The loading used for these three beam specimens produced shear and moment diagrams closely approximating those for a beam subjected to a uniform load. A constant shear connector spacing was used for beams B10 and B11 whereas a variable connector spacing was used for beam B12. The results of these tests are summarized in Table 9.

Failure in each beam test was caused by connector failure at a moment somewhat below the predicted ultimate moment. The connector forces at failure of the connectors in all three beams were nearly equal. The strain distributions at midspan in each of the three specimens were also similar. It is significant to note, however, that there was a difference between the three beams with respect to slip and uplift between the slab and beam.

The slip distribution pattern along the three beams was similar. However, for beams B10 and B11 with uniform connector spacing, it was of a greater magnitude at several locations along the beam. The distribution of uplift along the beams was different as can be noted from Fig. 26. For beams B10 and B11 with uniform connector spacing the separation between slab and beam was larger near the center of the beam while for beam B12 with a variable connector spacing the separation was larger near the ends.

A comparison of the overall behavior of the three specimens is provided in the nondimensional plot of Fig. 35. This figure indicates that the constant or variable connector spacing had little influence on the overall behavior of the specimens.

Beam B13

The load deflection curve for B13 with both spans loaded is shown in Fig. 21. In this figure the deflection in each span is plotted against the total load in each span. In the elastic range, the deflections in both spans are very nearly equal. It will be noted, however, that in the inelastic range the deflections in the east span were greater than those in the west span. The slips in the east span were also greater than those in the west span over this range of loads. The theoretical deflection curve was computed using the

moment of inertia of the uncracked section and the test results are in fair agreement with these values despite the fact that the slab did crack in the negative moment region.

The maximum separation recorded between slab and beam with both spans loaded was approximately 0.045" and occurred near the end of the east span. The maximum slip recorded for this loading was 0.135".

The moment curvature relations over the center support plotted in Fig. 32 were in good agreement with predicted values. The moments plotted as the ordinate in this figure were computed from the center reaction which was measured by means of the dynamometer at the center support.

The behavior of the slab and beam in the negative moment region is of primary importance in evaluating the performance of this continuous beam. The reinforcement in the longitudinal direction over this section consisted of the 6"x6"x1/4 in. mesh used throughout the positive moment region. Additional reinforcement consisting of No.5 bars on 6" centers was used in the transverse direction. The longitudinal reinforcement was 0.2% of the cross sectional area of the concrete slab.

In the preliminary tests in which alternate spans were loaded only one crack developed in the slab in each span. These cracks developed separately and occurred in the unloaded span. (East span loaded - crack developed 29" from

center support in west span, west span loaded - crack developed 19" from center support in east span). The stresses in the concrete at the locations of the cracks were 774 psi and 748 psi respectively at the instant of crack formation. These stresses were computed by assuming that the entire cross section was effective in resisting bending. For the case of the first crack which formed in the west span this assumption was valid since up to this point the entire slab was uncracked. When the alternate span was loaded (west span) the moment of inertia in the vicinity of the crack which had formed in the west span in the previous test was not that of the uncracked section and the assumption made is not strictly correct. Both cracks were approximately $1/32$ " wide on the top of the slab and progressed through the full depth of the slab. As the beam was unloaded the cracks closed but were still visible with the naked eye after the specimen was unloaded.

In the final test of this continuous beam, both spans were loaded and the slab cracked directly over the center support. The stress in the slab at the location of the crack when the crack developed was approximately 1000 psi. Point ① on the load deflection curve of Fig. 21 indicates the load at which this crack formed. The crack width at this load was less than $1/32$ " wide. When the specimen was unloaded this crack closed but it was still visible to the naked eye. As the beam was again loaded this crack began

to open and the points marked ②, ③, and ④ on Fig. 21 mark the loads at which the crack width was 1/16", 3/16" and 5/8" wide respectively. This crack over the center support was the only one which formed when both spans were loaded.

As the maximum load was reached the slab at the location of the crack over the center support began to twist. The test was stopped at this point despite the fact that there was no indication of connector failure. The load deflection curve indicated that the load was leveling off and further increase in load prior to connector failure was doubtful. The load at this point had reached 99% of the theoretical plastic load. The connector forces in the positive moment region were 15.5 kips per connector at this maximum load. Upon completion of testing the slab was removed from the beam. All the connectors in this beam were intact, the deformed shape of the connectors being given in Fig. 24.

5.2 General Results of Beam Tests

The strain measurements taken across the top of the slab at the centerline, Fig. 22, 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. 23 and 24.

In all 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 $10xP^k$. It will be noted that these load repetitions had no adverse effect on the specimens.

The computed connector forces at failure for Beam B7 were somewhat smaller than those for the same type of stud in previous tests.⁽¹⁾ Connector forces at failure for the 1/2" headed studs in B8 were of the same order of magnitude as those for 1/2" L studs in previous tests. The value of 33.8 kips per connector computed from Beam B9 for 3/4" headed studs was somewhat less than the value predicted according to the assumption that the strength of a connector is proportional to its cross sectional area. The studs in Beam B9, however, did not shear off as was the case with the 1/2" diameter studs.

The values obtained in Table 9 for the connector forces in Beams B10, B11, and B12 were all less than values obtained for the case of a beam subjected to two point loading. Since the loading was the only significant difference between the two types of specimen, it would appear that the manner of loading has an effect on the connector forces or on the validity of the assumptions made in the analysis used in this report.

5.3 Pushout Tests

A summary of the results of the pushout tests is given in Table 11 and the load slip curves for the three specimens appear in Fig. 36 through 38. 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 11 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 P7 and P8 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 specimen P9 a considerable amount of cracking of the slab occurred as the load on the specimen reached its maximum value.

A comparison of the ultimate connector forces in Table 11 and the load slip curves for P7 and P9 with the results of previous tests ⁽¹⁾ indicate that both the strength and deformation characteristics of the studs in P7 and P9 were considerably different from previous test results. Since the pushout specimens were essentially the same in both the present and previous tests, and the stud material was of comparable quality there is no obvious explanation for these differences. One possible explanation could be faulty alignment of the specimen during testing. Faulty alignment if any, was not apparent. It is felt that in the light of the considerable differences observed in the results of specimens P7 and P9 that they be neglected and the tests considered unsuccessful.

The load slip curve for specimen P8 is similar to those obtained in previous tests with the same type of stud.⁽¹⁾ The ultimate load reached in these tests was of the same magnitude as that recorded in the previous tests, but a comparison of the load slip curves of P8 and P5 and P6⁽¹⁾ indicate that the studs in P8 were more flexible. The curve for P8 does not rise as steeply as that for P5 and P6.

5.4 Comparison of Beam Tests and Pushout Tests

The tests of P7 and P9 were considered unsuccessful, therefore a comparison with the beam test results is not feasible. It was observed that the maximum slip at failure for specimen P8 was different from those observed in beam test B8 which had the same type of studs. The ultimate connector forces were also considerably different. These tests further substantiate the conclusion that the behavior of a shear connector in a pushout specimen is different from that in a beam specimen.

5.5 Comparison of Beam Tests and Previous Test Results

The value of 13.7 kips per connector obtained for beam B7 is somewhat lower than that of 15.8 kips obtained in Progress Report 1 for a similar test. This is due to the fact that crushing of the concrete occurred in beam B7 before the ultimate moment was reached. Beam B7 was designed so that a balanced design would result under the second load spreading,

i.e. the section should have reached the ultimate moment at the instant of connector failure. The fact that this beam did not reach the predicted ultimate moment and the connector forces were somewhat lower than in previous tests would seem to indicate that slip or incomplete interaction tends to reduce the carrying capacity of a composite beam. The beam did reach 88.7% of the ultimate moment.

There were no previous beam tests with which to compare beams B8 and B9.

The connector forces obtained for beams B10, B11, and B12 were all very close but were less than values obtained for beams with a different type of loading in Progress Report 1. This would indicate that the external loading influences the behavior of the shear connection or the validity of the design approach used in this report.

Since beam B13 was the first continuous composite concrete steel beam tested with 1/2" L studs there was no previous test data with which to compare the results.

6. CONCLUSIONS

The following conclusions were drawn from the tests discussed in this report:

1. The overall behavior of similar composite beams with the different types of stud shear connectors used is about the same. (1/2" L, 1/2" headed or 3/4" headed) - See nondimensional graphs in Figs. 34a, 34b, and 34c.
2. The strength of the stud shear connectors tested is very nearly proportional to the cross sectional area of the stud.
3. The bearing area of the stud (diameter) or the transverse spacing of the studs has an effect on the mode of failure of the connector and possibly its ultimate strength. Comparison of the manner of failure of beam B9 with that of beam B7 tends to indicate that the mode of failure is dependent on the size of the connector. For beam B9 with 3/4" dia. studs and beam B5* with channel connectors the failure was localized and in the vicinity of the connectors. This localized failure resulted in failure of the entire specimen since the beams were unable to carry additional load. (B7 shearing of studs vs B9 crushing of concrete around the studs).

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* Progress Report 1

4. The resistance to separation of slab and beam, uplift, provided by a headed stud is somewhat better than that provided by an L stud.
5. The overall performance of the composite section with a uniform shear connector spacing over regions of varying external shear was about the same as the behavior for the case of a variable shear connector spacing.
6. The strength of a shear connector obtained in a push-out specimen is different from that in a beam specimen. The connector force at failure in a pushout specimen was approximately 39% lower than the connector force at failure obtained in a beam specimen (P8 vs B8)
7. DESIGN RECOMMENDATIONS

The results of all the beam tests in this investigation indicate that plastic design of composite beams is feasible. In view of the economies and advantages of this method it is recommended that composite beams be designed on this basis.

In plastic design the composite section would be designed for the ultimate load. This load P_p would be computed by multiplying the working load P_w by a suitable factor of safety. After choosing the steel section and slab thickness and width, it would remain to design the shear connection.

A balanced design or one in which the factor of safety for connector failure is the same as that for flexural failure is the most reasonable. Certainly a weak shear connection is undesirable and the use of a higher factor of safety for the shear connection is unwarranted since this will not add to the carrying capacity of the section.

From the standpoint of the overall behavior of the composite section the designer may specify any of the three types of studs (1/2L, 1/2" headed, 3/4" headed). The ultimate strength of each of these three types of studs may be considered as proportional to the cross sectional area of the stud.

In designing the shear connection the slab is isolated between sections of zero moment and full plastic moment and equilibrium of this free body considered. The only force acting on this free body is the compressive force in the slab at the location of the plastic moment. The shear connection provided must resist this force and maintain equilibrium. The total number of connectors is determined by dividing the compressive force by a specified force for a single connector. These connectors are then spaced uniformly over this length regardless of the variation of external shear.

Design Values for Shear Connectors and Factor of Safety

An exact failure theory for composite beams with incomplete interaction is non-existent. In view of this fact

design recommendations must be made in the light of test results. There is probably incorporated in these test results as in any other test results what is called "experimental scatter". For this reason the test results must be carefully scrutinized. For instance, it might be asked whether the connector forces in beams B10, B11, and B12 were lower than in beam tests with a different type of loading due to the effect of the external loading on the beam, or due to experimental scatter. Also the decrease in the plastic moment due to incomplete interaction must be accounted for.

Two possibilities exist with regard to solving the problems posed above. First, further testing could be carried out to determine the exact influence of loading on the connector strength and to eliminate experimental scatter. Second, the factor of safety and the ultimate connector force can be adjusted to compensate for these effects. The authors chose to follow the second course in the design recommendations proposed herein. By increasing the factor of safety from 1.85 as is presently used in plastic design of steel beams to 2.0 the decrease in the plastic moment due to incomplete interaction may be compensated.

The ultimate connector force to be used in design must be determined from the test values in this investigation.

An average of all the test results, an average of only those test results in which the connectors were subjected to uniform external shear, or the lowest value for the failure load of a connector might be used for a design value. There is a difference of 5% between the lowest connector force of 13.8 kips per connector and the value of 14.3 kips per connector which is the average of all the test results for 1/2" L studs. Because this difference is small, it was felt that using the average of all the test results was a more realistic approach to the problem.

In view of the above discussion the following design values are recommended:

1. 1/2" L or 1/2" headed studs
 $Q_p = 14.0$ kips/connector
2. 3/4" headed studs
 $Q_p = 31.0$ kips/connector
3. Factor of Safety
F.S. = 2

The use of a single value for the connector strength neglects any influence which the concrete strength may have on connector strength. All the slabs in this investigation had cylinder strengths of approximately 3500 psi. In another report ², however, the concrete strength was around 5500 psi and values obtained for connector strength were of same order of magnitude as those in this investigation. For this

reason, the strength of the shear connection was assumed to be independent of the strength of the slab.

Composite design may be applied to continuous beams. In designing continuous beams on a plastic basis it is recommended that only the steel beam be considered as effective over the negative moment region. In view of the large rotations which must be sustained at the location of the plastic hinges, it seems advisable to provide expansion joints in the slab at these points to provide for this rotation. These expansion joints should eliminate cracking of the slab and confine all slab movement to one location namely the joint. The alternative of providing tension steel in this region requires further study before any recommendations are made on this design approach.

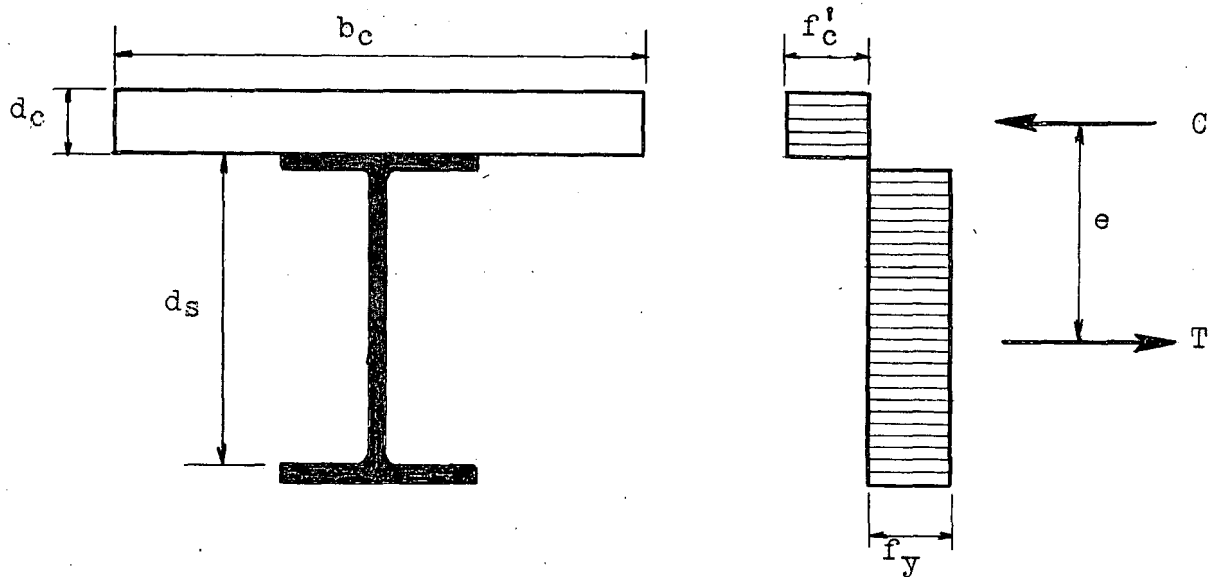
8. ACKNOWLEDGMENTS

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Stud shear connectors for the tests were donated and welded by KSM Products, Inc., of Merchantville, New Jersey.

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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

$f_y(fl_g)$ = 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
- $n = \frac{E_{\text{steel}}}{E_{\text{concrete}}}$
- 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
- S = 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. APPENDIX10.1 Section PropertiesA. Beam Specimens

a. Concrete Slab

$$b_c = 48 \text{ in.}$$

$$d_c = 4 \text{ in.}$$

$$f'_c \begin{cases} = 3300 \text{ psi (B7, B8, B9)} \\ = 3600 \text{ psi (B10, B11, B12, B13)} \end{cases}$$

The values of f'_c listed above are average values of a number of cylinders tested at various ages. All the cylinder test results are given in Table 3.

b. Steel Beam (12WF27)

The steel beams for B7, B8, and B9 were from one rolling. The steel beams for B10, B11, B12, and B13 were also from one rolling but not the same rolling as beams B7, B8, and B9. Measured values were all very close to the handbook properties so the handbook dimensions were used in the calculations.

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

$$d_s = 11.95 \text{ in.}$$

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

$$f_y^{**} \begin{cases} = 37.4 \text{ ksi (flange B7, B8, B9)} \\ = 41.9 \text{ ksi (web B7, B8, B9)} \\ = 36.6 \text{ ksi (flange B10, B11, B12, B13)} \\ = 44.7 \text{ ksi (web B10, B11, B12, B13)} \end{cases}$$

** 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 39.

APPENDIX

c. Connectors

- (1) L studs - B7, B10, B11, B12, B13
 diameter = $1/2$ in.
 height = 2.25 in.
 area = 0.196 in^2
- (2) Headed Studs - B8
 diameter = $1/2$ in.
 height = 3 in.
 area = 0.196 in^2
- (3) Headed Studs - B9
 diameter = $3/4$ in.
 height = 3 in.
 area = 0.441 in^2

d. Composite Section

$$n = 10$$

$$a_{st} = 11.60 \text{ in.}$$

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

$$m = 45.1 \text{ in}^3$$

B. Pushout Specimens

a. Concrete slab

28"x20"x4" - see Fig. 4

$$f'_c = 3063 \text{ psi}$$

reinforcement - mesh 6"x6"x $1/4$ " placed 1"
 from outer face of slab

b. Steel section - 8WF31

c. Connectors

P7 - $1/2$, dia. L studs

P8 = $1/2$ " dia. headed studs

P9 = $3/4$ " 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 (3).

Values of $f'_c = 3500$ psi, $f_y = 38$ ksi, and connector forces of 16 kips/connector for 1/2" diameter studs and 36 kips/connector for 3/4" diameter studs were assumed and used to determine the number and spacing of the connectors.

The design value of 36 kips per connector for 3/4" studs was arrived at by extrapolation of available data for 1/2" studs. This was necessary since no previous beam test data covering 3/4" studs was available. Assuming that the strength of a connector is proportional to its cross sectional area, the value of 36 kips per connector was determined in the following manner:

$$\text{Cross sectional area of 1/2" stud} = 0.196 \text{ in}^2$$

$$\text{" " " " 3/4" " } = 0.441 \text{ in}^2$$

$$\text{Design strength of 1/2" stud} = 16 \text{ kips per connector}$$

$$\text{" " " 3/4" " } = 16 \frac{\text{Area}_{3/4}}{\text{Area}_{1/2}}$$

$$= 16 \frac{0.441}{0.196} = 36 \text{ kips per connector}$$

The design procedure used for the shear connection 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

$$\sigma = \frac{Mc}{I} = \frac{M_{ast}}{I}$$

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

where:

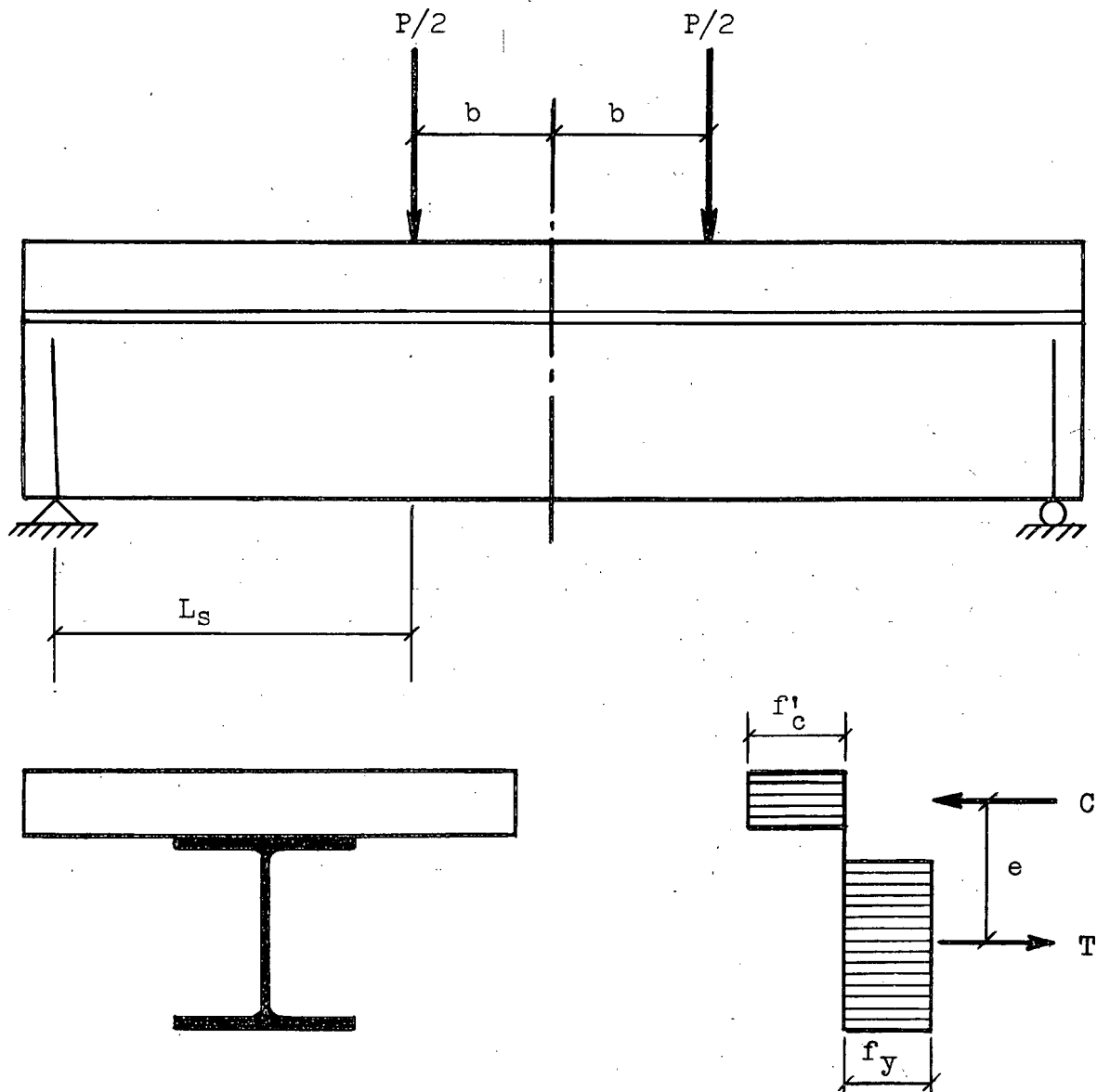
$$f_y \begin{cases} = 37.4 \text{ ksi (B7, B8, B9)} \\ = 36.6 \text{ ksi (B10, B11, B12, B13)} \end{cases}$$

$$c = 11.60 \text{ in.}$$

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

$$M_y = 1895 \text{ k-in. (B7, B8, B9)}$$

$$M_y = 1854 \text{ k-in. (B10, B11, B12, B13)}$$

B. Calculation of the Plastic Moment (M_p)

The proportions of the composite section 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:

$$\begin{aligned} T &= f_y(flg) \cdot 2A_{flg} + f_y(\text{web}) \cdot A_{\text{web}} \\ &= 37.4 (5.29) + 41.9 (2.68) \\ T &= 310 \text{ k (B7, B8, B9)} \\ &= 313 \text{ k (B10, B11, B12, B13)} \end{aligned}$$

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:

$$\begin{aligned} d_p &= \frac{T}{b_c f'_c} \\ &= \frac{310}{48.3.3} \\ &= 1.96 \text{ in.} \end{aligned}$$

The moment arm between the tensile and compressive forces is:

$$\begin{aligned} e &= \frac{d_s}{2} + d_c - \frac{d_p}{2} \\ &= 5.98 + 4 - \frac{1.96}{2} \\ &= 9.00 \text{ in.} \end{aligned}$$

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

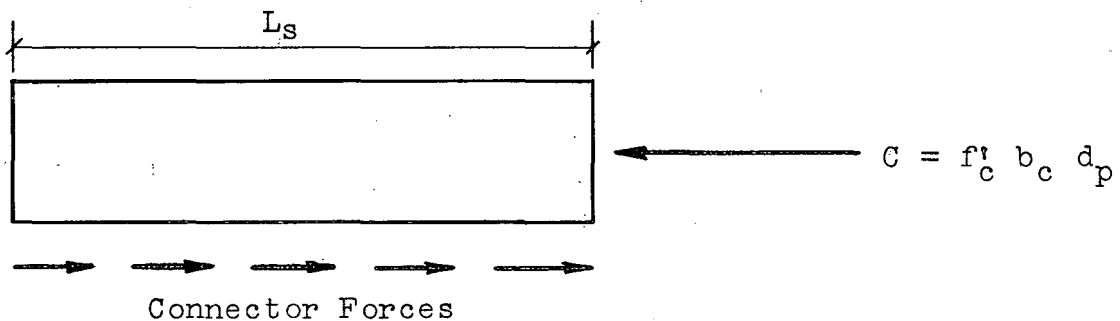
$$\begin{aligned} M_p &= T_e = C_e \\ &= 310 (9.00) \\ &= 2790 \text{ k-in. (B7, B8, B9)} \\ M_p &= 2840 \text{ k-in (B10, B11, B12, B13)} \end{aligned}$$

For beam B13 the plastic moment over the center support was computed neglecting any contribution due to the concrete slab. The plastic moment for the steel beam alone considering the difference in yield stress of the flange and web is:

$$M_p = 1456 \text{ k-in. (B13)}$$

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:

Example

B7-S1

$$C = 310 \text{ k}$$

no. of connectors over length L_s equals 22

$$Q_p = \frac{310}{22}$$

$$= 14.1 \text{ k}$$

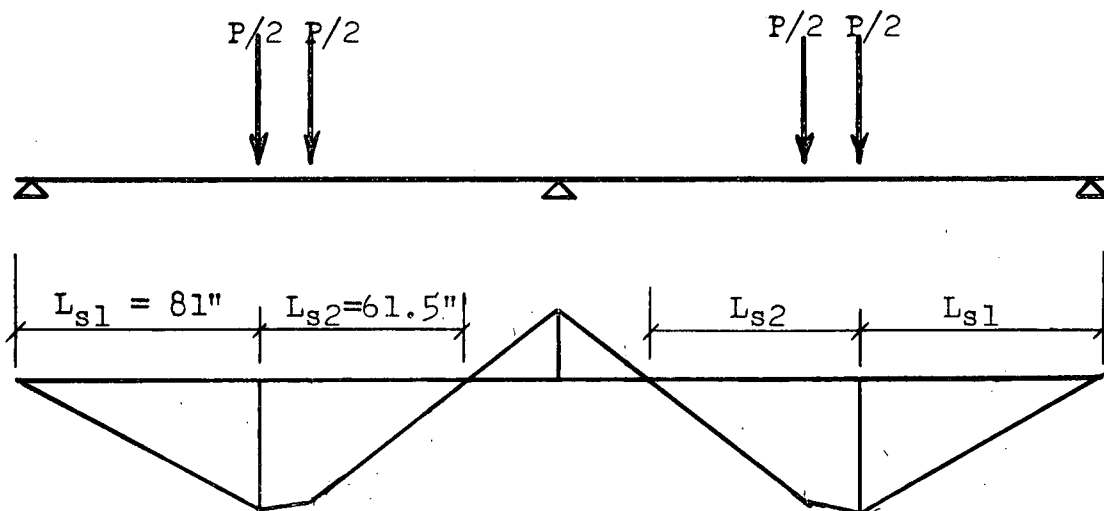
Beam	S1 (2b=18")		S2 (2b=36")		S3, 4, or 5	
	Force per Connector Q_p (kips)	τ ksi	Force per connector Q_p (kips)	τ ksi	Force per connector Q_p (kips)	τ ksi
B7	14.1	72.0	15.5	79.0	-	
B8	14.1	72.0	15.5	79.0	19.4 (2b=66")	99.0
B9	25.8	58.5	-	-	38.8 (2b=72")	88.0

For beams B10, B11, and B12 the length L_s was equal to one half the span length or 90". There were 20 connectors spaced over this length in each beam. The computed connector forces when the section reached the plastic moment were:

Beam	Connector Force Q_p (kips)	τ (ksi)
B10	15.7	80.0
B11	15.7	80.0
B12	15.7	80.0

For beam B13 the value of the connector force was dependent on the loading arrangement, i.e., it depended upon whether one span or both spans were loaded. The connector forces were only computed for the case of both spans loaded at ultimate. For the case of one span loading the connector forces were below these ultimate values. The direction of the force on the connectors in the negative moment region in the unloaded span was different from the direction of the connector force when both spans were loaded.

In computing the connector forces, the length L_s and the number of connectors over this length must be determined. For this continuous beam there were two lengths L_s in each span. The first length (L_{s1}) is the distance from the plastic hinge to the end of the specimen, the second (L_{s2}) is the distance from the plastic hinge to the point of contraflexure.



The computed connector forces for beam B13 were

$$Q_p = 15.65 \text{ k } (L_{s1})$$

$$Q_p = 15.65 \text{ k } (L_{s2})$$

10.4 Deflection Calculations (Theoretical)

1. Due to Bending

B7, B8, B9

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

B10, B11, B12

$$\delta_B = \frac{PL^3}{48EI} + \frac{Pa}{24EI} (3L^2 - 4a^2)$$

B13

The deflections in the elastic range were computed by the unit load method. For the calculations the entire cross section of the composite section was considered effective in resisting bending over the negative moment region.

where

$$L = 15' - 00''$$

$$E = 30 \times 10^3 \text{ ksi}$$

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

$$a = \text{variable}$$

2. Due to Shear

B7, B8, B9

$$\delta_s = \frac{\tau a}{G} = \frac{Pa}{2AwG}$$

where $A_w = 2.68 \text{ in}^2$ (web area of steel beam)

B10, B11, B12

$$\delta_s = \frac{Pa}{2AwG}$$

B13

The shearing deflections were computed by the unit load method.

3. Total Deflection

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

<u>B7, B8, B9</u>	<u>2b=18"</u>	<u>2b=66"</u>	<u>2b=72"</u>
Load (P)	40k	60k	70k
Deflection due to Bending δ_B (in)	0.271	0.341	0.382
Deflection due to shear δ_S (in)	<u>0.052</u>	<u>0.055</u>	<u>0.061</u>
Total Deflection $\delta_B + \delta_S$	0.323	0.396	0.443

B12, B11, B12

Load P	60k
Deflection due to Bending δ_B (in)	0.309
Deflection due to shear δ_S (in)	<u>0.052</u>
Total Deflection $\delta_B + \delta_S$	0.356

B13

Both spans loaded

Load P	40k
Deflection due to Bending δ_B (in)	0.121
Deflection due to Shear δ_S (in)	<u>0.060</u>
Total Deflection $\delta_B + \delta_S$	0.181

10.5 Analysis of Test ResultsA. 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 maximum moment reached in testing to the calculated plastic moment $\frac{M_u}{M_p}$ $Q_p = Q_F$

Example

B7-S1

$$M_u = 2430$$

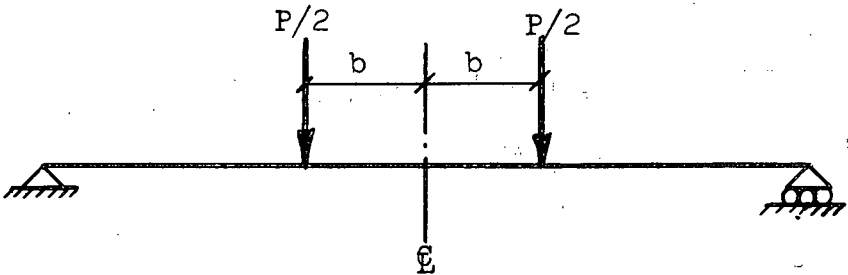
$$M_p = 2790 \text{ k in}$$

$$Q_p = 14.1 \text{ k}$$

$$Q_F = \frac{2430}{2790} 14.1 = 12.3 \text{ k}$$

These connector forces are listed in Table 8 and 9 under the column "Connector Forces".

TABLE 1
Designation of Beam Specimens

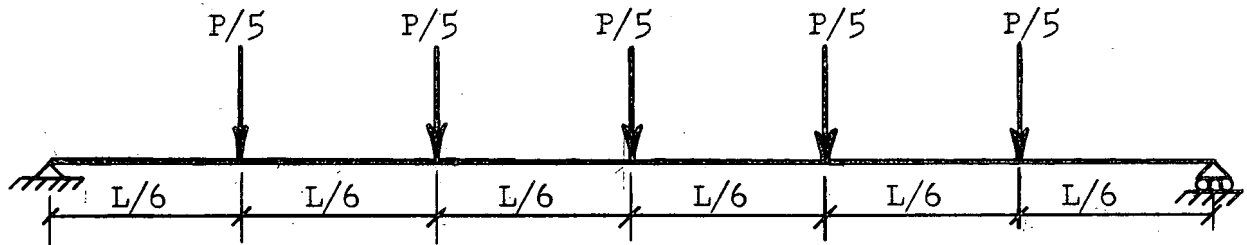


$P/2$ b b $P/2$
 E

Specimen	Connector Type	Connector Spacing c (in.)	Test No.	Load Spacing $2b$ (in.)	Test Designation
B7	1/2" dia. L studs	2at7.5	1	18	B7-S1
			2	36	B7-S2
B8	1/2" dia. headed studs	2at7.5	1	18	B8-S1
			2	36	B8-S2
			3	66	B8-S4
B9	3/4" dia. headed studs	2at15	1	18	B9-S1
			2	42	B9-S3
			3	72	B9-S5

Note: All specimens were loaded on the top of the slab

TABLE 2

Designation of Beam Specimens


Specimen	Connector Type	Type of Connector Spacing	Connector Spacing (in)	Test No.	Test Designation
B10	1/2"dia. L studs	Constant	2 at 9	1	B10-C
B11	1/2"dia. L studs	Constant	2 at 9	1	B11-C
B12	1/2"dia. L studs	Variable	-	1	B12-V

TABLE 3
Cylinder Strength of Concrete in Beam
Slabs and Pushout Specimens

<u>Cylinder No.</u>	<u>Age at Test</u> <u>(days)</u>	<u>Strength</u> <u>(psi)</u>
<u>Beams B7, B8, B9</u>		
1	35	3242
2	35	3500
3	35	3360
4	35	3230
5	42	3460
6	42	<u>3210</u>
	Ave.	3337 psi
<u>Pushout Specimens</u>		
<u>P7, P8, P9</u>		
1	22	3000
2	22	2990
3	22	3075
4	25	3120
5	25	3020
6	25	<u>3175</u>
	Ave.	3063 psi
<u>Beams B10, B11, B12, B13</u>		
1	34	3550
2	34	3582
3	34	3500
4	34	3430
5	40	3919
6	40	<u>3592</u>
	Ave.	3595 psi

TABLE 4
Coupon Tests of Material in I2WF27

Coupon No.	Material	Location of Coupon	Static Yield Stress (ksi)	Ultimate Strength (ksi)	Modulus of Elasticity E (ksi)
<u>Beams B7, B8, B9</u>					
1	(ASTM A7 Structural Steel)	Flange	37.3	64.8	31.6
2		Flange	37.4	63.8	31.1
3		Web	42.0	66.2	29.2
4		Web	41.7	66.0	30.7
		Ave	<u>37.35</u>		
		Ave	<u>41.85</u>		
<u>Beams B10, B11, B12, B13</u>					
1	(ASTM A7 Structural Steel)	Web	44.5	63.3	-
2		Web	43.7	63.6	27.4
3		Web	46.0	65.5	30.7
4		Flange	37.7	61.7	31.9
		Ave	<u>44.7</u>		
5	(ASTM A7 Structural Steel)	Flange	35.2	61.3	32.3
6		Flange	37.9	61.8	31.0
7		Flange	35.8	61.0	29.5
		Ave	<u>36.6</u>		

Average Values used in calculations

B7, B8, B9

$$f_y = 37.4 \text{ ksi (Flange)}$$

$$f_y = 41.9 \text{ ksi (Web)}$$

B10, B11, B12, B13

$$f_y = 36.6 \text{ ksi (Flange)}$$

$$f_y = 44.7 \text{ ksi (Web)}$$

TABLE 5Coupon Tests of Connector Material

<u>Specimen</u>	<u>Connector Material</u>	<u>Type of Coupon</u>	<u>Static Yield Stress (ksi)</u>	<u>Ultimate Strength (ksi)</u>	<u>Modulus of Elasticity E (ksi)</u>
1	1/2" dia. L studs	1/2" bar 20" long	58.4	66.9	30.6x10 ³
2	1/2" dia. headed studs	1/2" bar 20" long	59.4	67.7	30.6
3	3/4" dia.	Round Tensile Coupon; 0.505"Ø	62.5	76.2	29.1
4	3/4" dia. headed stud	Round Tensile Coupon; 0.505"Ø	61.5	75.4	29.6

TABLE 6Double Shear Tests of Connector Material

Specimen No.	Material*	Stud Type	Ultimate Shear Load (lbs)	Ultimate Shear Stress (psi)
1	C1010-C1017	1/2" L	17,740	45,300
2	"	1/2" L	17,540	44,700
3	"	1/2" headed	17,460	44,500
4	"	1/2" headed	17,600	44,900
5	C1015-C1017	3/4" headed	42,400**	49,800
6	"	3/4" headed	42,750**	50,000

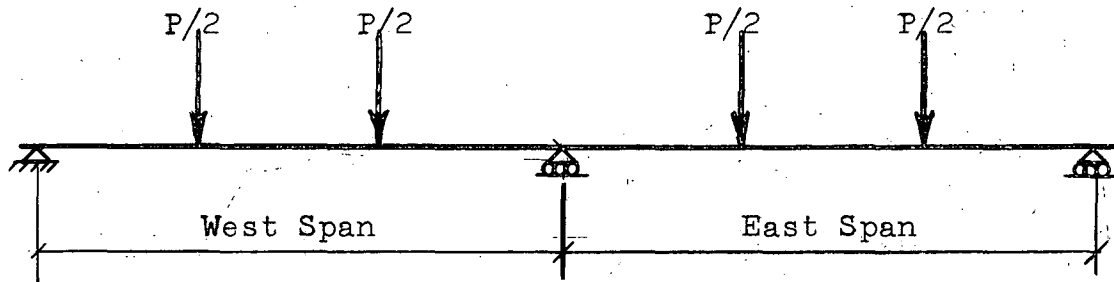
* Material designations are those of the American Iron and Steel Institute

** Area = 0.426 in²

The manufacturers specified properties of the stud material are as follows:

<u>1/2" L</u>	<u>3/4" headed</u>
Tensile strength 72,000 psi min	Tensile strength 65,000 psi min
Yield strength 61,000 psi min	
Elongation - 20% (2" gage length)	

TABLE 7

Loading Procedure Used for Continuous Beam

In order to determine the effect of the manner of loading on the behavior of a continuous composite beam, the following loading procedure was followed in testing Beam B13:

Loading	Span Loaded	Max. Load P/2 (kips)	Remarks
1	East Span	25	-
2	West Span	25	-
3	East Span	25	-
4	West Span	25	-
5	East Span	25	Load applied ten times
6	West Span	25	-
7	West Span	25	Load applied ten times
8	East Span	25	-
9	Both Spans	43.5	Loaded to failure

TABLE 8

Summary of Beam Test Results
(B7, B8, B9)

Specimen	Test	Load Spacing 2b (in.)	Failure Type	C _L Moment M (k-in.)		Connector Force Q (kips)	Max. End Slip at P _U (in.)	Residual End Slip (in.)
				M _P	M _U			
B7	B7-S1		(A)	2790	2430	12.3	0.059	0.046
B8	B8-S1	18	(A)	2790	2542	12.9	0.035	0.030
B9	B9-S1		(A)	2790	2510	23.2	0.040	0.029
B7	B7-S2	36	(C)	2790	2478	13.7	0.139	0.206*
B8	B8-S2	36	(A)	2790	2558	14.2	0.063	0.053
B9	B9-S3	42	(A)	2790	2498	27.7	0.039	0.027
B8	B8-S4	66	(C)	2790	2415	16.8	0.129	0.361*
B9	B9-S5	72	(B)	2790	2438	33.8	0.198	0.380

Failure Type: (A) Test stopped short of crushing of slab

(B) Failure to carry additional load

(C) Crushing of concrete slab

* After connector failure

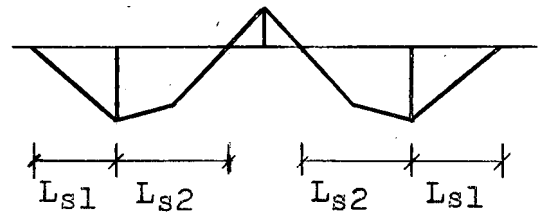
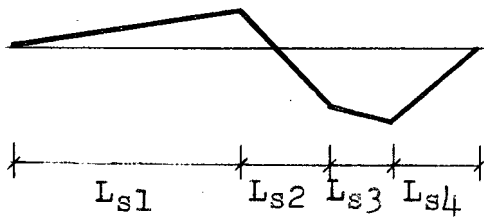
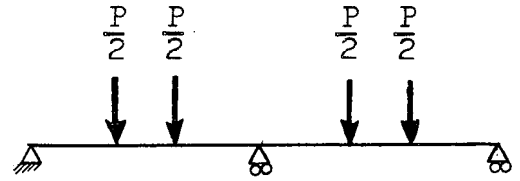
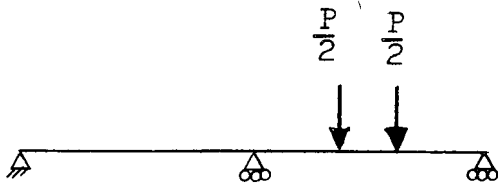
TABLE 9

Summary of Beam Test Results
(B10,B11,B12)

Specimen	Connector Spacing	Failure Type	Moment M (k-in.)		Connector Force Q (kips)	Max. End Slip at P_u (in.)	Residual End Slip (in.)
			M_p	M_u			
B10C	Uniform	Connector Failure	2840	2520	13.9	0.268	0.535
B11C	Uniform	Connector Failure	2840	2460	13.6	0.199	0.278
B12V	Variable: spaced in accordance with shear diagram	Connector Failure	2840	2510	13.9	0.170	0.372

Summary of Beam Test Results

(B13)



Load $P/2$ (kips)	Maximum Connector Force Q (kips)			
	L_{s1}	L_{s2}	L_{s3}	L_{s4}
25	1.4	7.4	11.6	6.2

Load $P/2$ (kips)	Connector Force Q (kips)	
	L_{s1}	L_{s2}
43.5	15.55	15.55

TABLE 11

Summary of Pushout Test Results

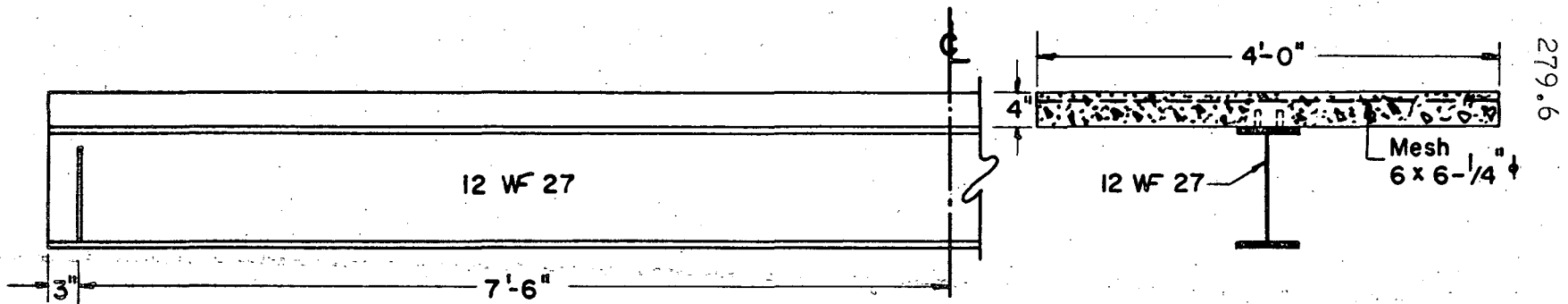
Specimen	Connector Type	Ultimate Connector Force QF (kips)	Shear Stress* (ksi)	Type of Failure	Remarks
P7	1/2" dia. L studs	6.75	34.4	Shearing of Studs	No Cracks in slab
P8	1/2" dia. headed studs	12.1	61.8	Shearing of Studs	No cracks in slab
P9	3/4" dia. headed studs	16.0	36.3	Shearing of Studs	Large cracks in slab

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

TABLE 12

Comparison of Beam Tests and Pushout Tests

Specimen	Connector Force QF	Manner of Failure	$\frac{Q_{beam}}{Q_{pushout}}$
B7	B7 - 13.7	B7 - shearing of studs	$Q_{B7}/Q_{P7} = 2.03$
	P7 - 6.75	P7 - shearing of studs	
B8	B8 - 16.8	B8 - shearing of studs	$Q_{B8}/Q_{P8} = 1.39$
	P8 - 12.1	P8 - shearing of studs	
B9	B9 - 33.8	B9 - failure to carry additional load	$Q_{B9}/Q_{P9} = 2.17$
	P9 - 16.0	P9 - shearing of studs	



Elevation

Cross Section

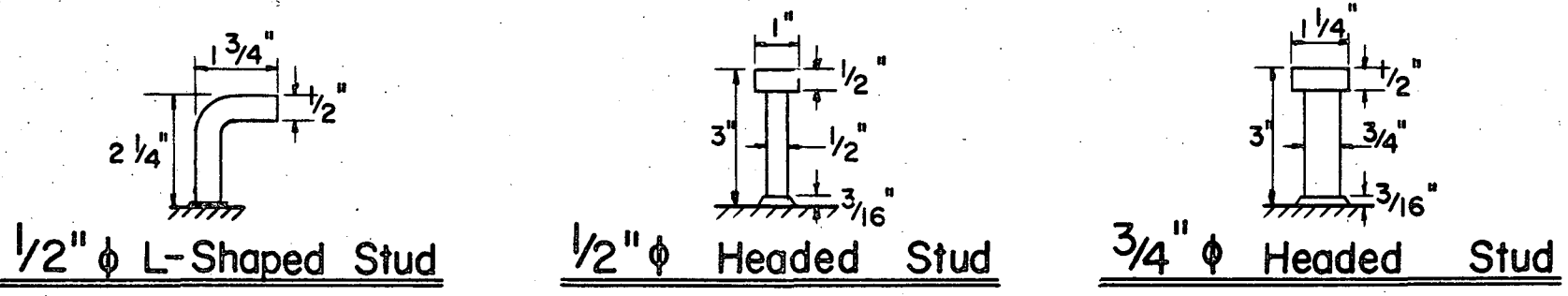
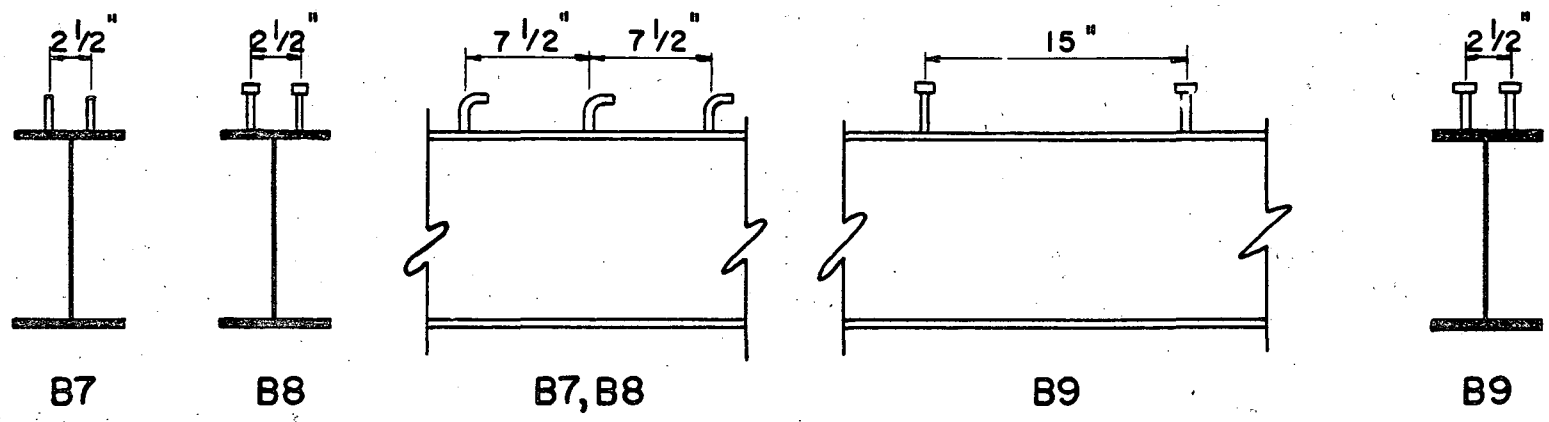


Fig. 1 - Dimensions of Beam Specimens B7, B8, B9

279.6

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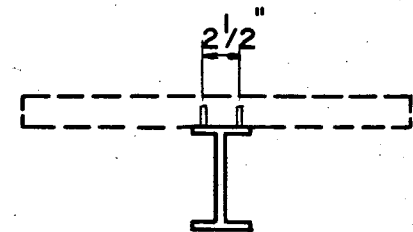
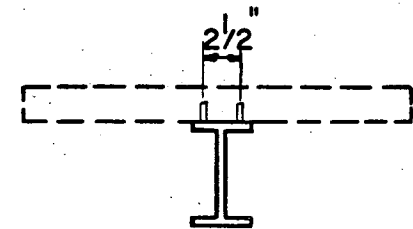
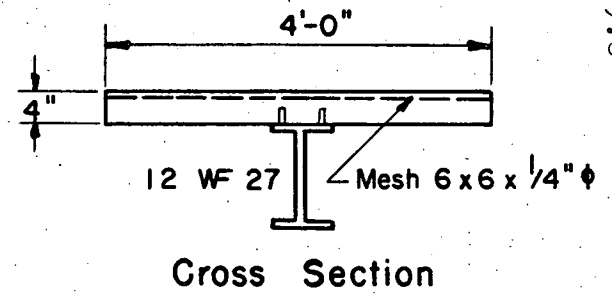
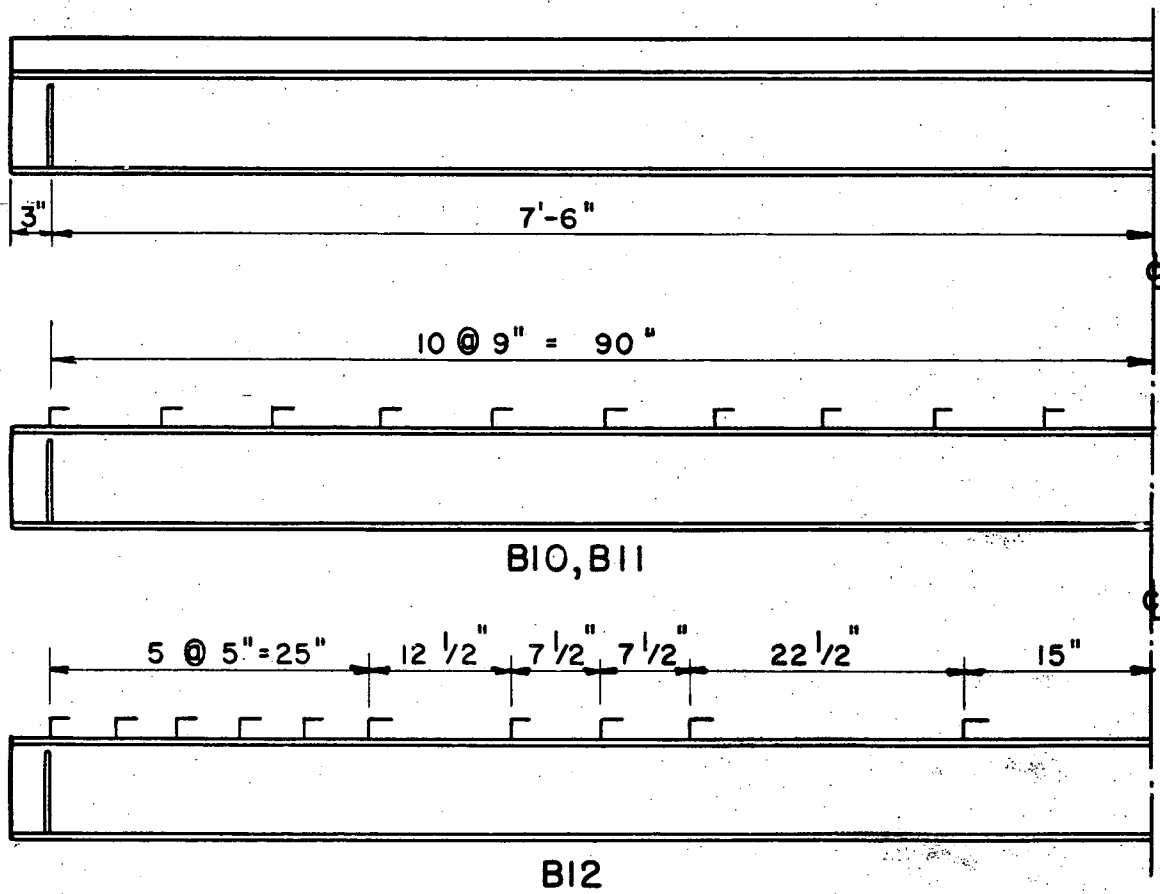
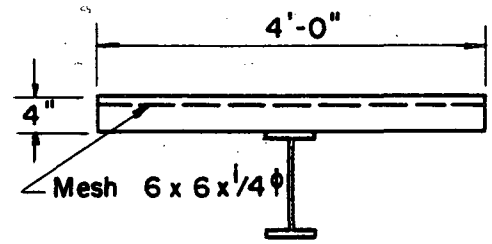
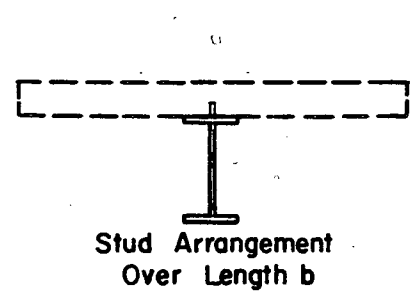
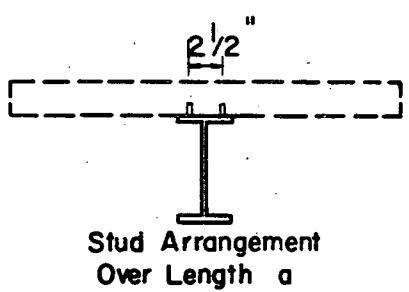
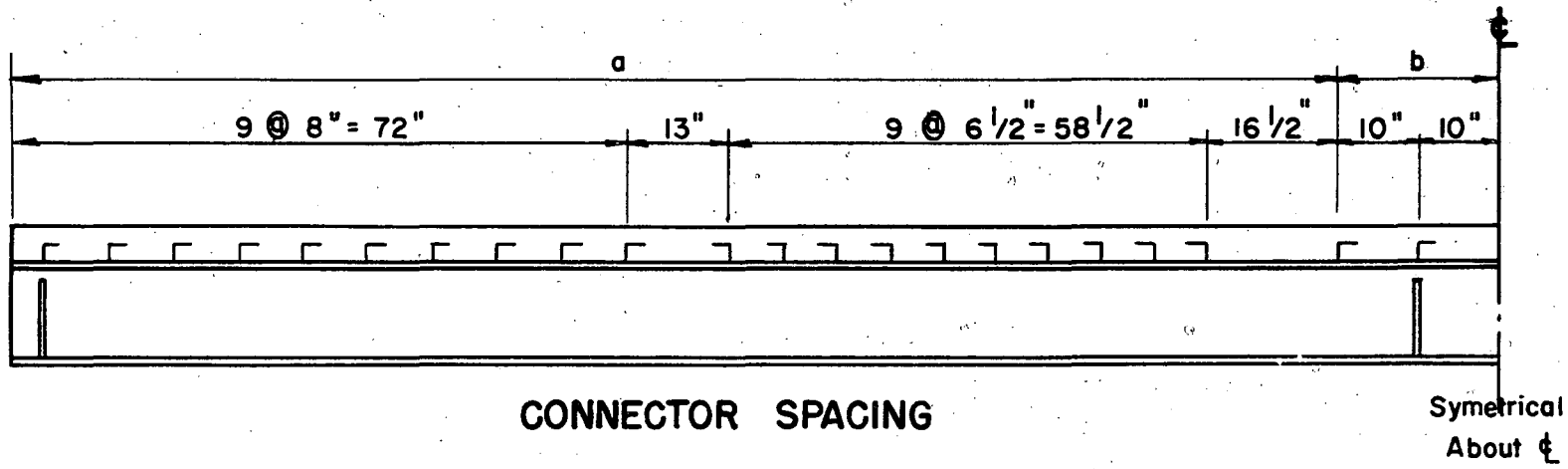
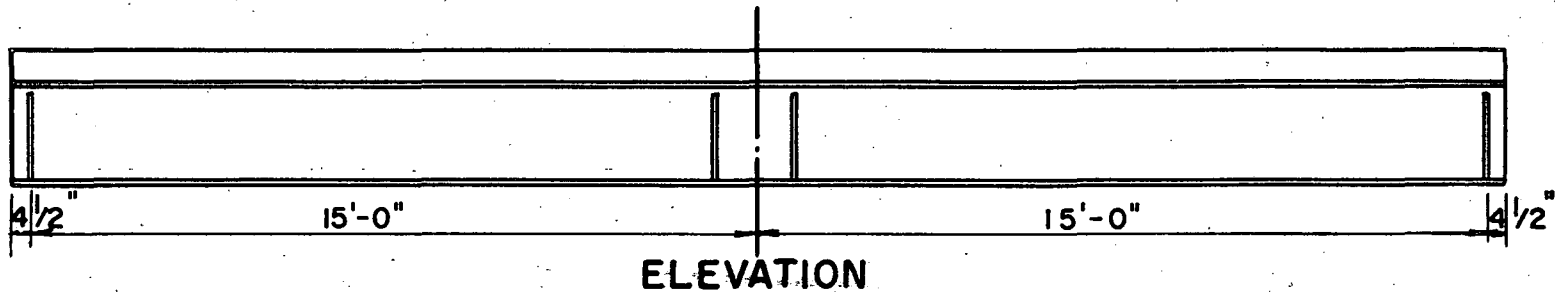


Fig. 2 - Dimensions of Beam Specimens B10, B11, and B12

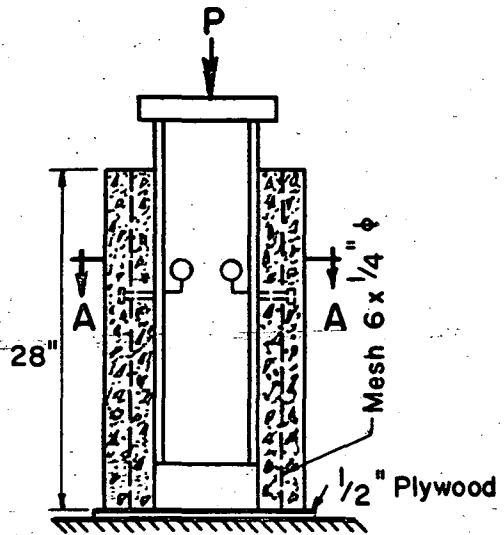
279.6



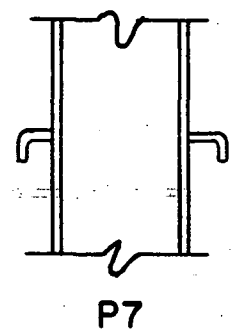
-61

Fig. 3 - Dimensions of Beam Specimen B13

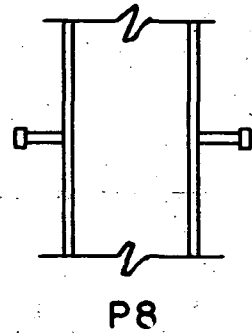
Types of Pushout Specimens



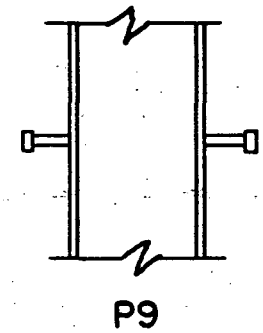
Typical Pushout Specimen



P7

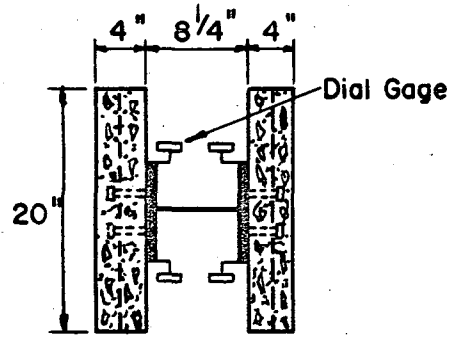


P8

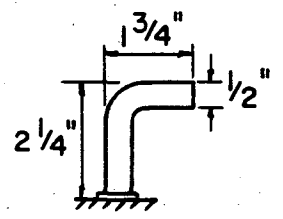


P9

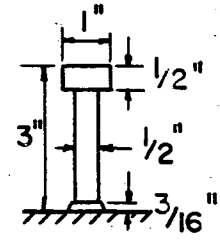
Details of Shear Devices



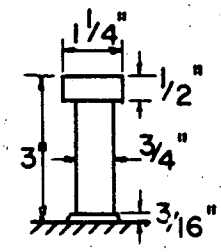
Section A-A



1/2" ϕ L-Shaped Stud P7



1/2" ϕ Headed Stud P8



3/4" ϕ Headed Stud P9

Fig. 4 - Dimensions of Pushout Specimens

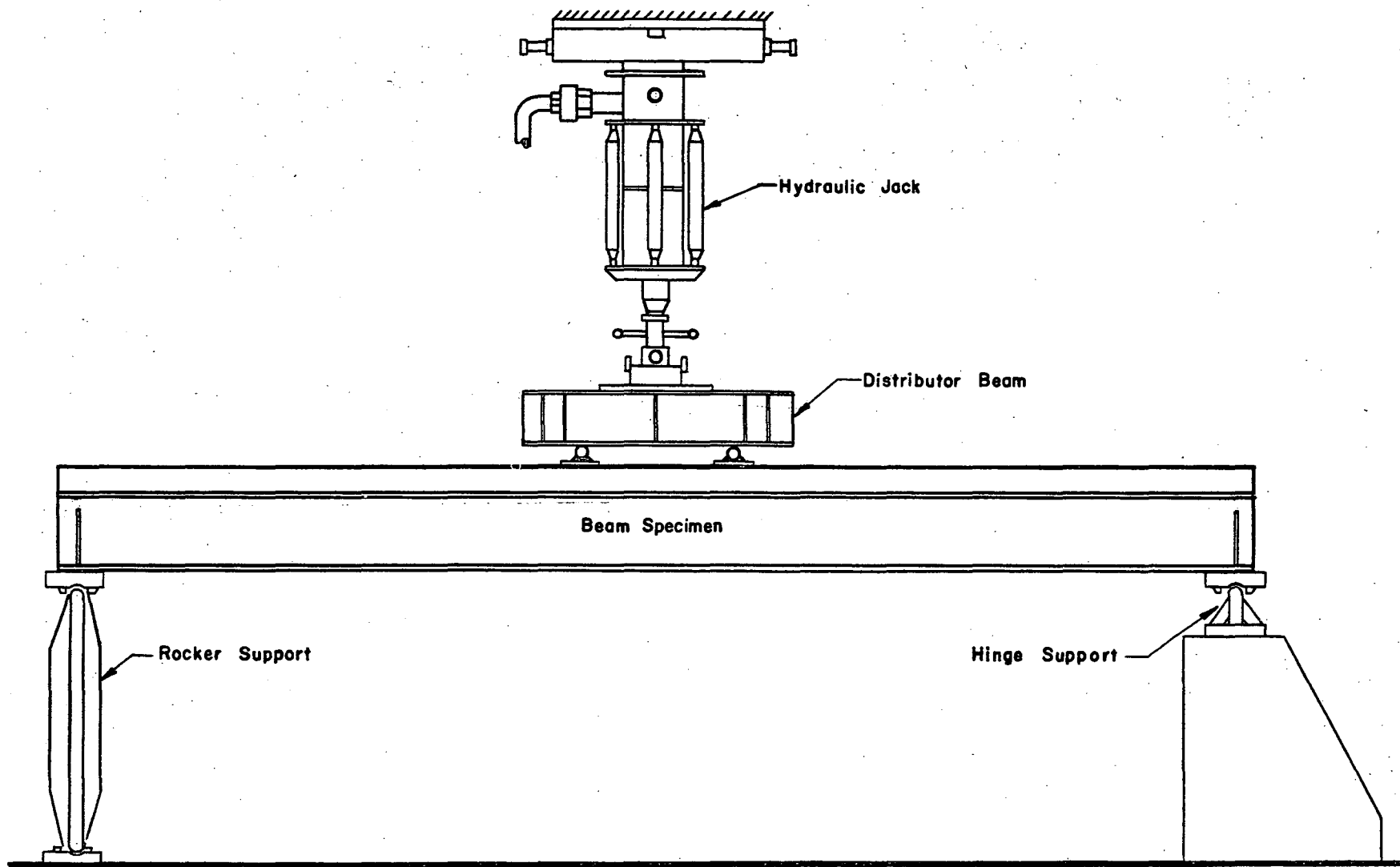


Fig. 5 - Test Setup for Beams B7, B8, B9

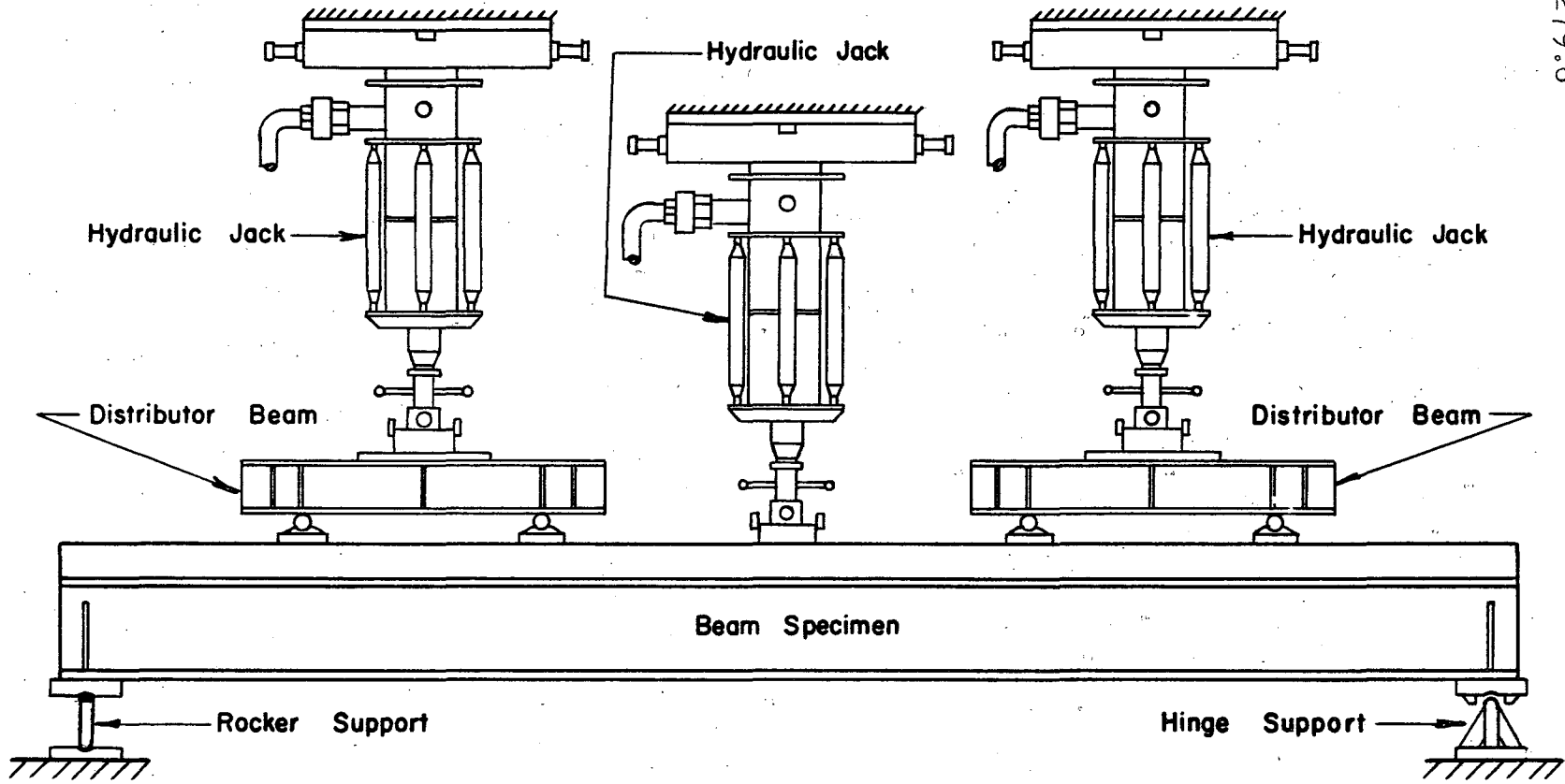
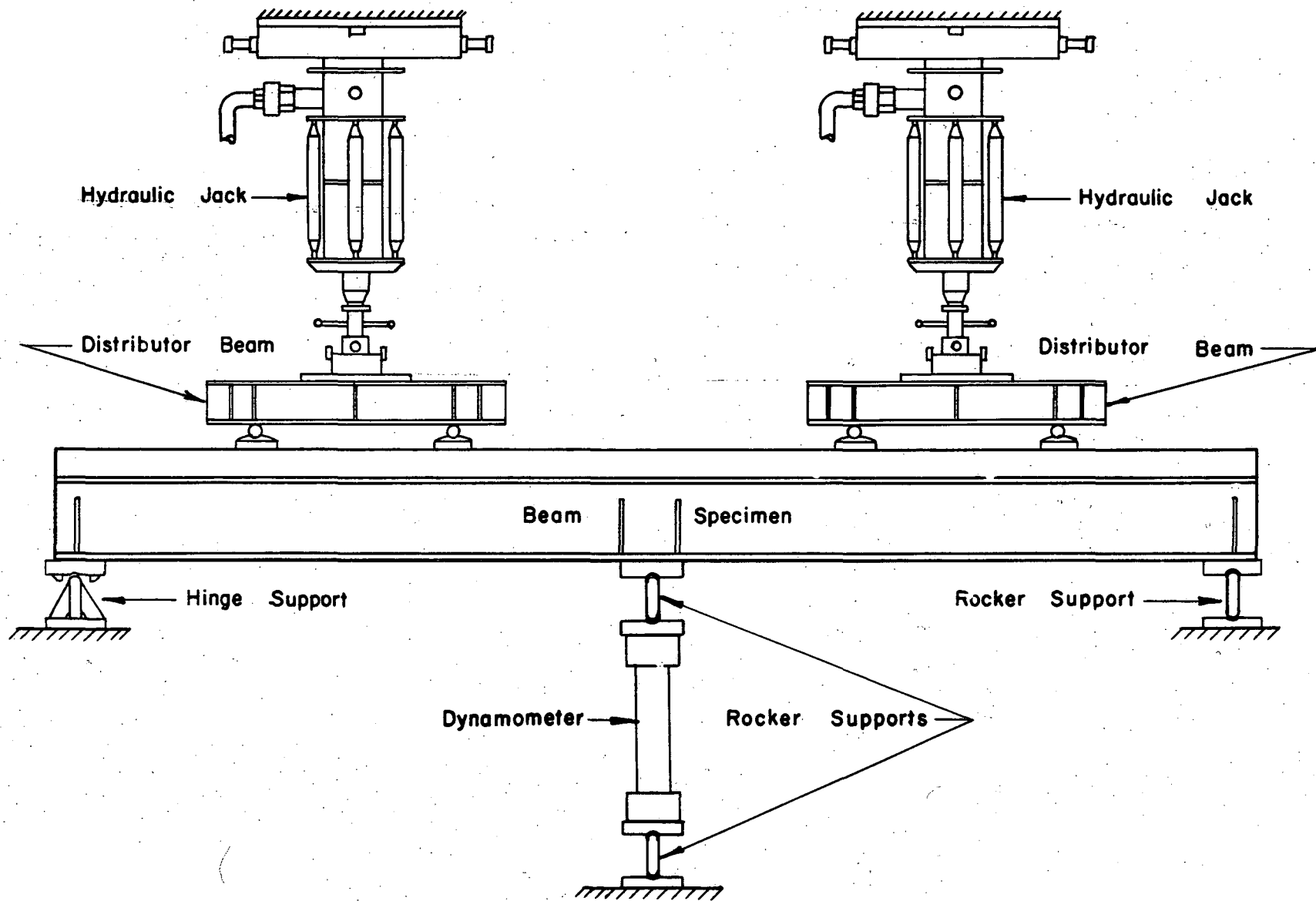


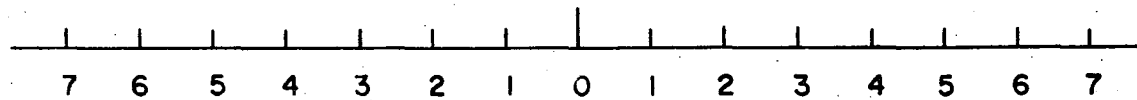
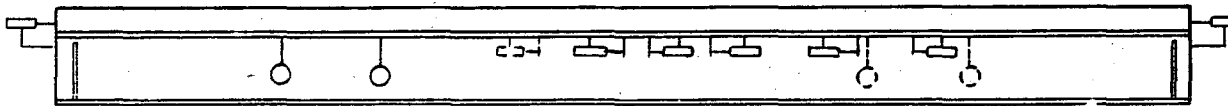
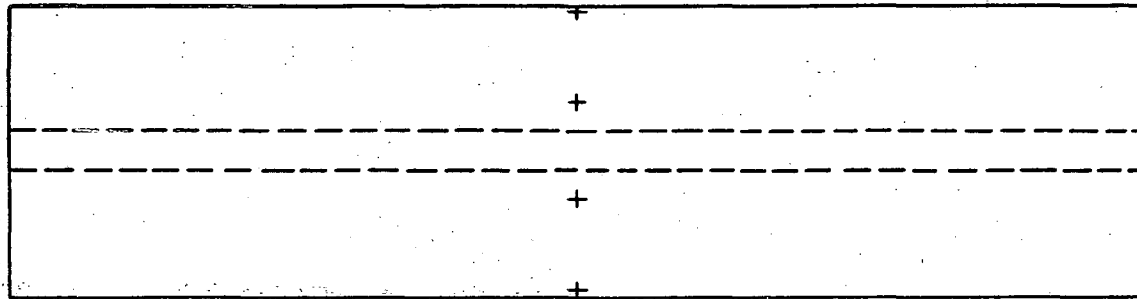
Fig. 6 - Test Setup for Beams B10, B11, B12



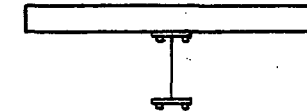
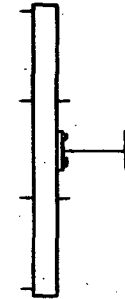
279.6

-65

Fig. 7 - Test Setup for Beam B13



DISTANCE FROM MIDSPAN IN FEET



279.6

GAGE NOTATION:




-  Uplift Dials
-  Slip Dials
- + Center Line 10" Whittemore Gage Line
-  1" SR-4 Gage at Center Line

Fig. 8 - Typical Arrangement of Recording Gages for Beam Specimens

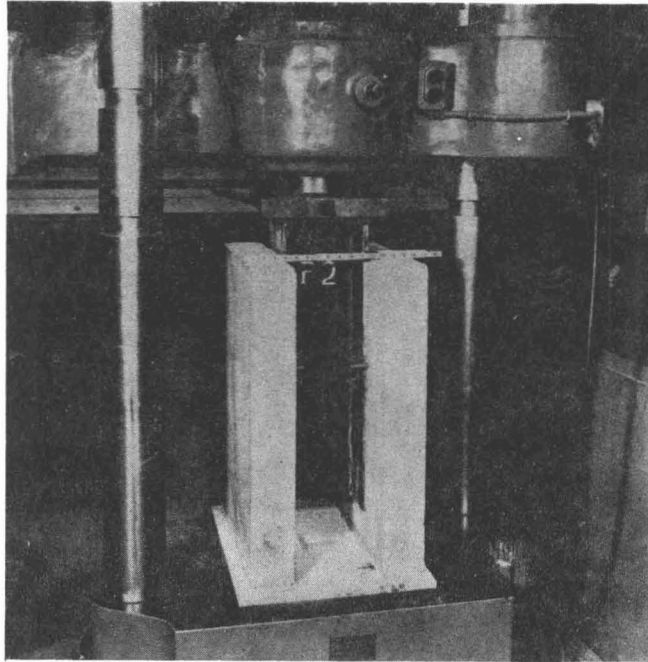
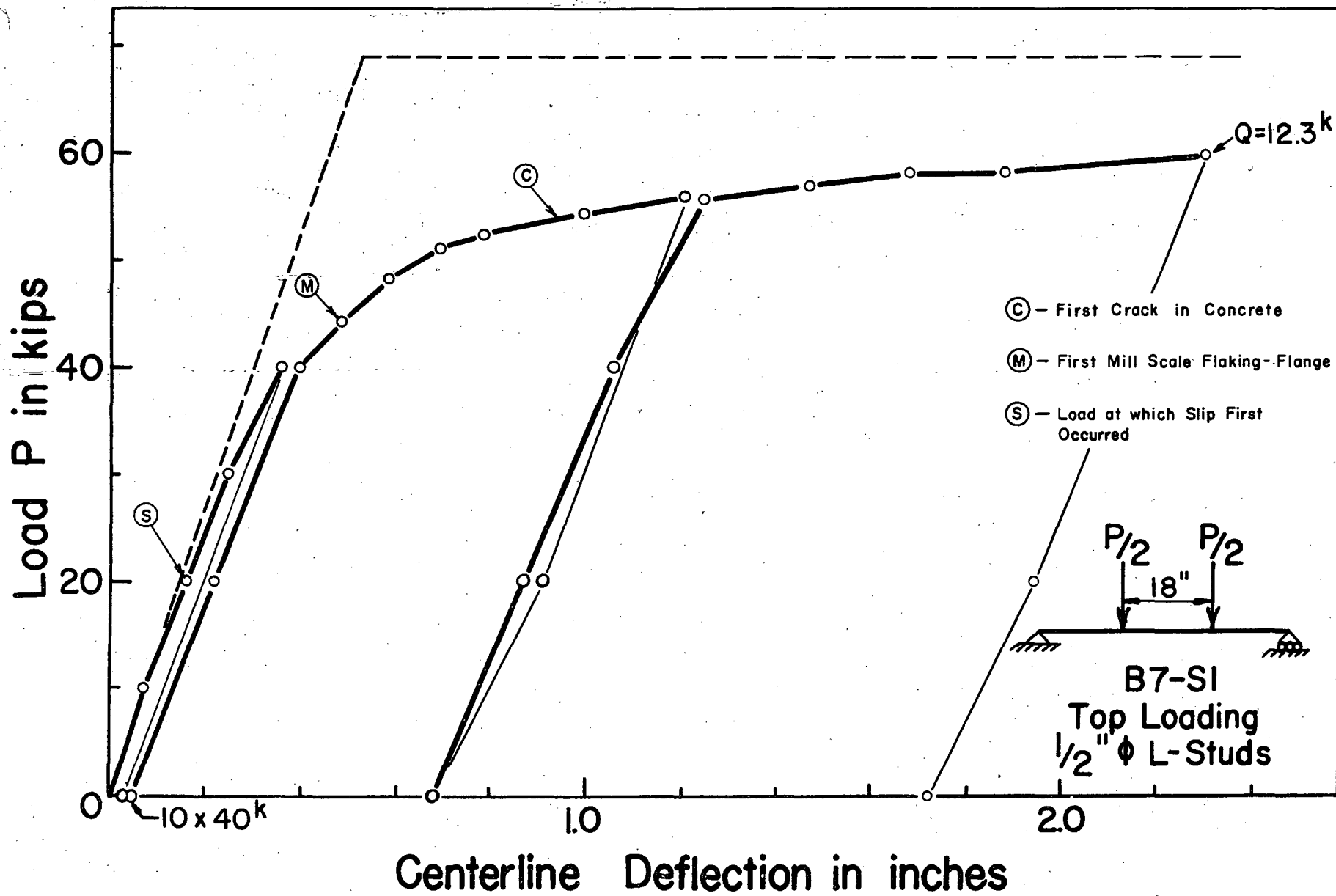


Fig. 9 - Test of Pushout Specimen



279.6

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Fig. 10 - Load Deflection Curve for First Test of Beam B7 with 1/2" L-Studs

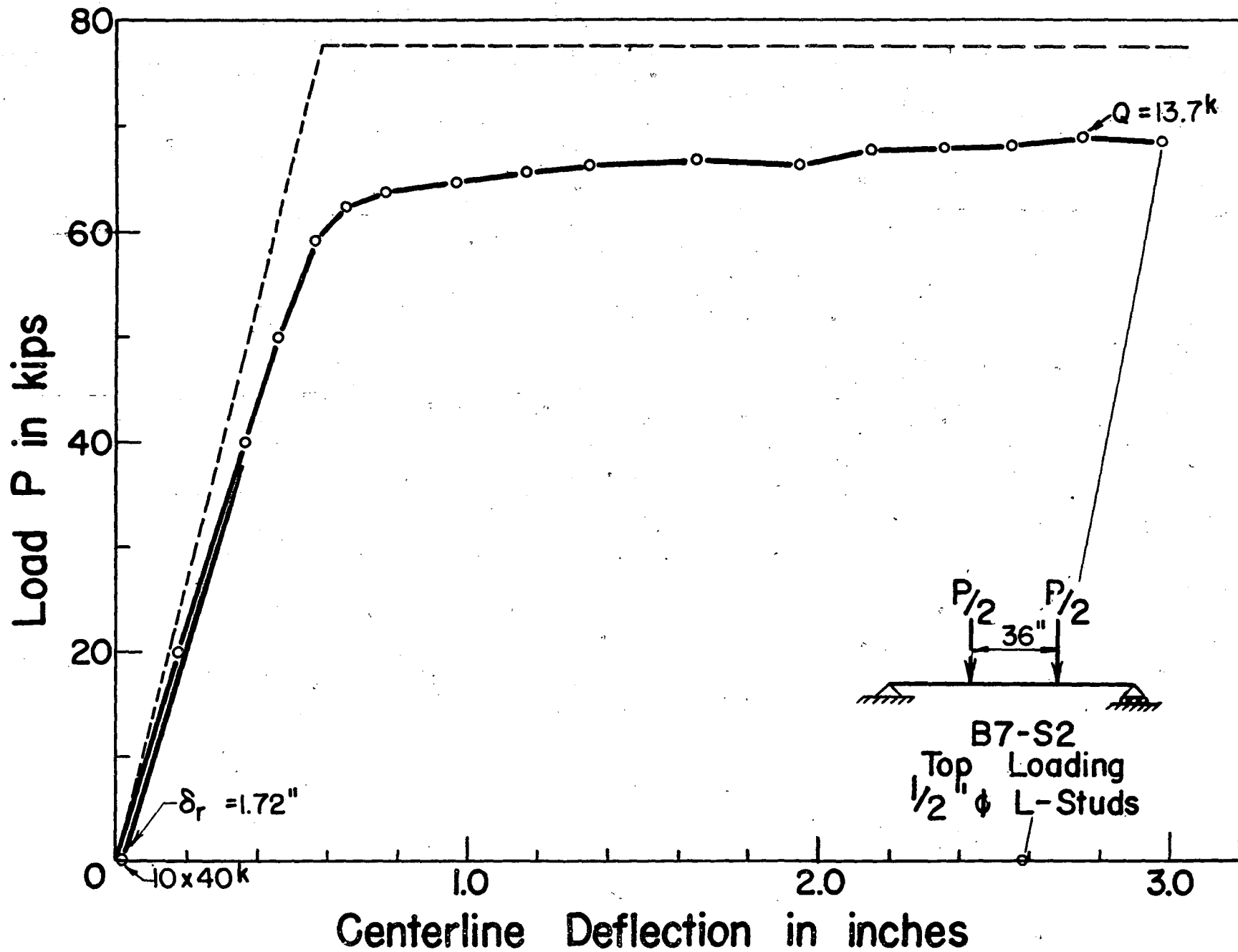


FIG. 11 - Load Deflection Curve for Final Test of Beam B7 with 1/2" Straight Studs

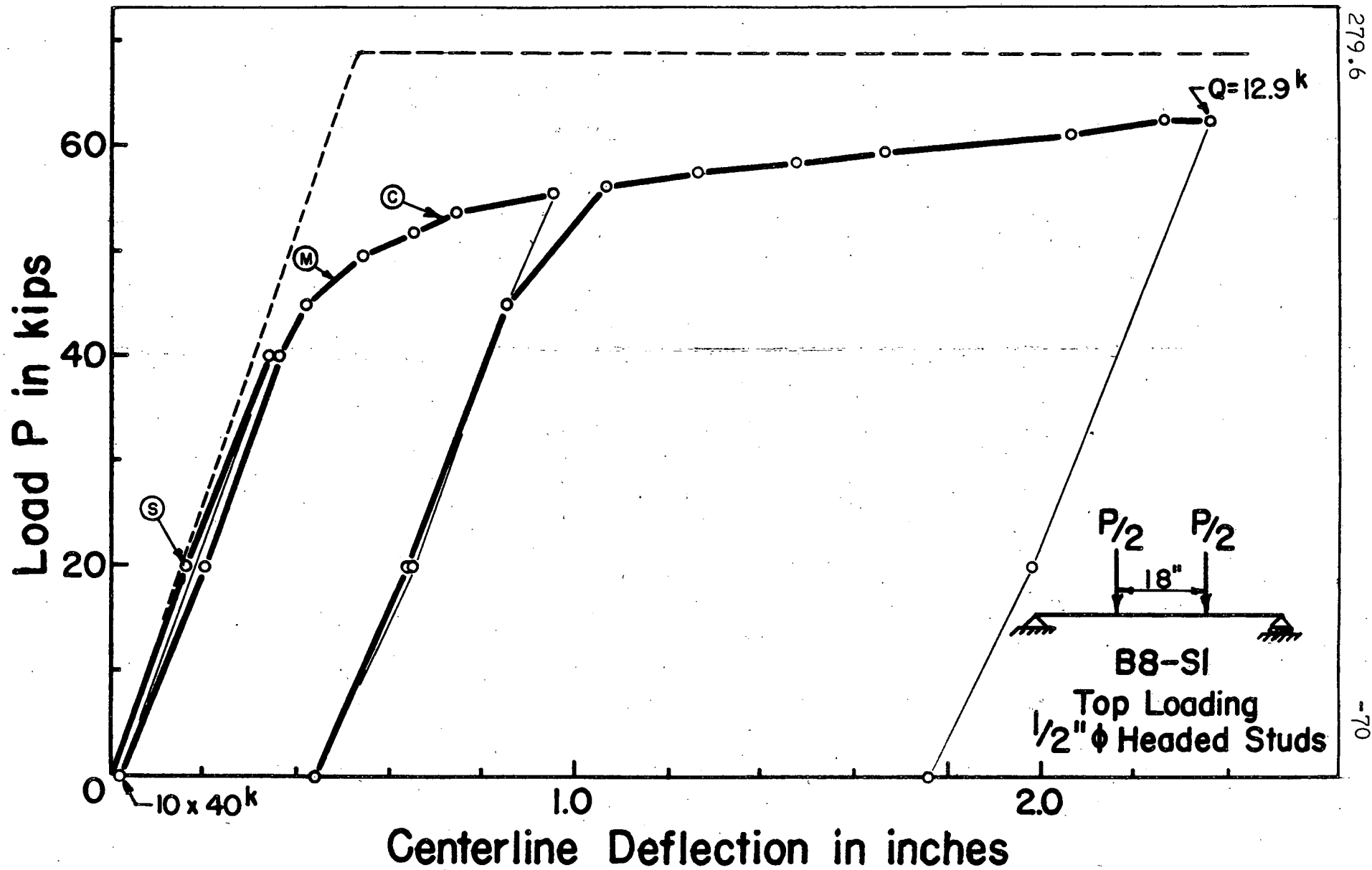
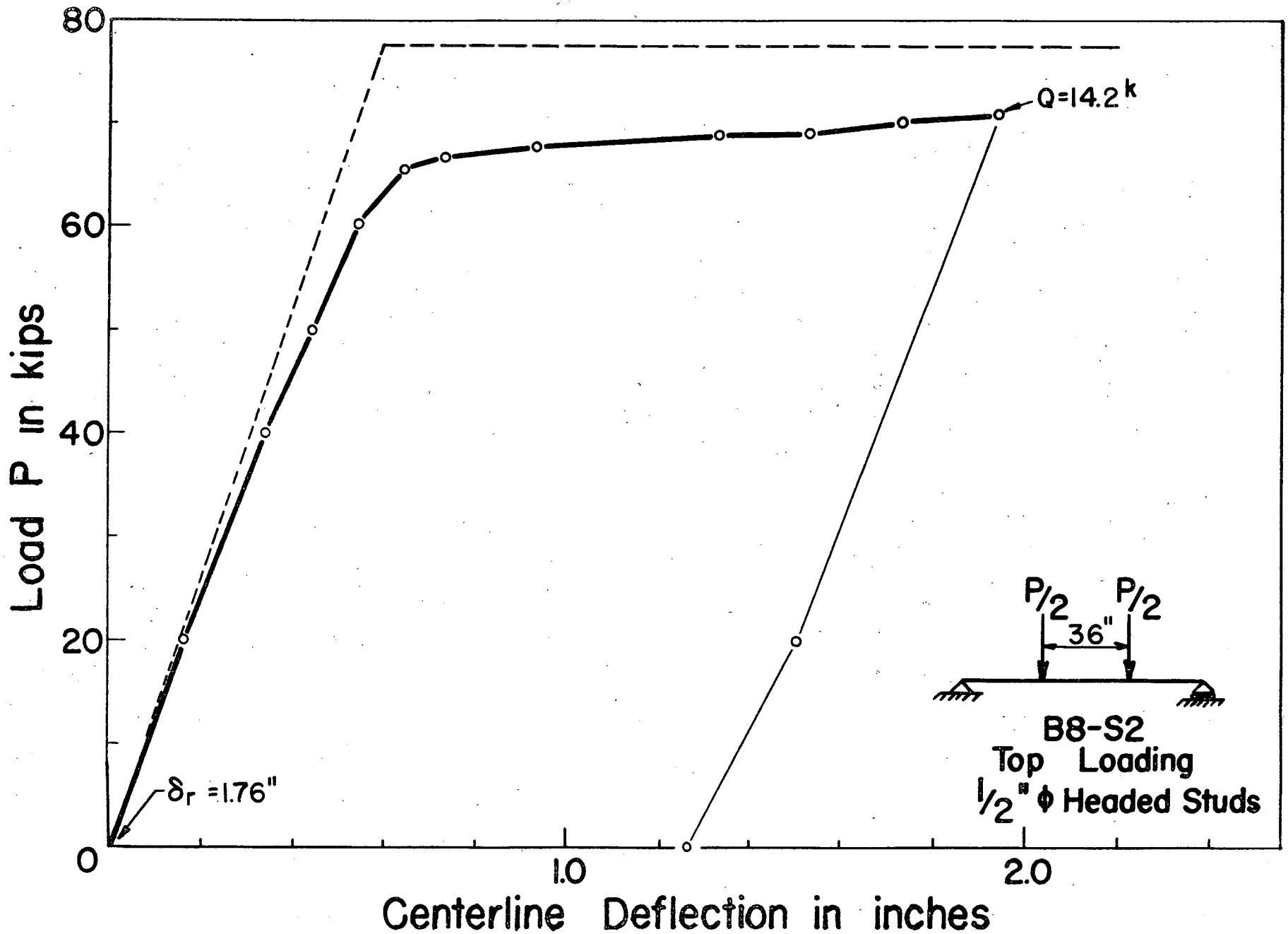


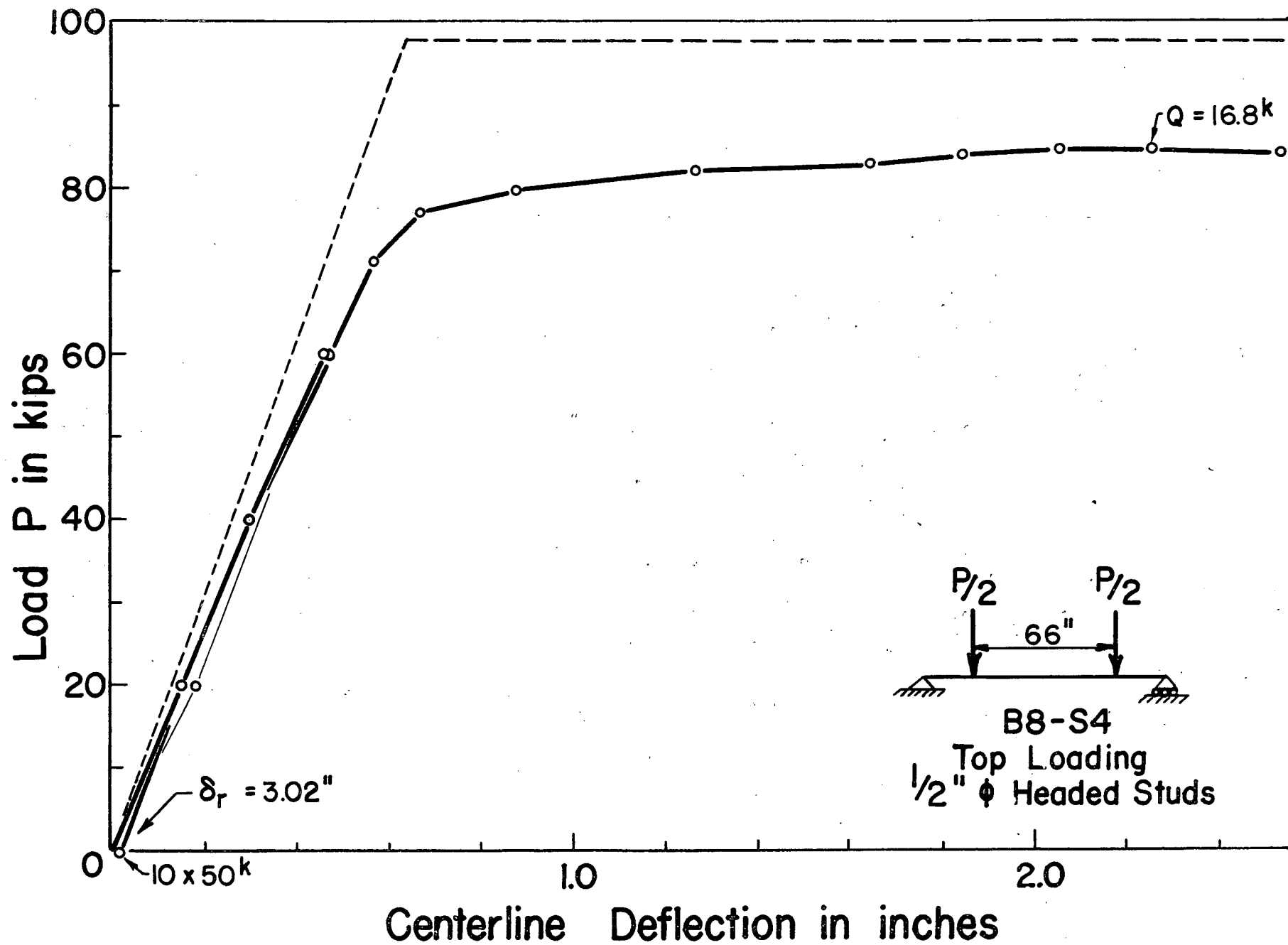
Fig. 12 - Load Deflection Curve for First Test of Beam B8 with 1/2" Straight Studs



279.6

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Fig. 13 - Load Deflection Curve for Second Test of Beam B8 with 1/2" Straight Studs



279.6

-72

Fig. 14 - Load Deflection Curve for Final Test of Beam B8 with 1/2" Straight Studs

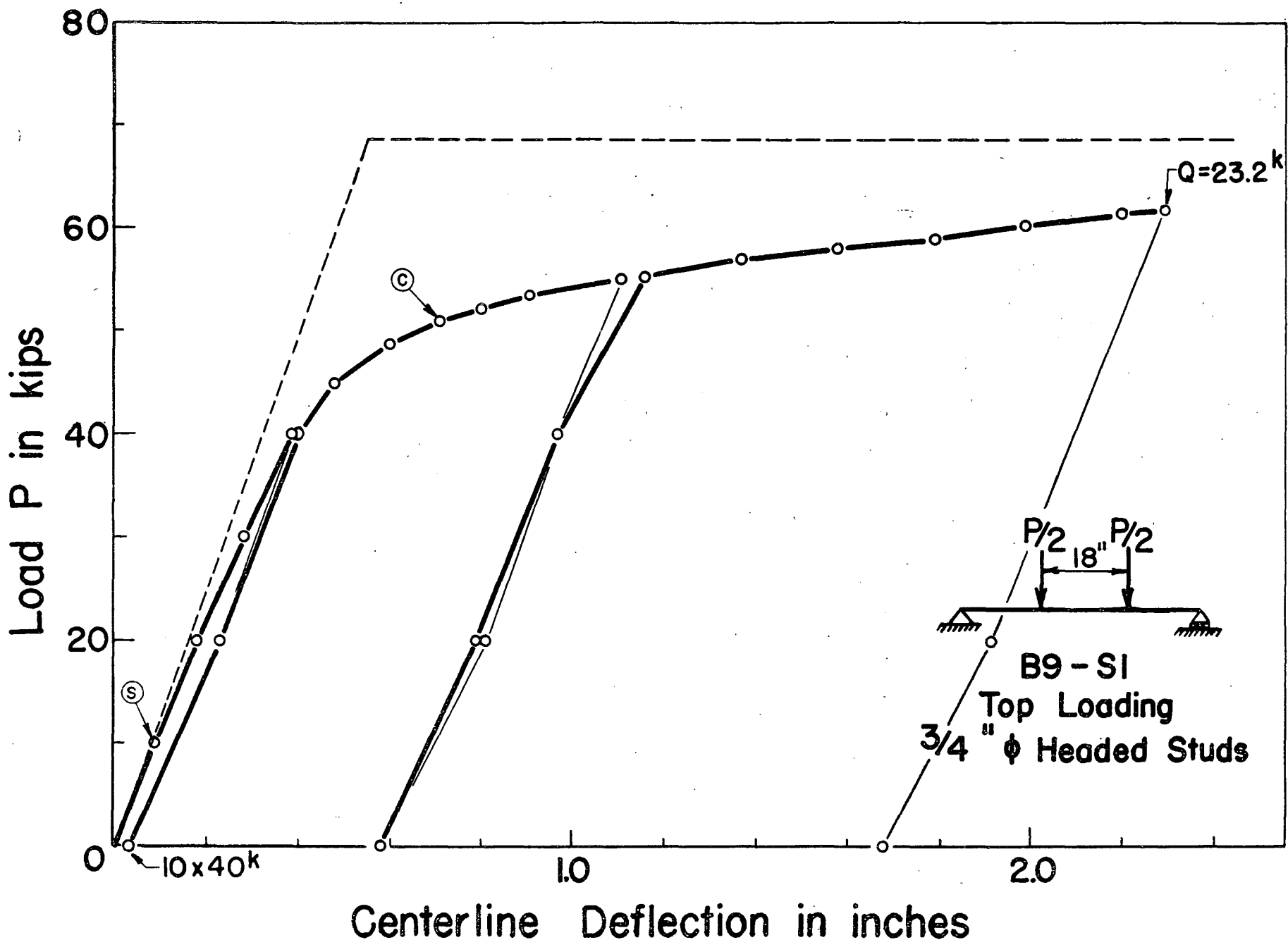
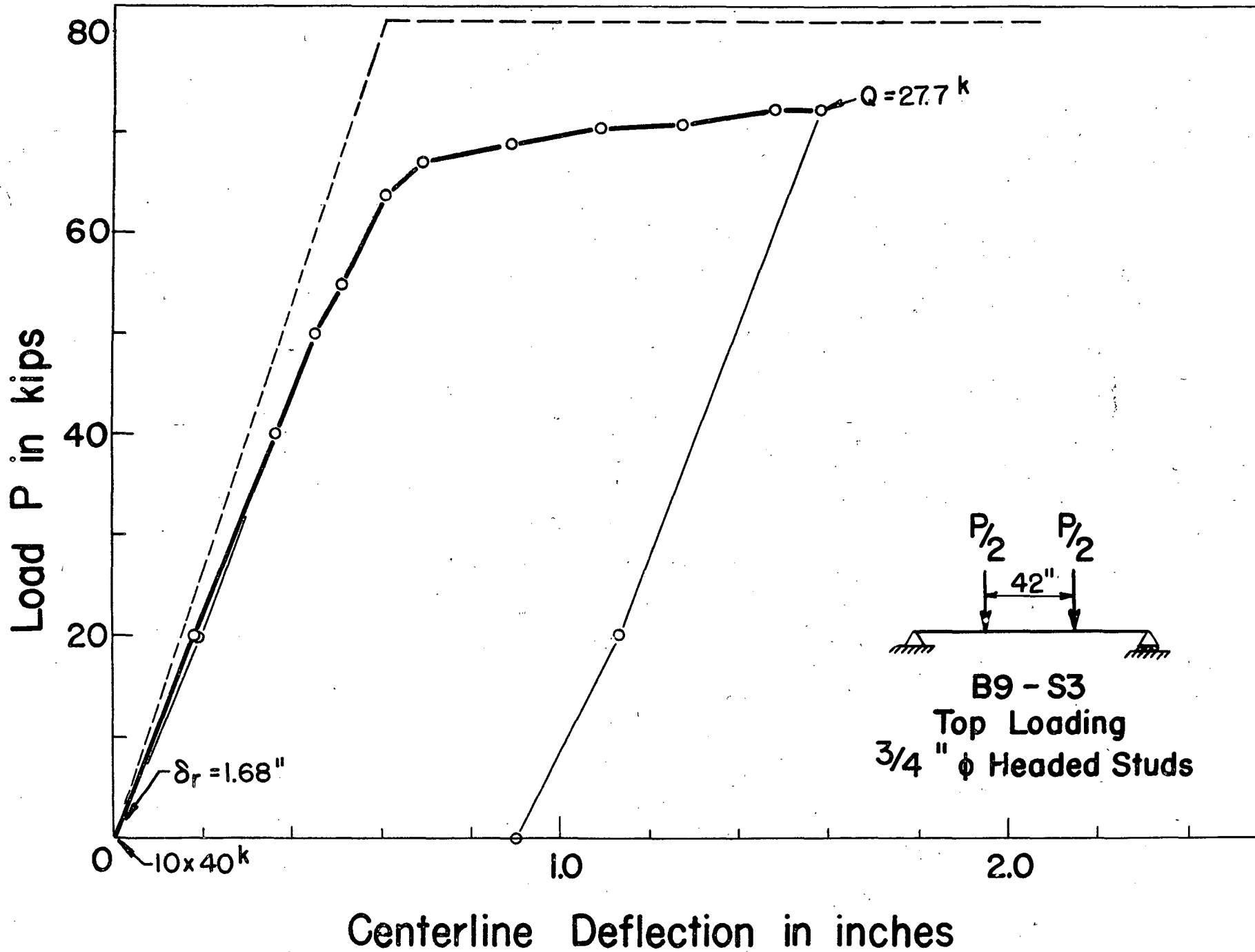


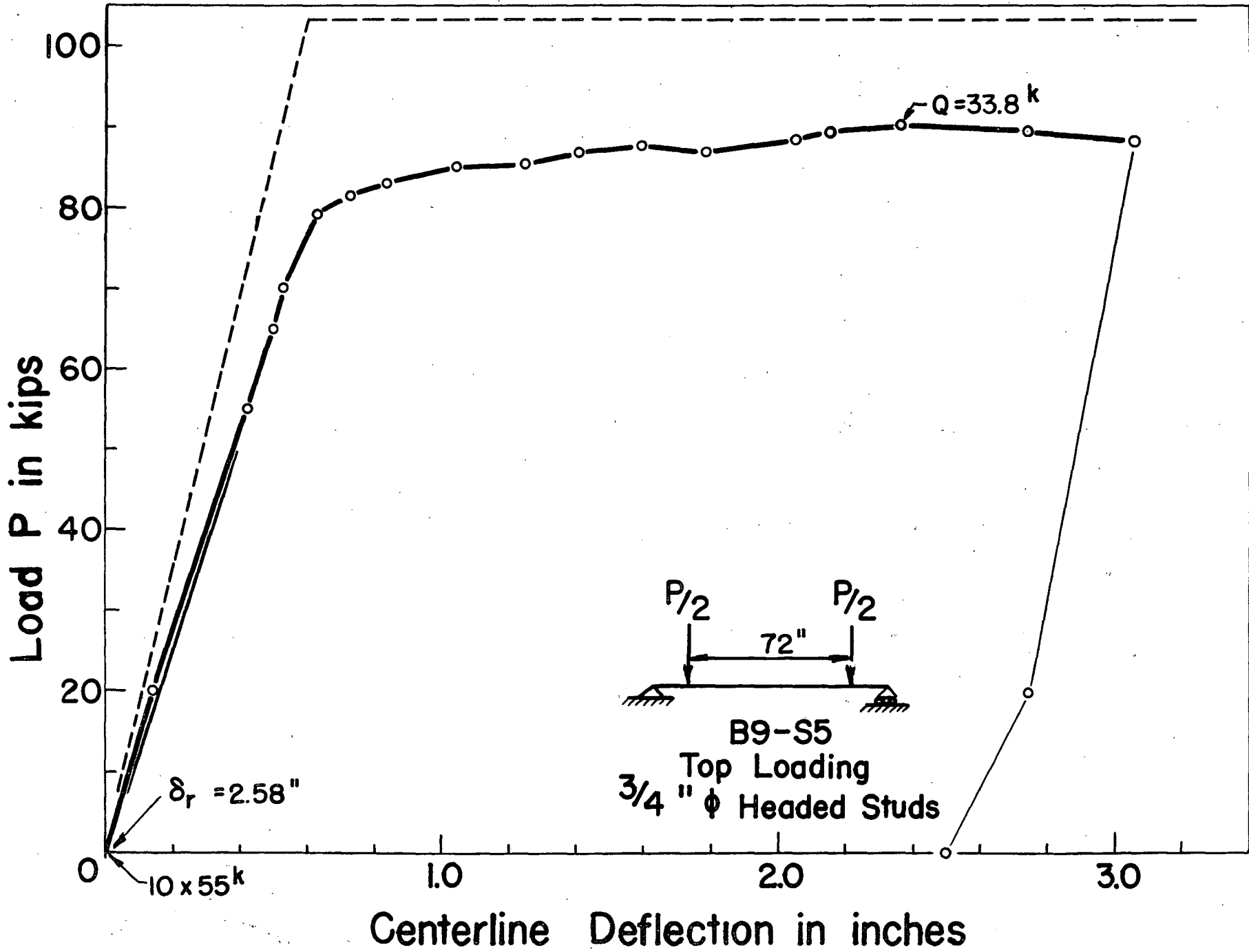
Fig. 15 - Load Deflection Curve for First Test of Beam B9 with 3/4" Straight Studs



279.6

74

Fig. 16 - Load Deflection Curve for Second Test of Beam B9 with $3/4''$ Straight Studs



279.6

-75

Fig. 17 - Load Deflection Curve for Final Test of Beam B9 with 3/4" Straight Studs

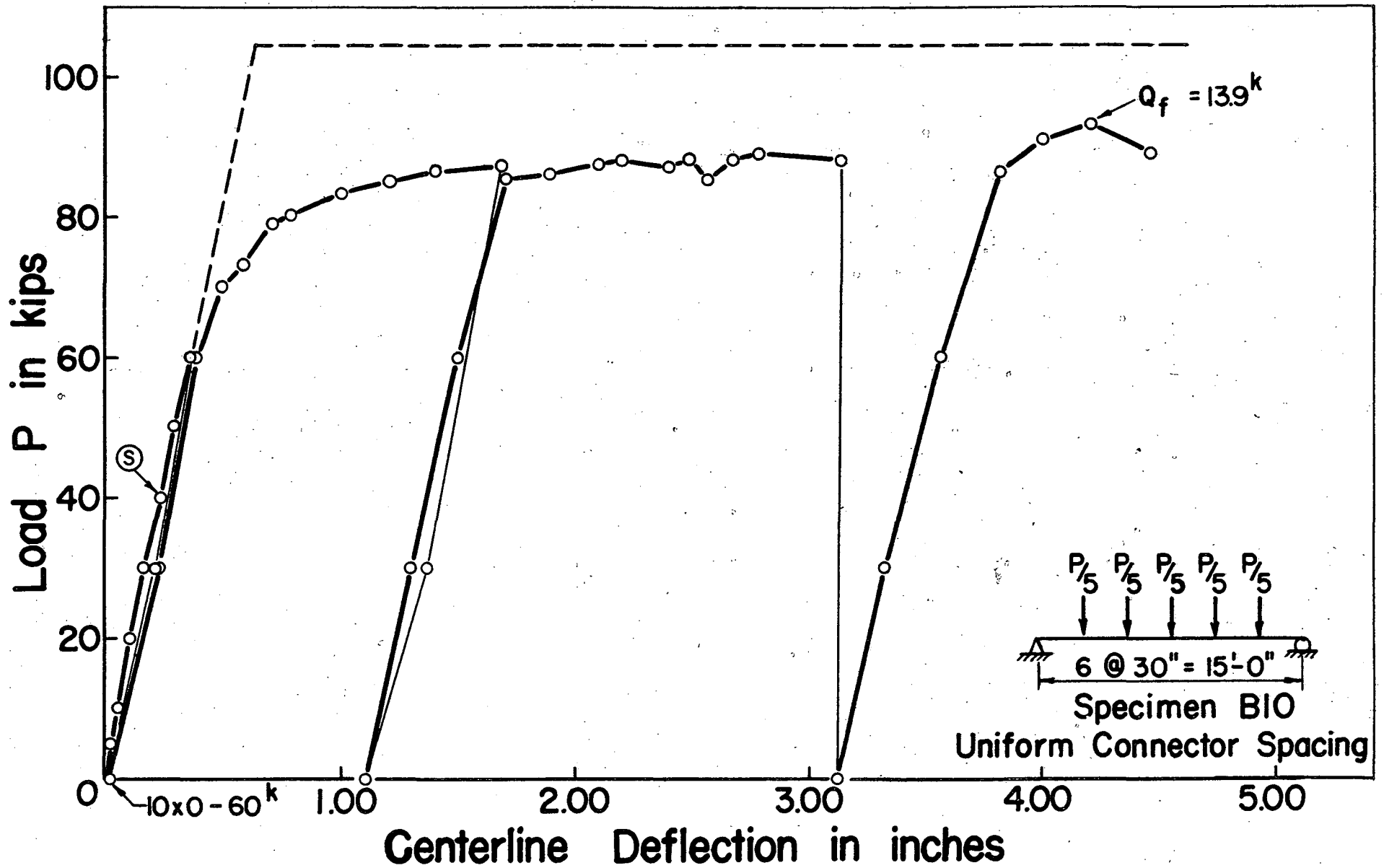


Fig. 18 - Load Deflection Curve for Test of Beam B10
with Uniform Connector Spacing

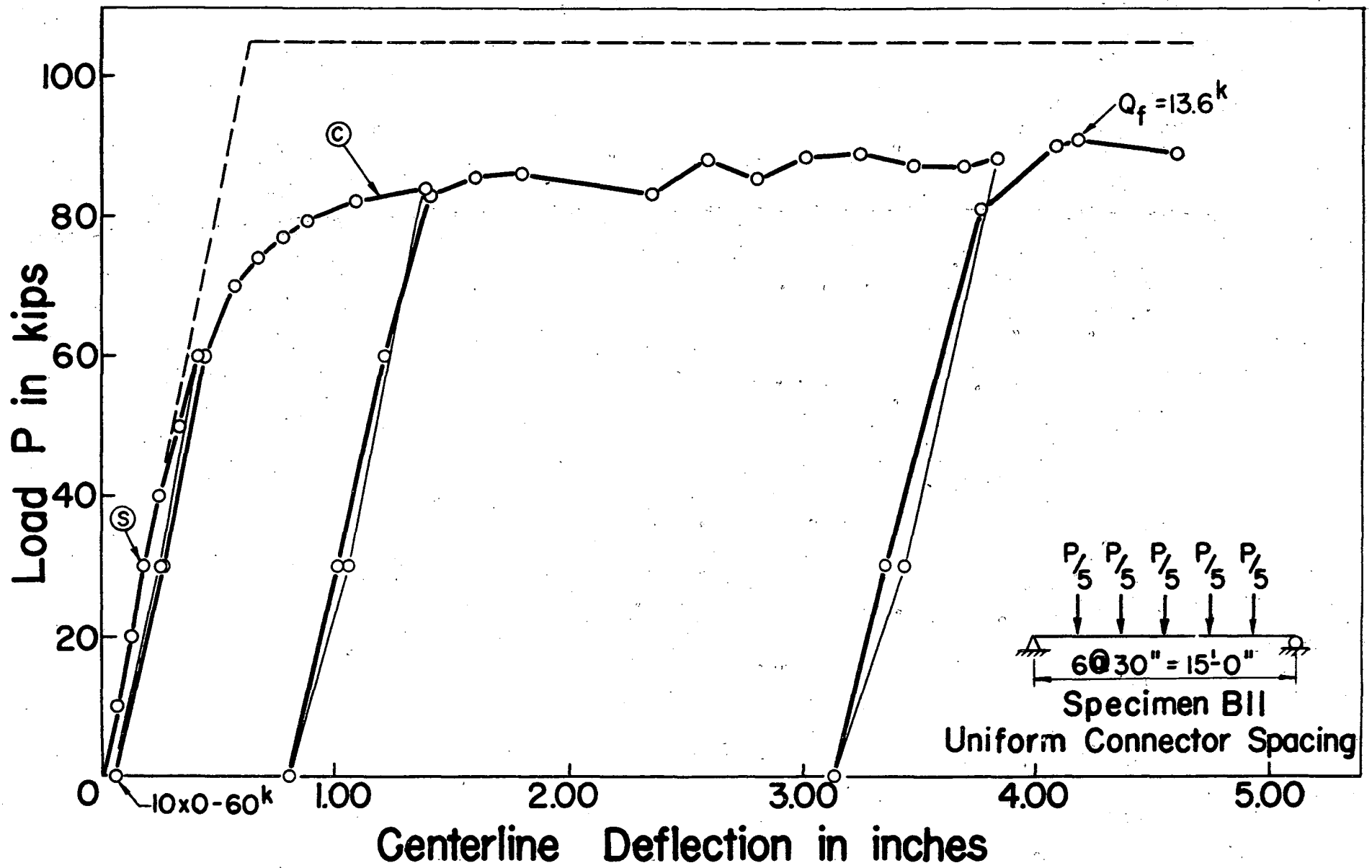


Fig. 19 - Load Deflection Curve for Test of Beam B11
with Uniform Connector Spacing

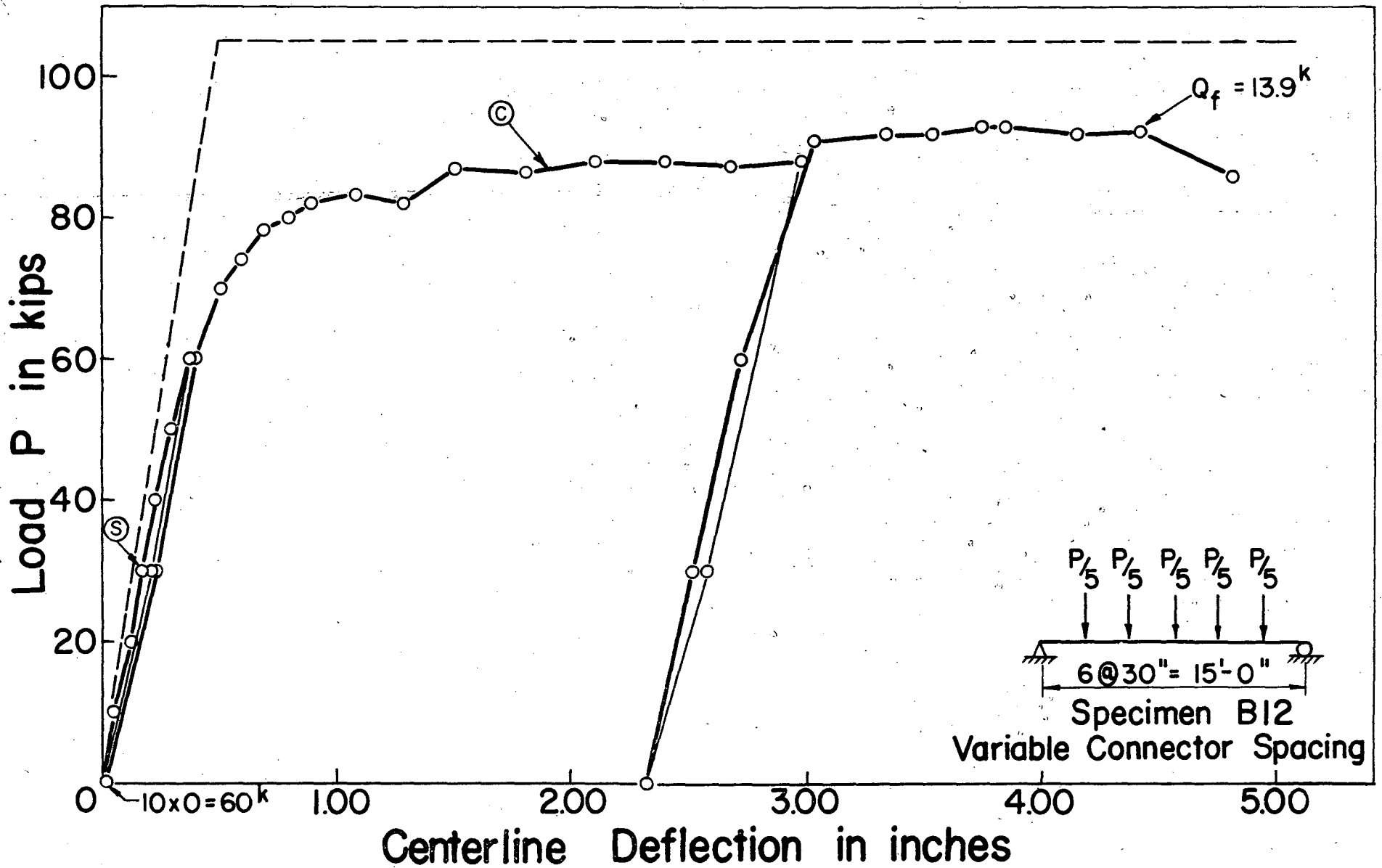
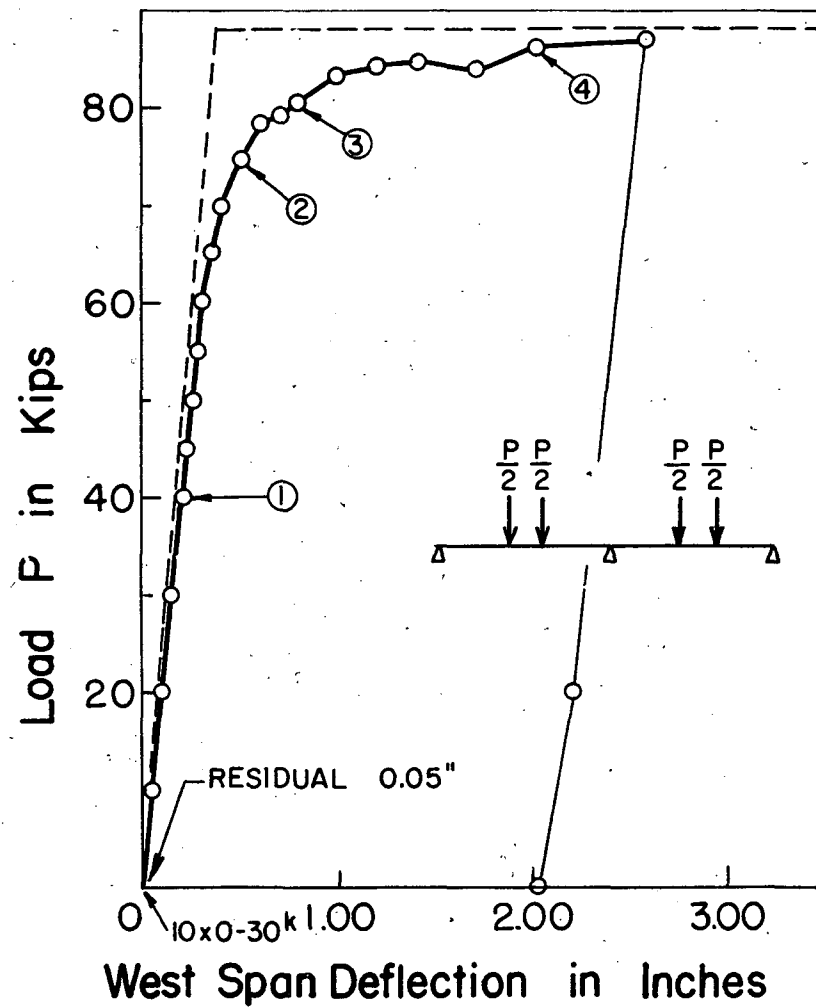
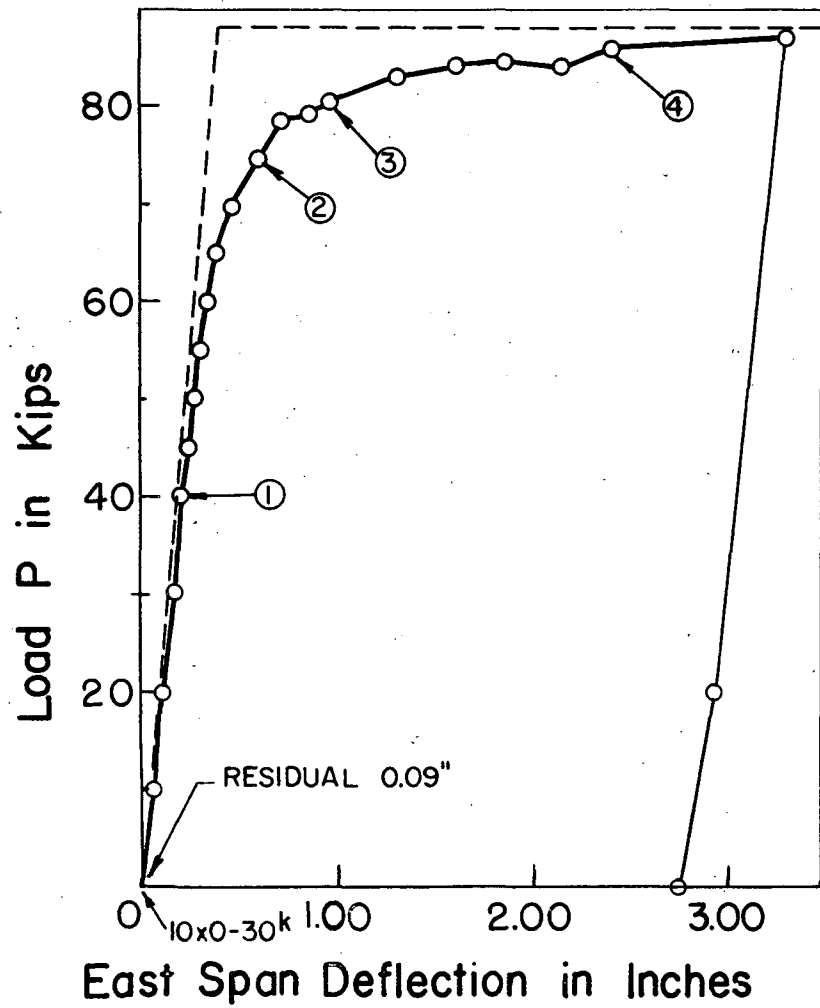


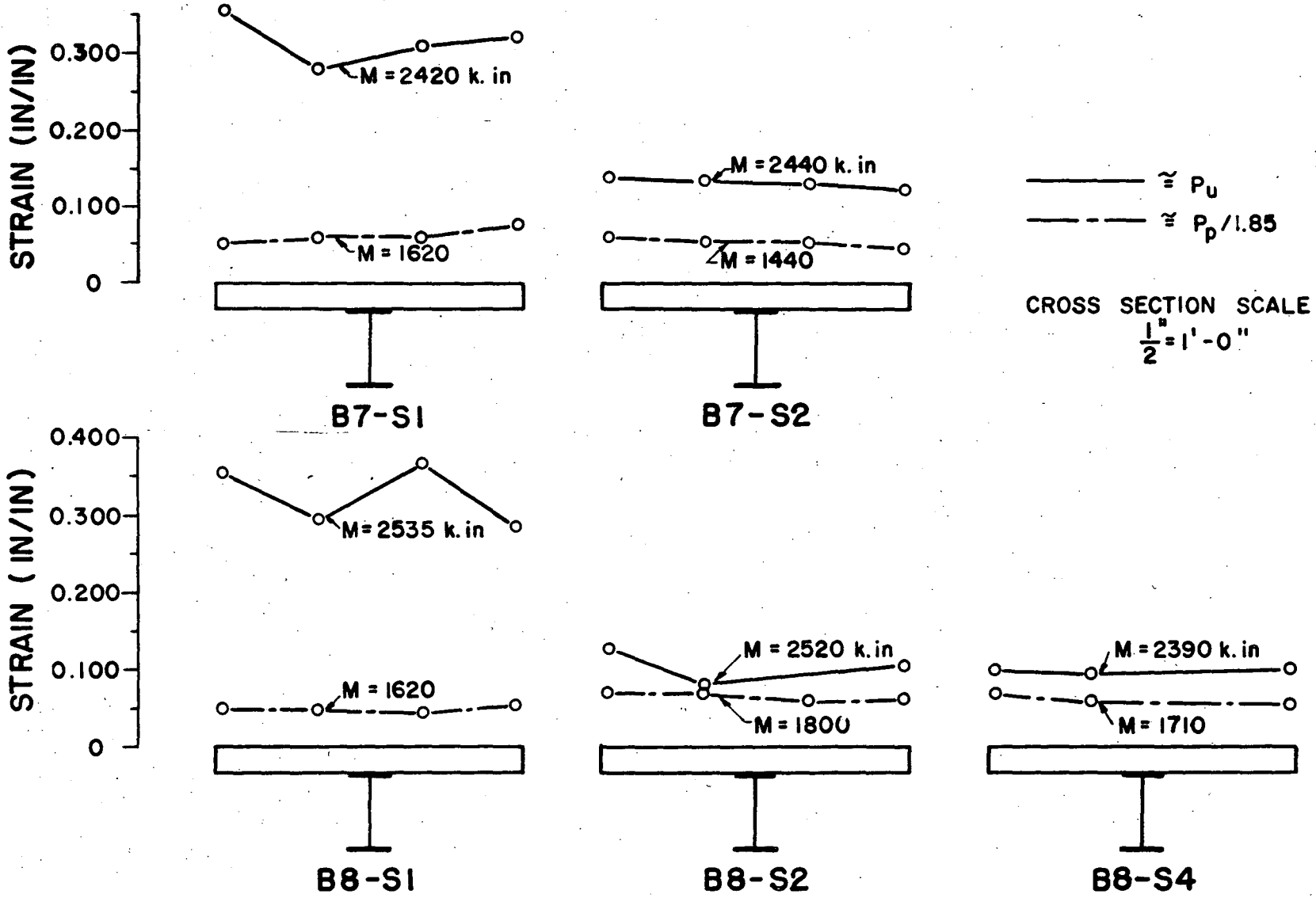
Fig. 20 - Load Deflection Curve for Test of Beam B12
with Variable Connector Spacing



279.6

-79

Fig. 21 - Load Deflection Curve for Test of Continuous Beam B13

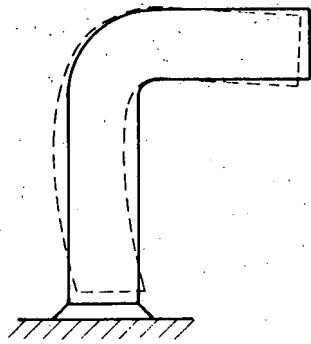


279.6

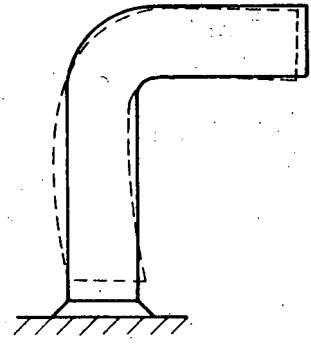
80

Strain Distribution Across Slab at Midspan

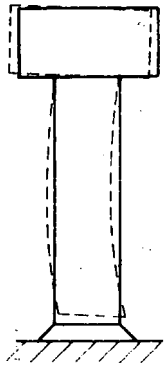
Fig. 22 - Typical Strain Distribution Across Slab



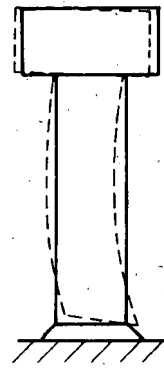
B7



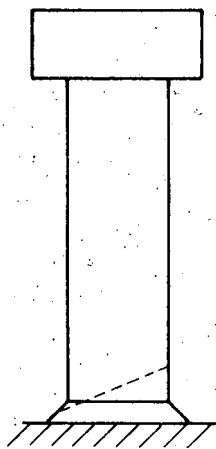
P7



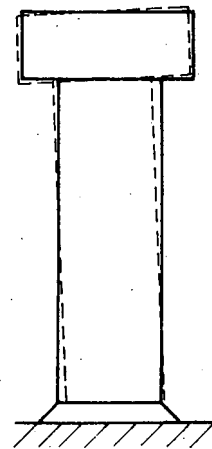
B8



P8



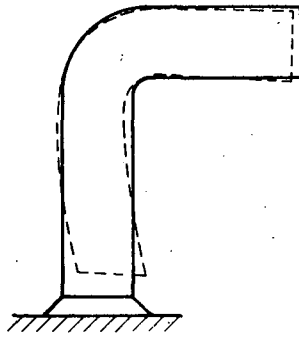
B9



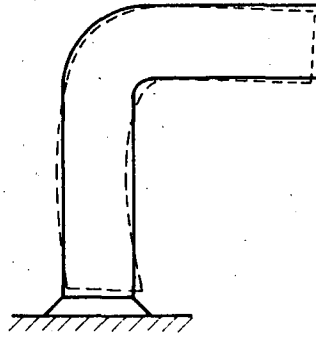
P9

Typical Connector Failures

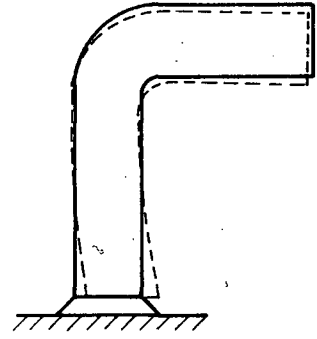
Fig. 23 - Comparison of Beam and Pushout Connector Failures



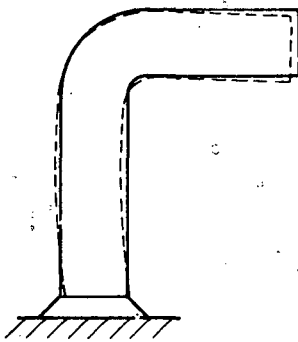
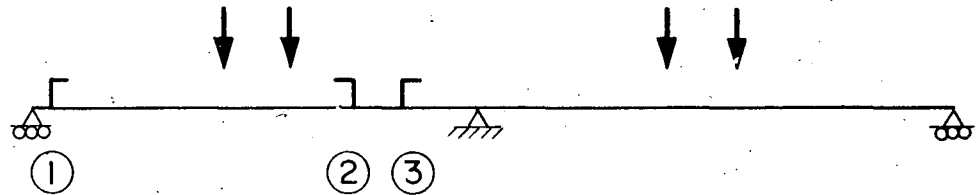
B10



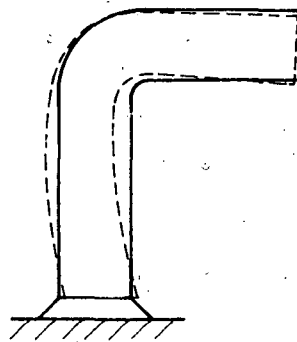
B11



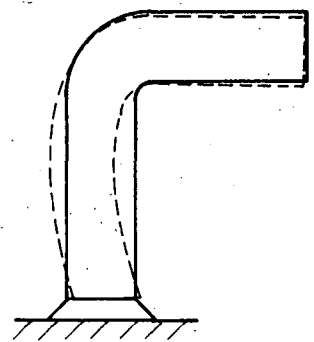
B12



①



②



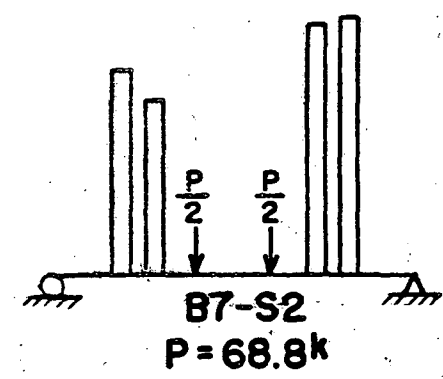
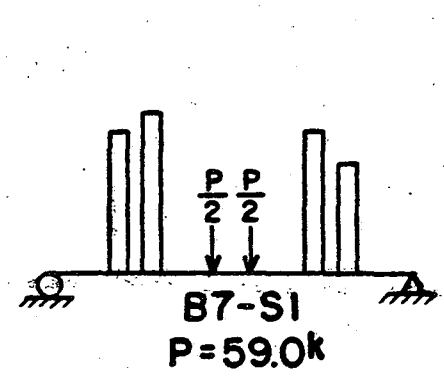
③

B13

Typical Connector Failures

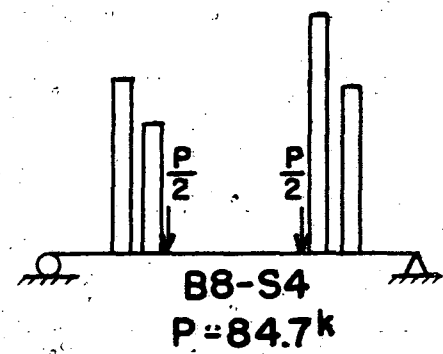
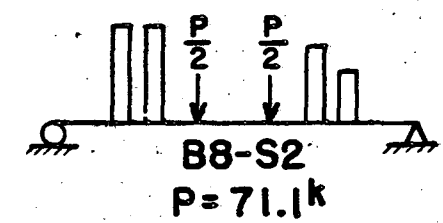
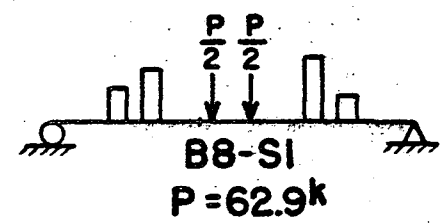
Fig. 24 - Deformed Shape of Connectors After Failure

SEPARATION (IN)
0.040
0.030
0.020
0.010
0

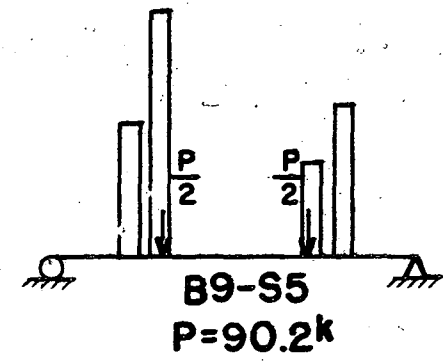
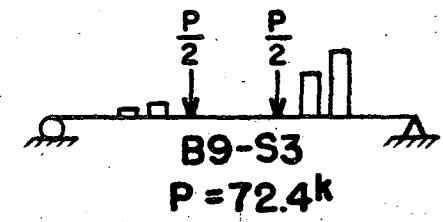
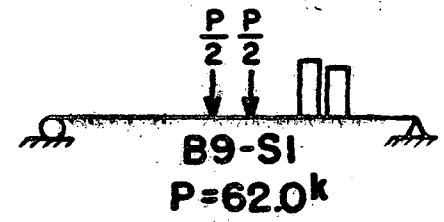


HORIZONTAL SCALE
 $\frac{1}{8}'' = 1'-0''$

SEPARATION (IN)
0.040
0.030
0.020
0.010
0

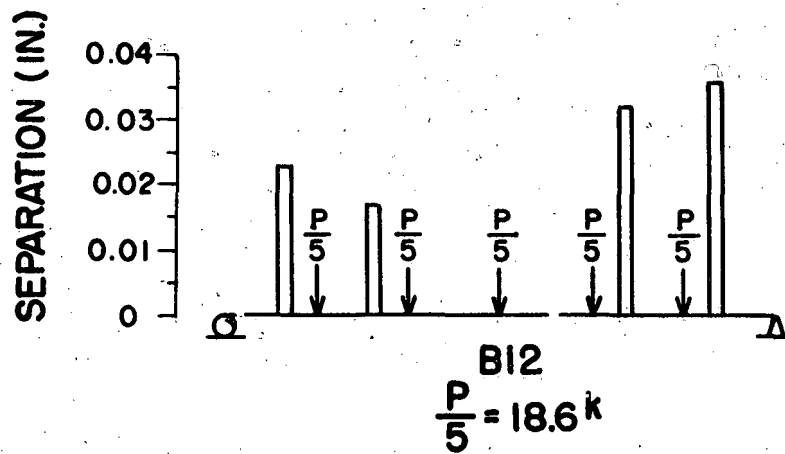
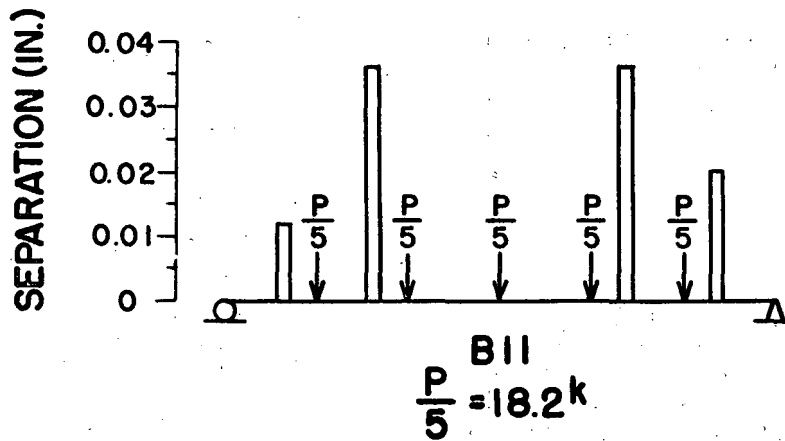
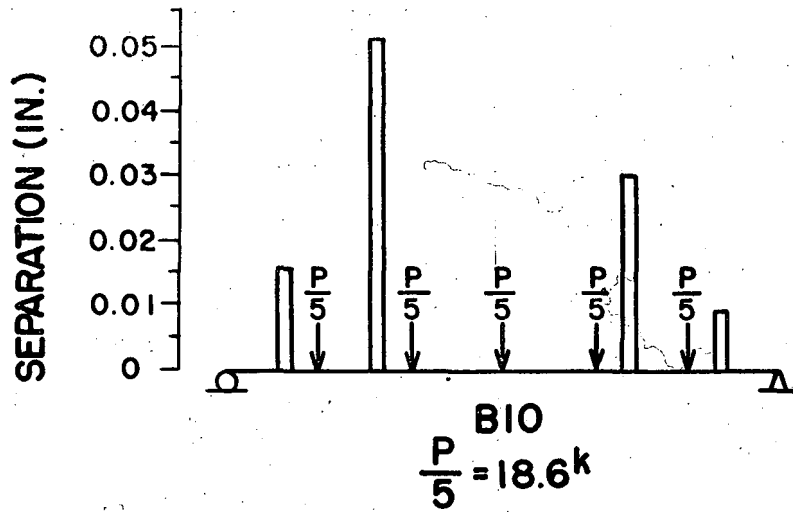


SEPARATION (IN)
0.040
0.030
0.020
0.010
0



Separation of Slab and Beam

Fig. 25 - Separation of Slab and Beam



HORIZONTAL SCALE $\frac{3}{16}'' = 1'-0''$

Separation of Slab and Beam

Fig. 26 - Separation of Slab and Beam

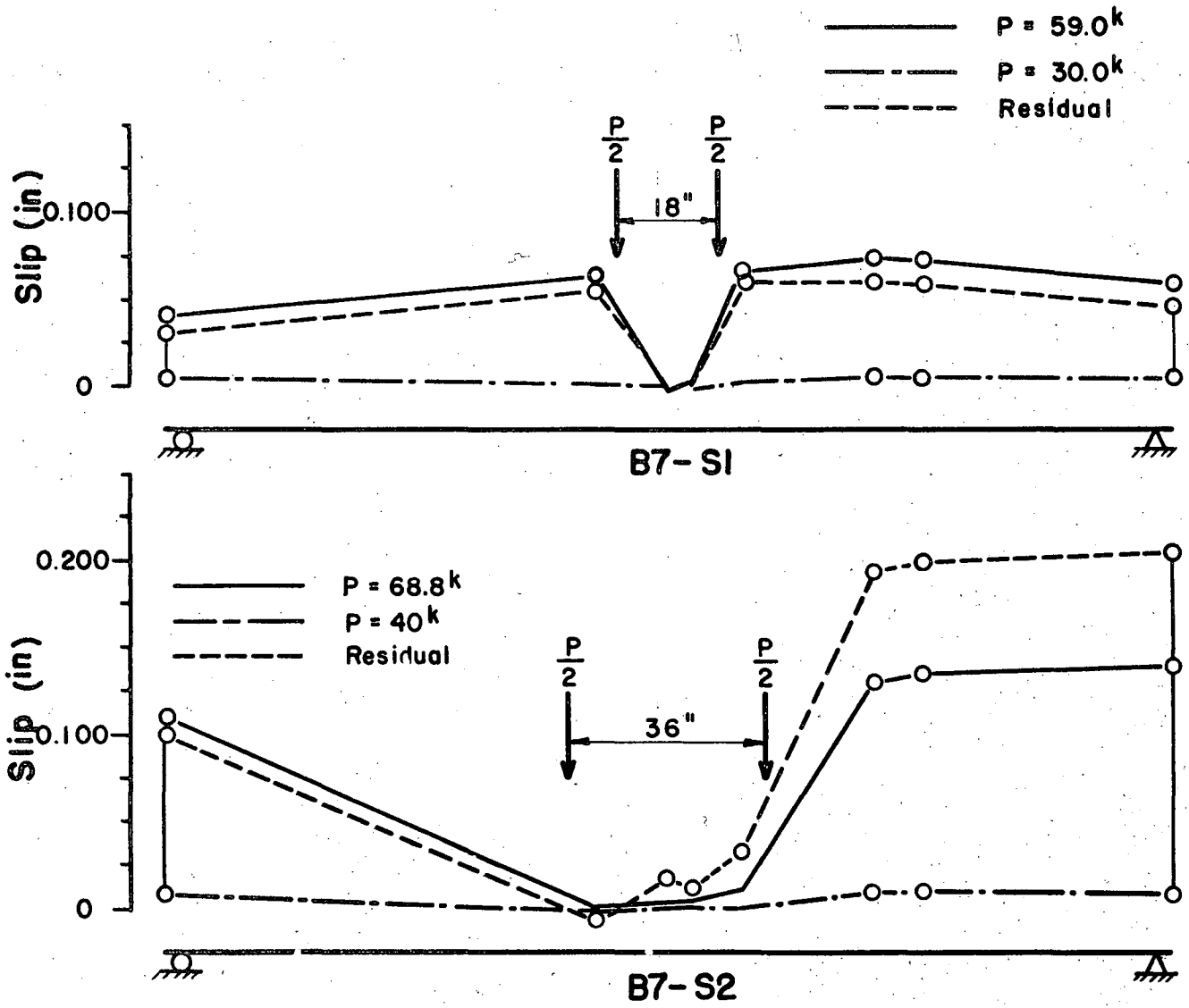


Fig. 27 - Slip Distribution Along Beam B7

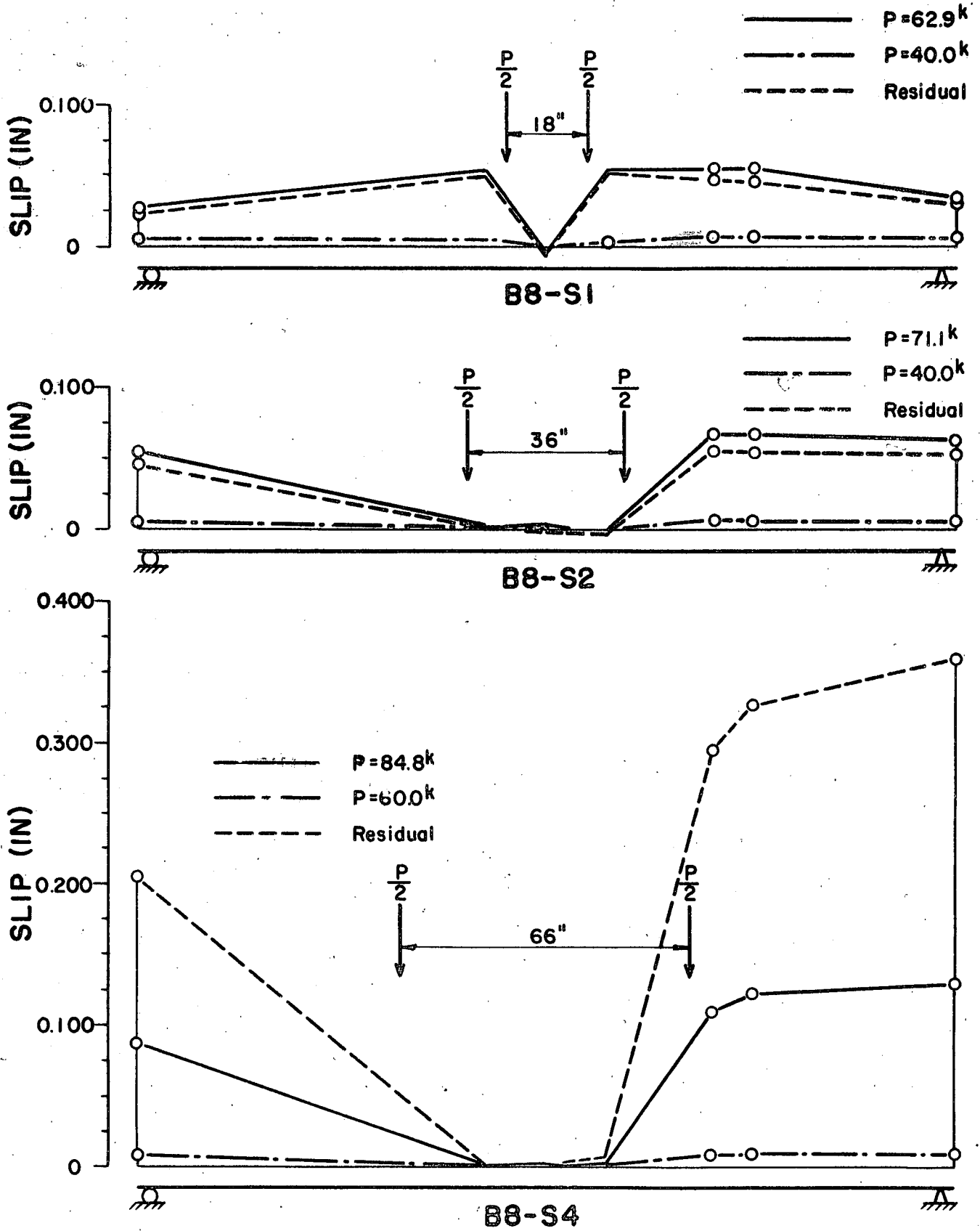


Fig. 28 - Slip Distribution Along Beam B8

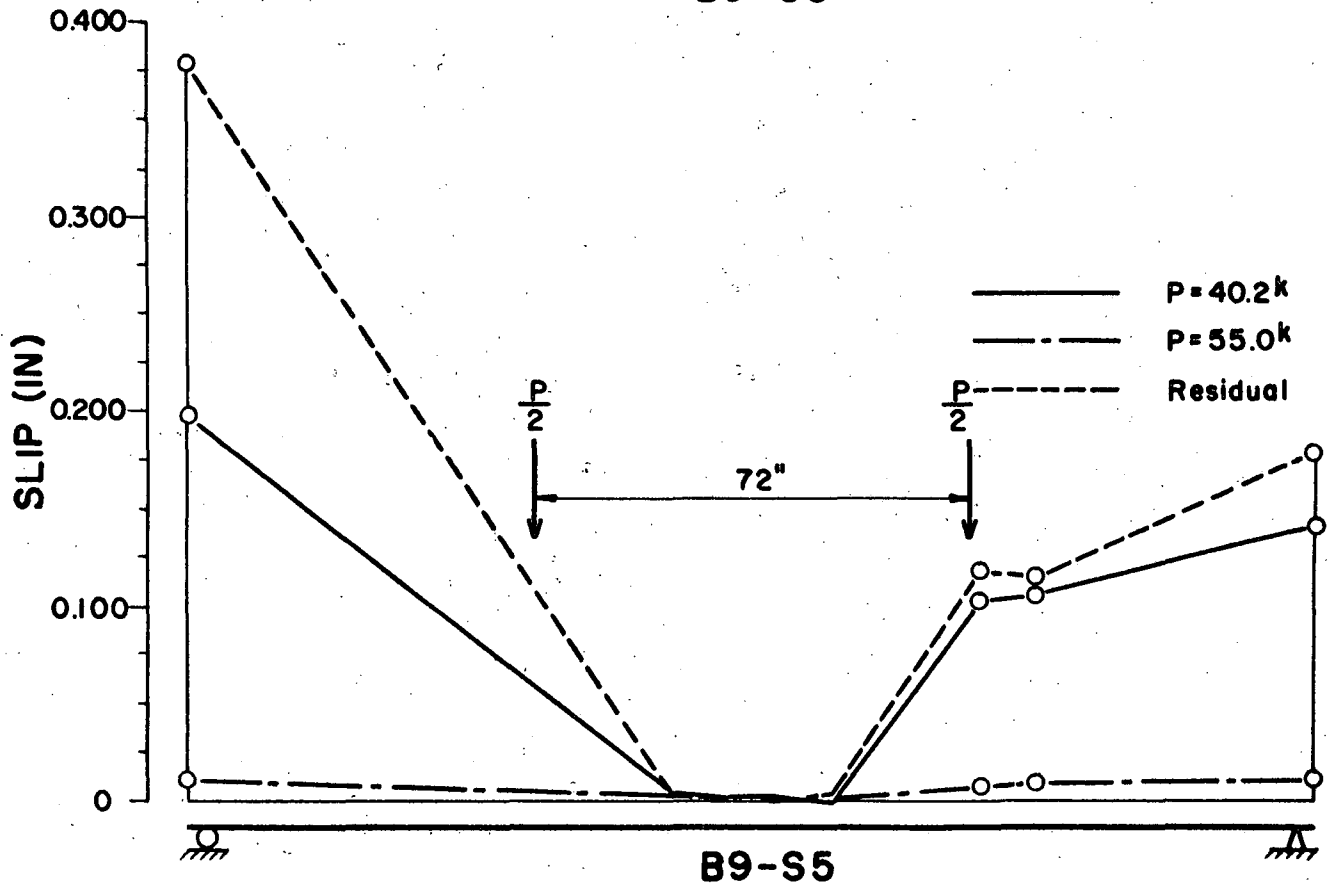
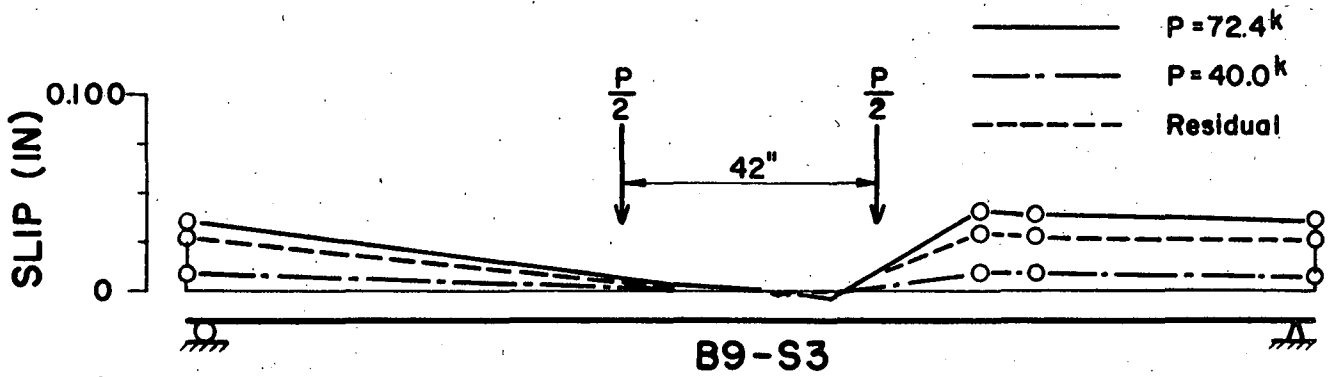
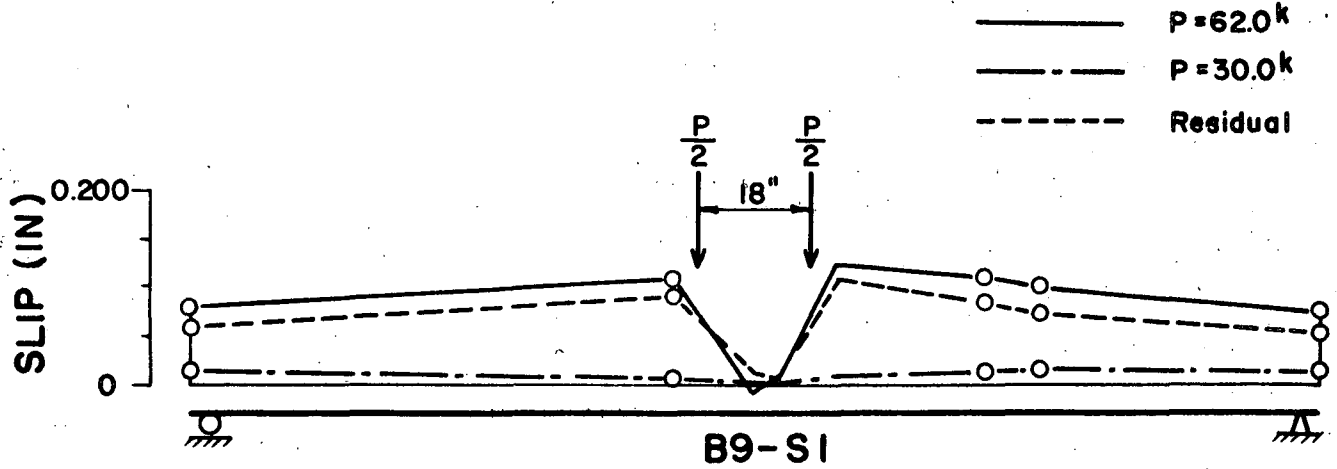
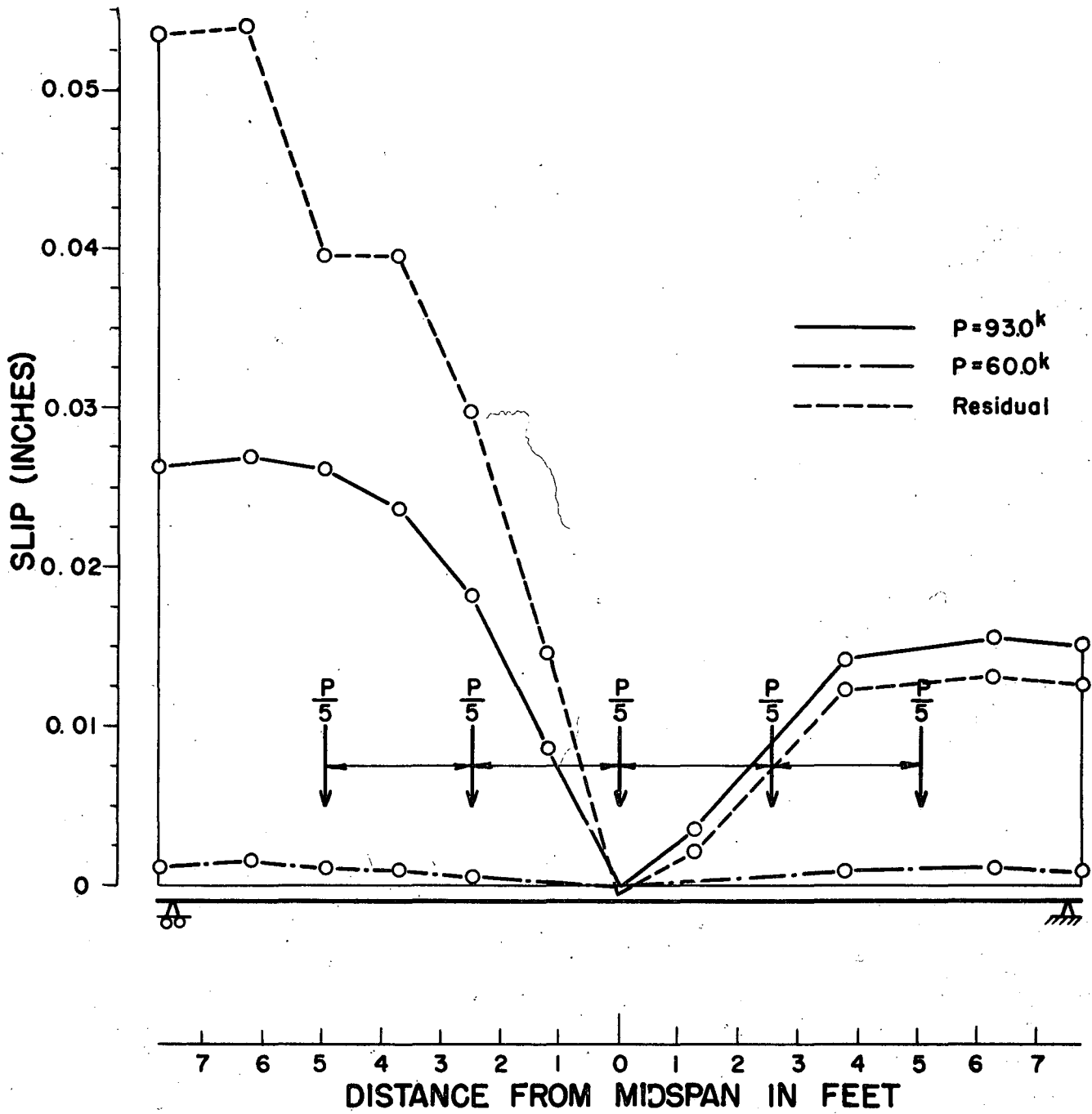
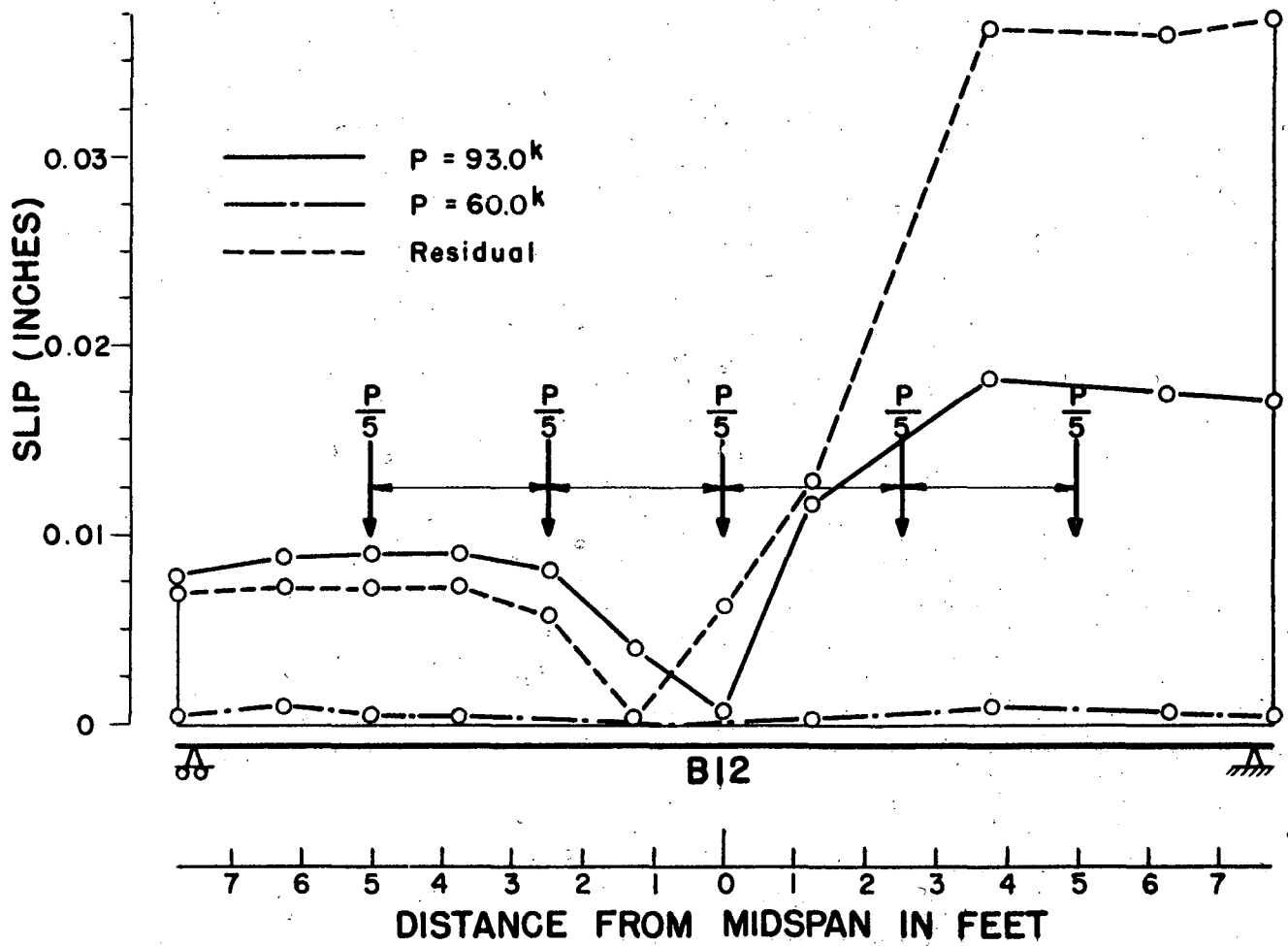
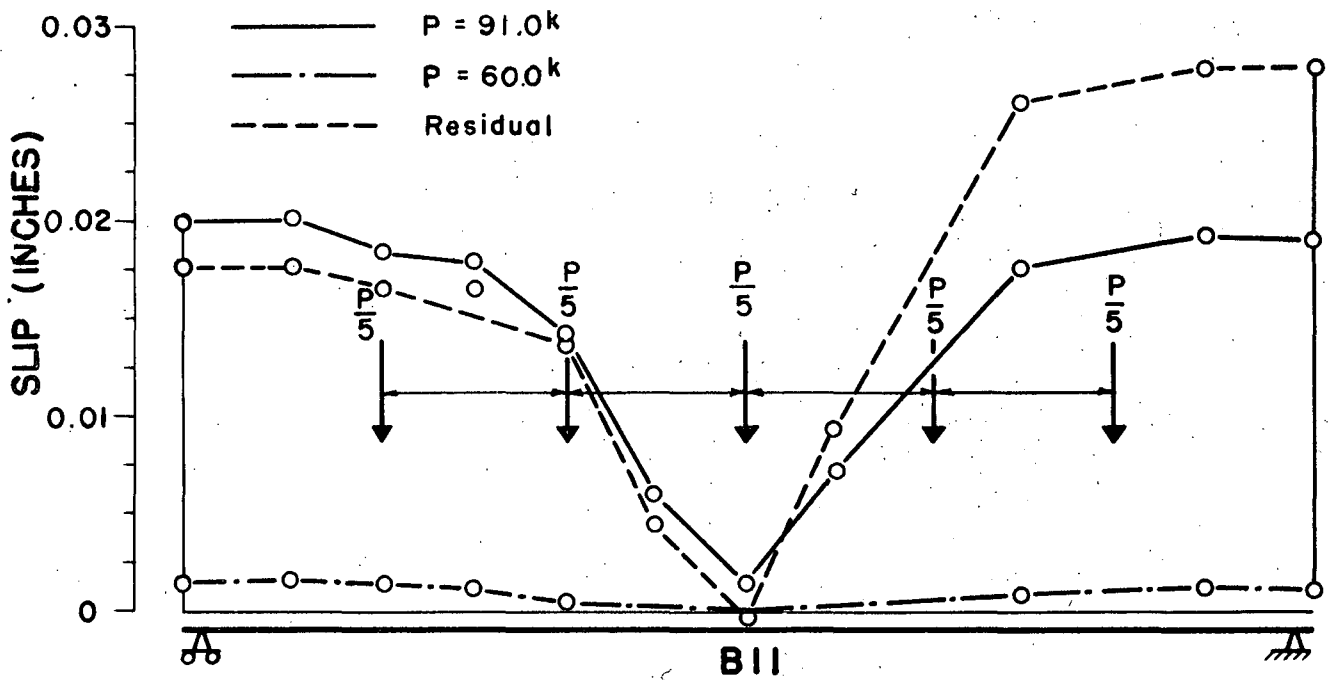


Fig. 29 - Slip Distribution Along Beam B9



Slip Distribution for B10

Fig. 30 - Slip Distribution Along Beam B10



Slip Distribution

Fig. 31 - Slip Distribution Along Beams B11 and B12

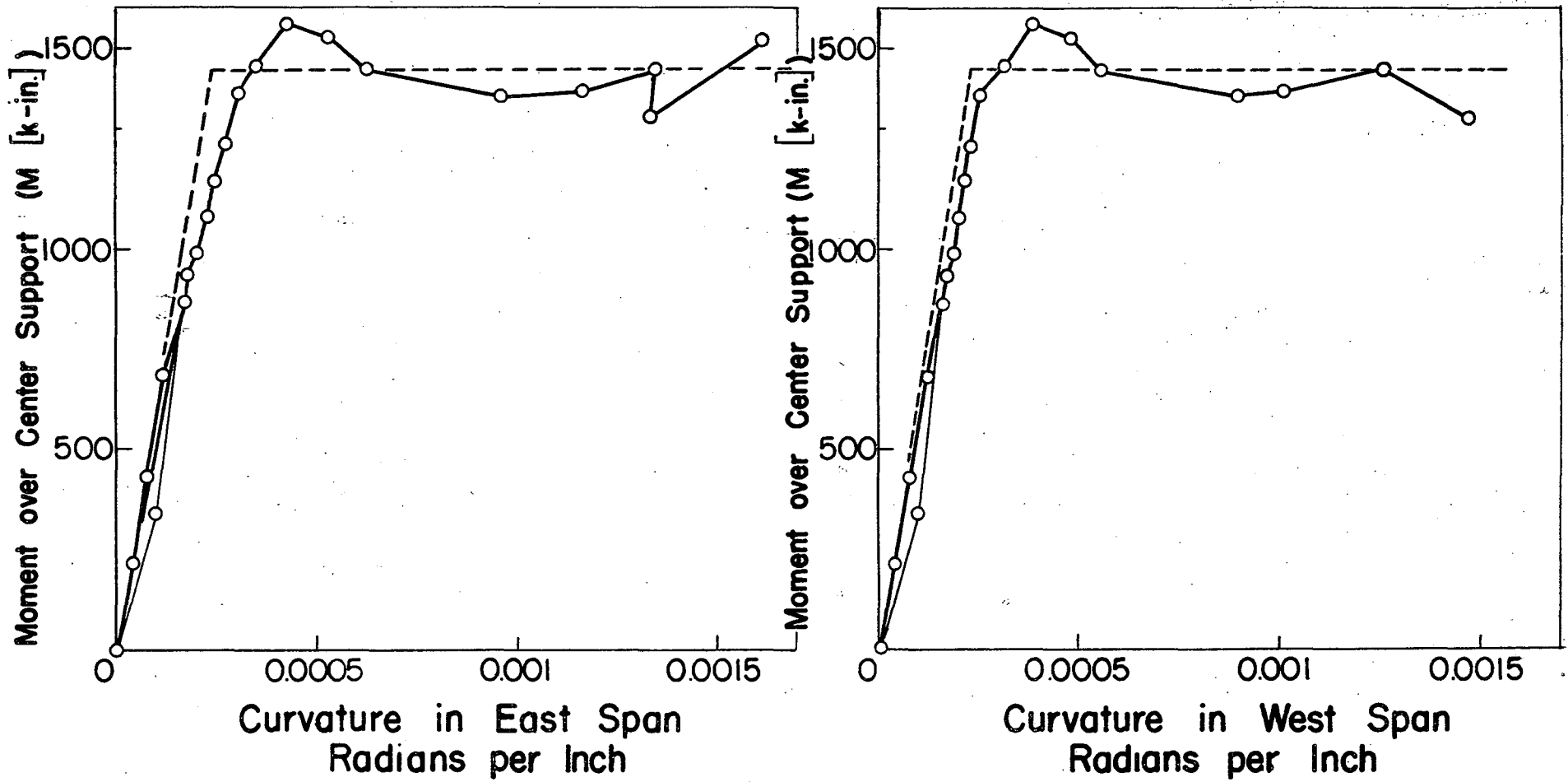


Fig. 32 - Moment Curvature Relations for Continuous Beam B13



Fig. 33 - Cracking At End of Slab Around
Connectors on Beam B 9

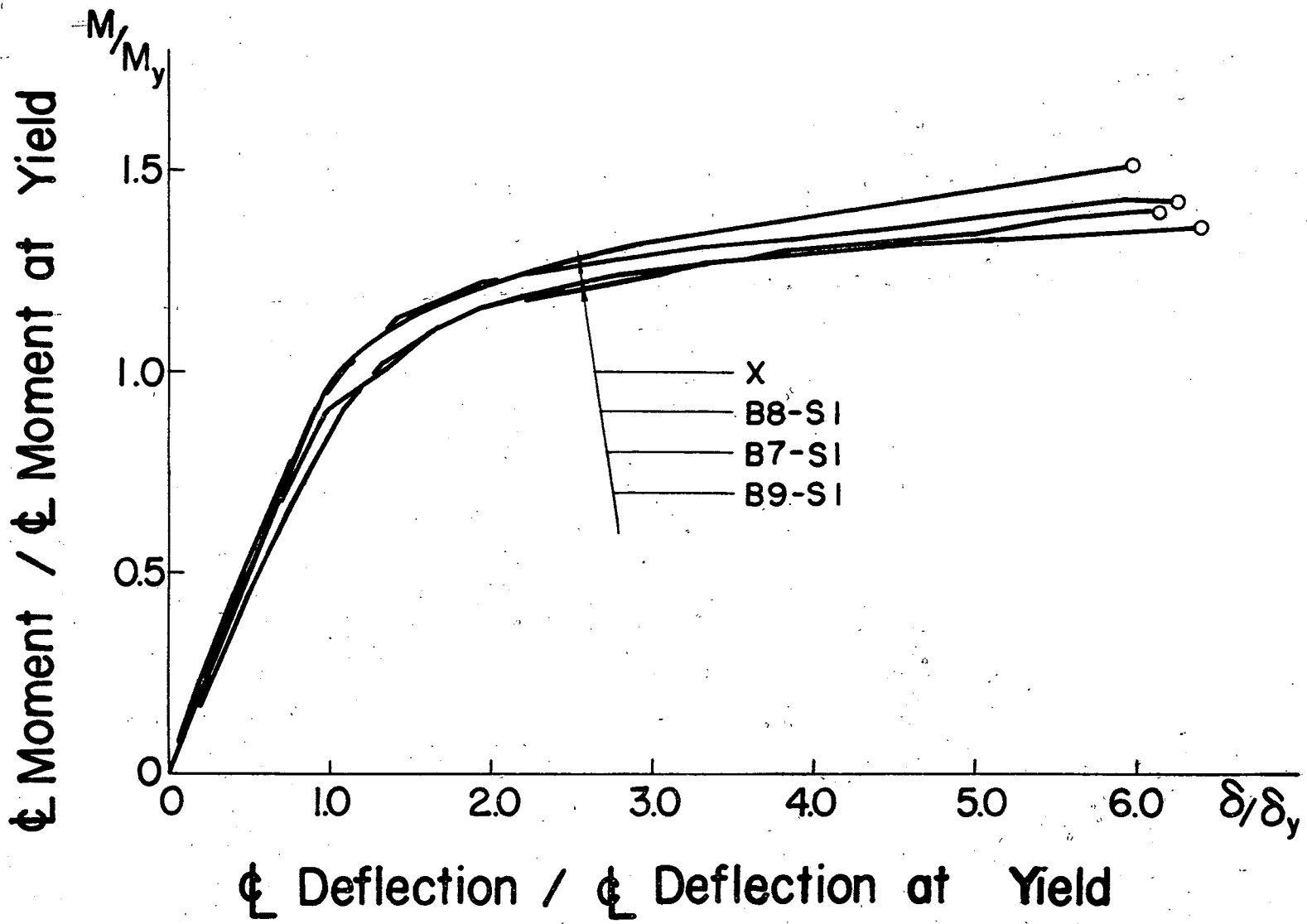


Fig. 34a - Comparison of Beam Tests B7, B8, and B9 with First Load Spreading

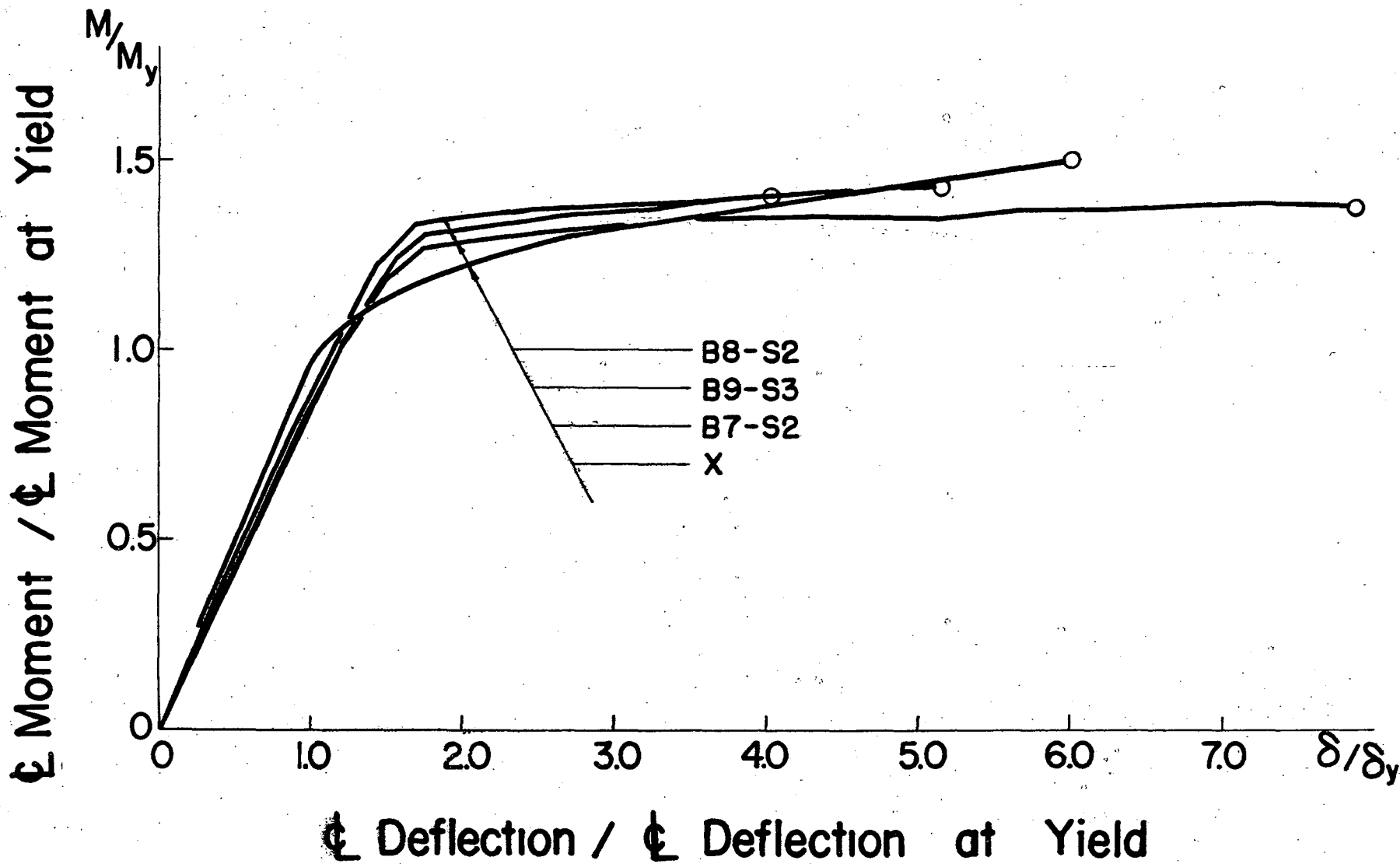


Fig. 34b - Comparison of Beam Tests B7, B8, and B9 with Load Spreading Giving Balanced Shear and Moment

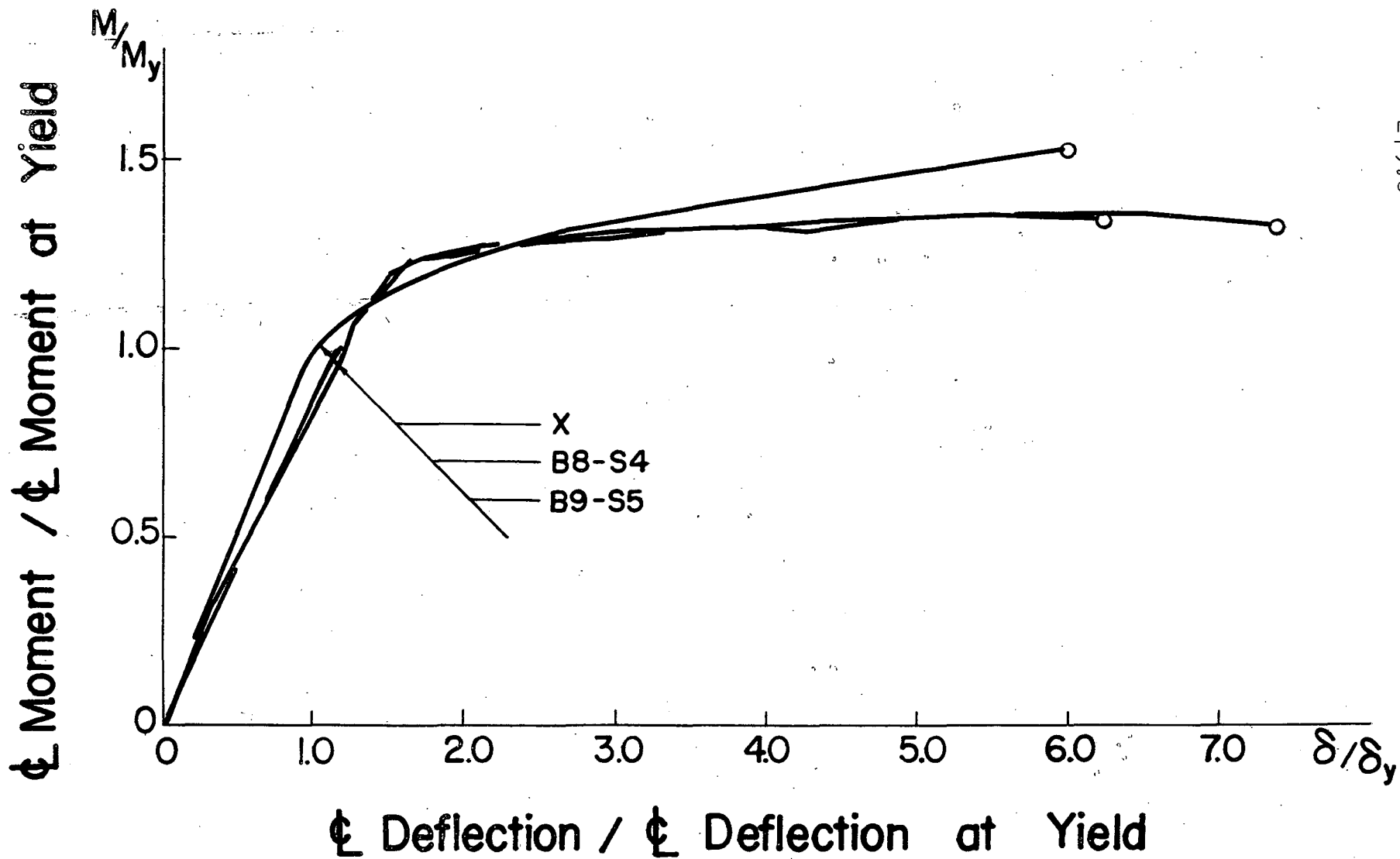


Fig. 34c - Comparison of Beam Tests B8 and B9
With Load Spreading Giving Excess Shear

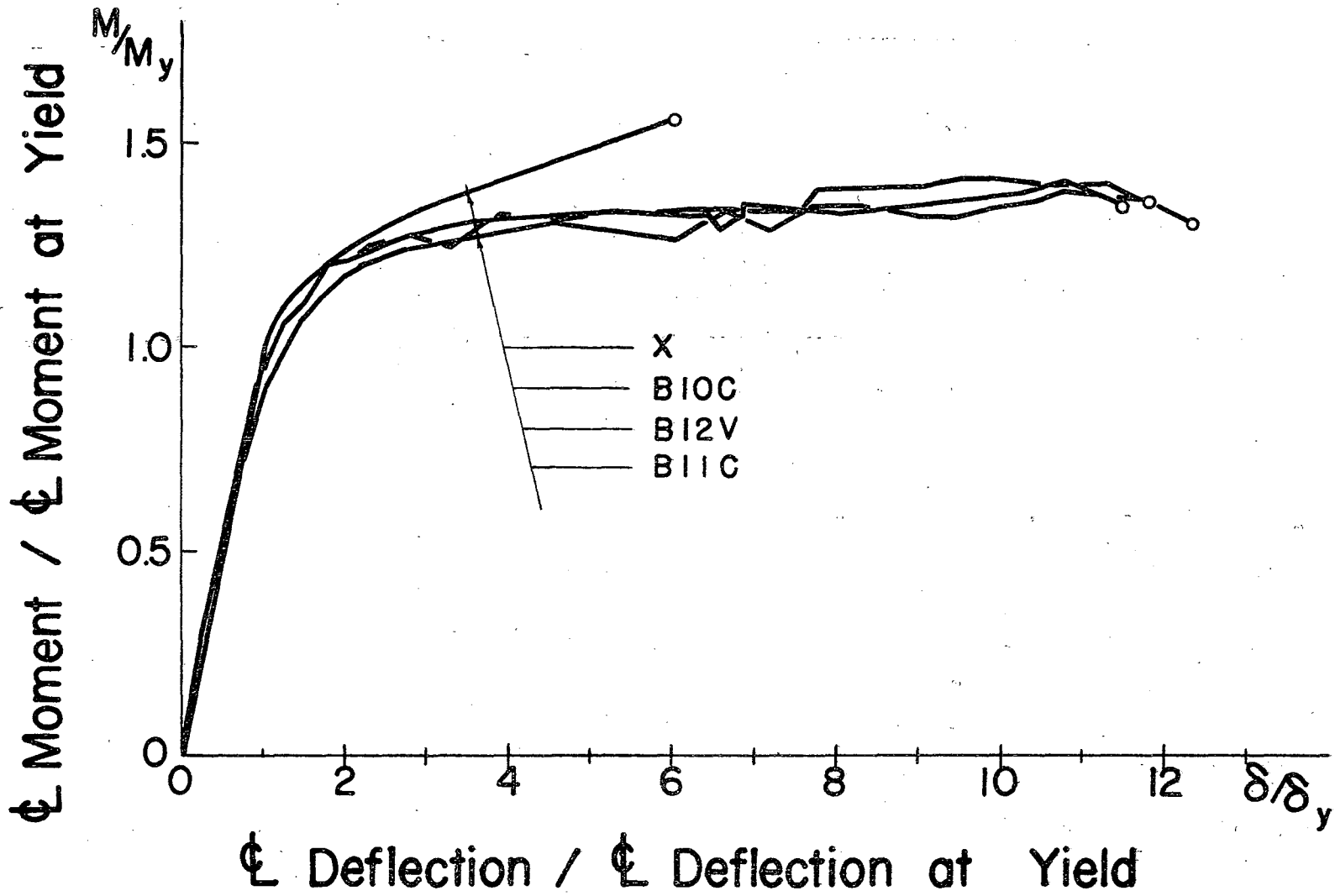


Fig. 35 - Comparison of Beam Tests B10, B11, and B12

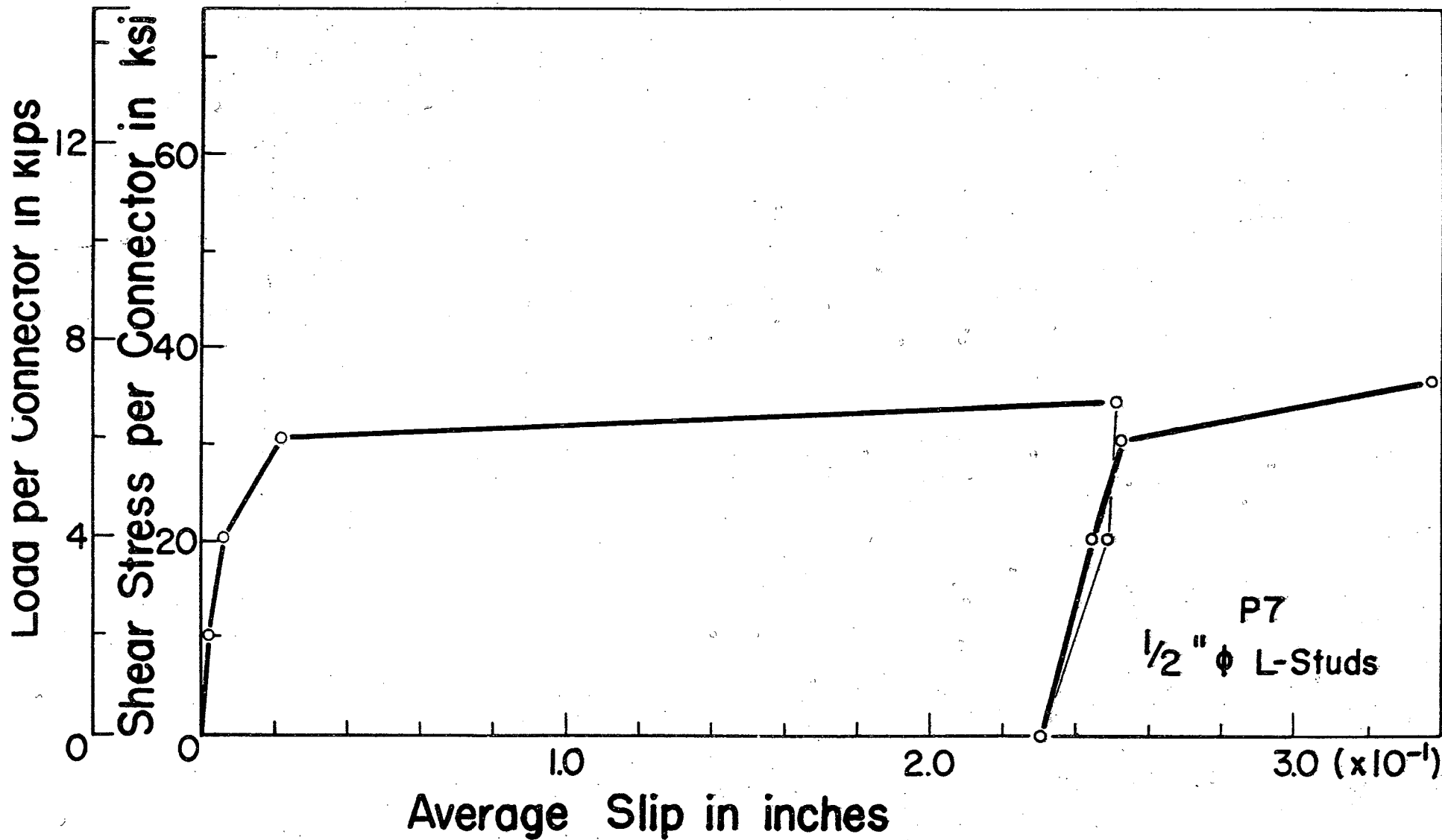


Fig. 36 - Load Slip Curve for Pushout Specimen P7

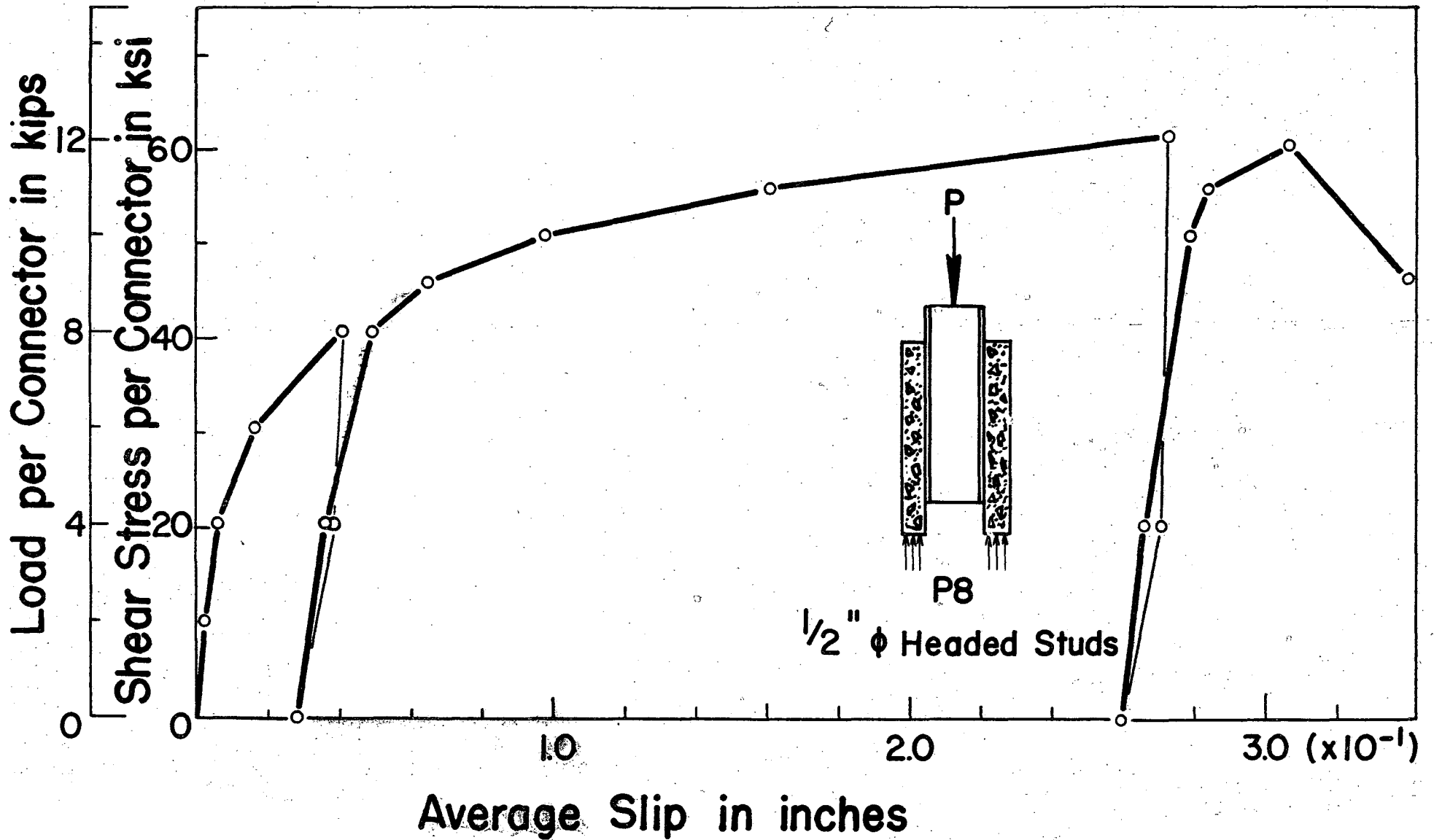


Fig. 37 - Load Slip Curve for Pushout Specimen P8

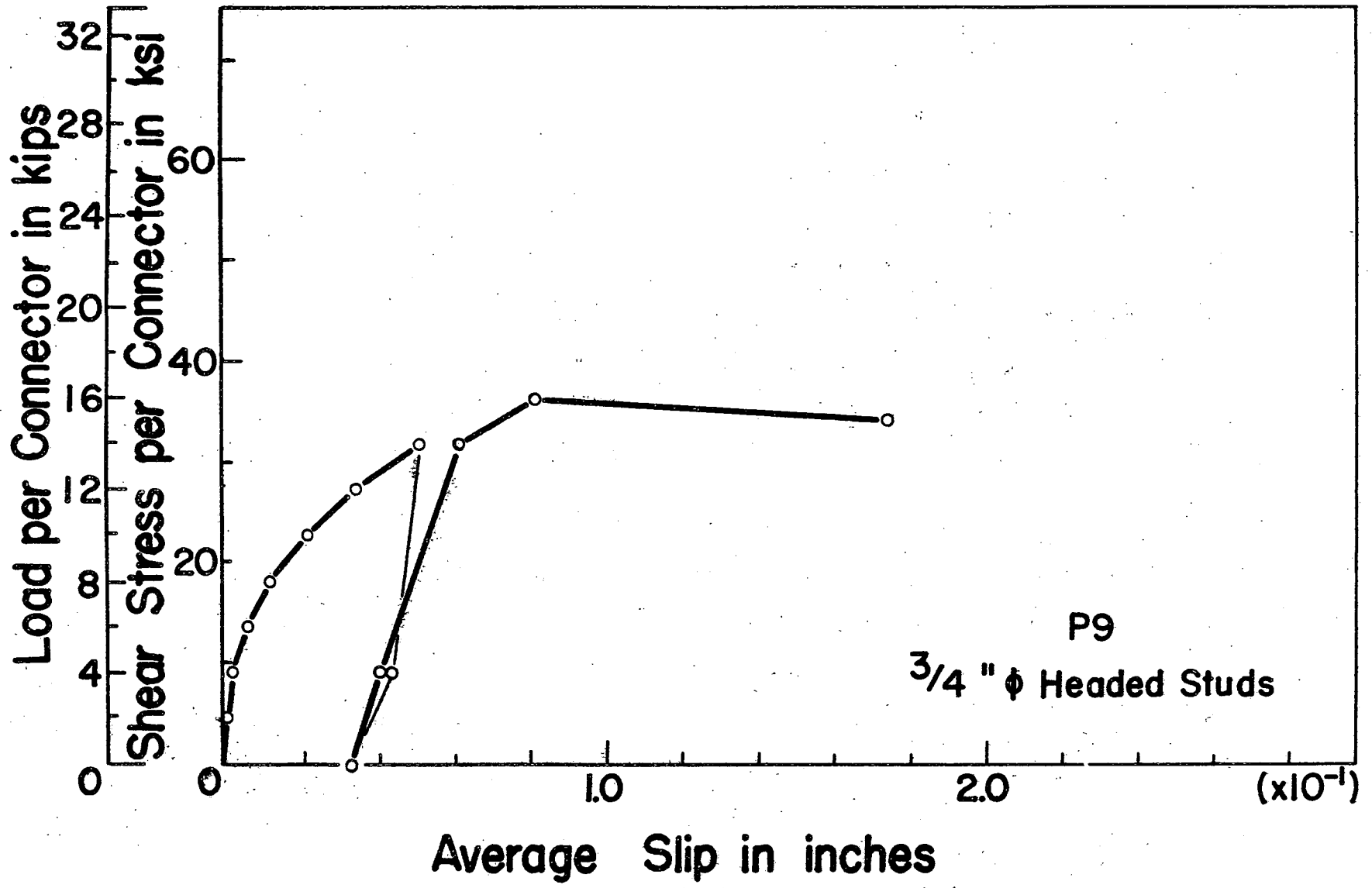


Fig. 38 - Load Slip Curve for Pushout Specimen P9

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