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TESTS OF BEAM-TO-COLUMN WEB MOMENT CONNECTIONS

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A testing program involving steel beam-to-column web moment connections was recently conducted at Lehigh University. The connections studied had the beam attached to the web side of a wide-flange column by different combinations of bolting and welding with the beam loads tending to bend the column about its weak axis. Four subassemblages each consisting of a W14x246 column and a W27x94 beam on one side of the column only were tested to ultimate under simulated static loading conditions by applying a load to the beam while the column was experiencing an axial load. Presented is a description of the testing program, load versus deflection plots and a description of the failure modes of the four connections. Conclusions are drawn regarding the performance of the assemblages.

TESTS OF BEAM-TO-COLUMN WEB MOMENT CONNECTIONS

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INTRODUCTION

Background

One of the most influential elements in the behavior and cost of multistory steel building frames is the moment resisting beam-to-column connection. A majority of these connections are column flange connections where the beam frames into the column flange. Considerable research work has been done on this type of connection at Lehigh University (1,2,3) as well as many other research institutions [(5), for example].

However, another type of moment resisting connection commonly found in building frames is the web connection. In this connection, the beams attached to the column perpendicular to the plane of the column web (Fig. 1). The action of the beam bending moment tends to bend the column about its weak axis. It is a study of this type of connection which is currently underway at Lehigh University.

The previous research done on web connections in the United States has been limited to static testing of symmetric web connections (4) and testing of unsymmetrical web connections under repeated and reversed loading (5) with no axial force. The current research work at Lehigh University centers on a study of unsymmetrical web connections where there is only a beam on one side of the column (Fig. 1) and where there is axial load applied on the column. This is a more severe type of loading on the beam and column

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assemblage than the symmetrically loaded connections previously studied. This study is under the guidance of the Welding Research Council Task Group on Beam-to-Column Connections.

Scope

The entire study of beam-to-column web connections was divided into two distinct phases of activity. Each phase consisted of both experimental and theoretical investigations.

The first phase of activity was called the pilot test program. In attempting to organize a comprehensive research program of web connections, it was felt that, by isolating certain variables, a better insight into different aspects of connection behavior could be obtained. Since the study centers around moment-resisting web connections, the critical variable chosen to be examined prior to the development of full-scale connection assemblages was the effects that concentrated beam bending forces have on a column when applied in a way to simulate a web connection. For the pilot study, the effects of column axial load and beam shear on the behavior of the web connections were ignored. The main purpose of the pilot tests was to gain knowledge to attain the proper design of the full-scale specimens. These pilot tests were planned to provide answers to questions concerning members sizes, connection geometry and stiffener requirements. The details of the testing program and test results are reported in Ref. 6.

The second phase, and the emphasis of this paper, was the testing of four full-scale web connection assemblages. Each assemblage consisted of an 18 ft long column and a beam approximately five feet long connected at midheight of the column. Four different geometries of welding and bolting the beam to the column were tested. These connections simulate actual building connections with the beam transmitting shear and moment to the column and the column being acted upon by an axial load. Presented herein will be a summary of the results of this testing program. Theoretical investigations are currently being undertaken and will be combined with the test results to formulate design recommendations at a future time.

Objective

For the beam-to-column web connection assemblage shown in Fig. 1, theoretically, the maximum strength of this assemblage is reached when plastic hinges form at sections X and Y in the column or in the beam at the beam-to-column juncture. However, there exist other factors that may limit the maximum strength of such connections. For example, if the beam flange is narrower than the distance between column fillets, and the beam is welded directly to the column web, a yield line mechanism may form in the column web before the formation of any plastic hinges. This depends upon the width of beam flange, depth of beam and column web thickness. Even if the attachment of the beam to the column is such that the yield line mechanism will not form, the maximum load based on simple plastic theory might not be attained due to high stress concentrations or a lack of connection ductility leading to fracture of the material. If it is necessary to carry a beam load larger than that obtained at the occurrence of the above described limits to load carrying capacity, the concept of stiffening of the connection must be considered.

Since lateral deflection is very important in the design of tall buildings, web connections must also be looked at from a deformation standpoint. If such a connection lacks sufficient stiffness in the working load range, due to large deformations of components or to localized yielding, even if it has the ability to achieve a load required to form a plastic hinge, its value as a structural component is suspect. Again, stiffening may have to be examined as a means of increasing the stiffness of the joint.

Thus the overall objective of the study is to examine web connections from the viewpoint of strength, stiffness, and ductility and to consider connection stiffening when required to attain the desired connection load or stiffness. The ultimate objective is the formulation of guidelines for the design of such connections.

TESTING PROGRAM

General

The test program consisted of four different connection geometries. The connections were full-scale using realistic beam and column sections, unsymmetrically loaded by an increasing monotonic load to simulate static conditions. A complete description of the testing program is given in Ref. 7.

The specimens were designed according to the AISC Specification (Ref. 8). The connections were proportioned to resist the moment and shear generated by the full factored load. Since the loading condition resembles gravity type loading (dead load plus live load) the load factor used was 1.7. The stresses used in proportioning welds, shear plates, and top and bottom moment plates were then equal to 1.7 times those given in Sec. 1.5 of the AISC

Specification. For A490 high-strength bolts in bearing-type connections, the design shear stresses used were equal to 1.7 times 40 ksi, instead of 32 ksi as suggested in the current Specification.

The specimens were fabricated from ASTM A572 Grade 50 steel. This steel was selected due to its increased use in building design and because there is a narrower margin between yield and ultimate for this high strength steel than for a lower grade steel. Thus, if the web connections perform well using the high strength steel, similar connections using mild steel should also perform well.

The column and beam sizes were the same for all four specimens. The column was a W14x246 and the beam was a W27x94. These connection components were chosen so that there would be a realistic combination of members to simulate a connection in a multistory frame.

The column section was chosen to avoid a failure in the column outside of the connection region. This section was arrived at after considering both stability of the 18-foot long column under axial load and bending as well as yielding of the columns cross section above and below the beam. Since in the test setup the column was to have fixed ends, the column length of 18 feet was chosen so that the distance between column inflection points was twelve feet, a figure fairly reflective of buildings currently being designed.

The beam lengths varied for the four specimens. The connections were proportioned such that the beam section at the beam-to-column juncture could resist the beam plastic bending moment M_p and 81% of the beam shear, V_p, required to cause shear yielding of the beam web. This section is called the critical section and is different for the various connections. (The locations of the critical section for the four connections will be given in a later paragraph.) The beam span to the critical section from application of the beam load is then simply the ratio of M_p to 81% V_p. For the beam section used, the length of the beam from application of load to the critical section was 48 inches.

In connections where some of the elements were bolted, ASTM A490 bolts were used to assemble the joint. All bolted joints were designed as bearing-type using an allowable bolt shear stress for A490 bolts of 40 ksi. The use of this higher allowable shear stress has been proven satisfactory when used in previous beam-to-column connection studies (Ref. 9).

Although oversize and slotted holes are desirable to facilitate erection adjustments, the effect of using holes cut in this manner on the performance of beam-to-column connections has already been shown (Ref. 9). It has been observed that slotted holes do not affect the strength of bearing-type joints. For this reason, it was decided to use round holes 1/16 in. larger than the bolt diameter to assess their effects on the behavior of bolted beam-to-column connections. Holes were punched, sub-punched and reamed, or drilled as required by the AISC Specification. All bolts were installed by the turn-of-nut method.

The connection specimens were welded according to the AWS Building Code (Ref. 10). For fillet welds, the weld electrodes were E70XX. In determining the size of the fillet weld, the design shear stress on the effective throat was 1.7 times 21 ksi. The full penetration groove welds were made using the flux-cored arc welding technique. All welds were checked ultrasonically for defects.

Test Setup

A schematic view of the test setup for the full-scale tests is shown in Fig. 1. Figure 2 is a photograph of the setup taken during testing. The assemblage was placed in the 5 million pound universal testing machine with that machine applying a constant axial load to the column. An axial load was applied to the column to have as realistic an assemblage as possible. It was found (Ref. 11) that axial load has an effect on yielding and deformation of connections. The lower end of the column was bolted to the floor and the upper end was held in a fixed-end condition position by the testing machine head and bracing beam. An upward load to the beam was then applied by a hydraulic jack in increments to simulate static loading.

After it was placed in the testing machine and properly aligned, the column was loaded in 250 kip increments to a load of 1520 kips. This is equal to the value of the column axial load P obtained from $P/P_y = 0.5$ (1810 kips) minus 290 kips. The value of 290 kips is the beam load (V) calculated to cause M_p in the beam at the critical section. P_y is the axial load required to cause yielding in the column. Both P and V values were calculated using nominal yield stress values.

The beam was then loaded in increments of approximately 25 kips until deflections became excessive, at which time a deflection increment was applied. The value of the column load as applied by the upper head of the testing machine was adjusted at each increment to read 1520 kips plus the beam load V.

Thus, the column in the top half of the assemblage had an axial load of P+V and the column in the bottom half had a value of P. Once the theoretical plastic moment of the beam was attained at the critical section, the value of the axial load in the upper column was equal to the desired value of $P/P_y = 0.5$. If the value of V exceeded 290 kips, the axial load in the column was allowed to increase beyond $P/P_y = 0.5$. Thus, the test assemblage simulated an inverted assemblage of a building frame where the load on a particular floor level increases the load on the column below that level relative to the column above.

Description of Specimens

Specimen 14-1 shown in Fig. 3 was a flange-welded web-bolted connection. The beam flanges were groove welded to the flange moment plates which, in turn, were fillet welded to the column web and flanges. A one-sided shear plate bolted with seven 7/8 in. diameter A490 high-strength bolts was used to resist vertical shear. The shear plate was fillet welded to both the column web and flange moment plates. Round holes 1/16 in. greater than the bolt diameter were used in the web plate and beam web. The flange moment plates were3/4 in. thick which is the thickness of the beam flanges, and the web plate was 1/2 in. thick, which is the beam web thickness. The critical section here was at the column flange tip approximately eight inches from the centerline of the column web. This then provided a beam span from the centerline of column of 4'-8" to the application of the beam load.

Specimen 14-2 (Fig. 4) was also a flange-welded web-bolted connection. The beam flanges were welded directly to the column web by a full penetration groove weld. The beam web was bolted directly to the column web by a pair of back-to-back angles to resist shear. The angles were 3½x3½x3/8 in. and the bolts were eight 3/4 in. diameter A490 high-strength bolts. The angles were fillet welded to the web of the beam. Here, the critical section was at the centerline of the columns giving a beam span length of 4'-0".

Shown in Fig. 5 is connection 14-3, a fully bolted connection. Top and bottom moment plates were bolted to the beam flange by ten 1 in. diameter A490 high-strength bolts in 1-1/16 in. round holes. These moment plates were fillet welded to the column web and flanges. This connection was designed as a flange bearing connection. The beam web shear attachment was the same as Specimen 14-1. Here, the critical section was taken as the outer row of flange bolts giving a beam span length of 5'-10".

Specimen 14-4 shown in Fig. 6 was a fully-welded connection and was used as a control test. The connection was similar to 14-1 in that the beam flanges were groove welded to the flange moment plates which in turn, were fillet welded to the column web and flanges. However, in this connection, the beam web was groove welded to the shear plate to transfer the beam shear. The web shear plate was again fillet welded to both the column web and flange moment plates. The beam web was welded to the web shear plate after being held in position by three 3/4 in. A307 erection bolts. As in 14-1, the critical section was at the column flange tips with a similar beam span length of 4'-8".

TEST RESULTS

Connection 14-1

The load-deflection curve plot of beam load V vs. beam deflection Δ is given in Fig. 7. The connection initially exhibited a definite linear-elastic V-Δ slope. The effect of yielding of the assemblage was indicated by a reduction of stiffness at higher load levels. The failure of this specimen occurred at a beam load of 273 kips which is 94% of the beam load V_{mp} calculated to produce the beam plastic moment M_p at the critical section. Since most design and analysis of beams use span lengths from center to center of column, the percentage of computed M_p attained on this basis is much higher. If the centerline of the column were to be taken as the critical section, the load level reached was 109% of the load calculated to produce M_p. The beam deflection at the maximum load was 2.07 inches.

Failure of this specimen was due to tearing across the entire width of the tension flange connection plate in the region of the transverse groove weld as shown in Fig. 8. The failure was instantaneous with no evidence of tearing prior to the last load increment. The beam load dropped to zero immediately with no opportunity to observe an unloading slope for the connection. Figure 9 gives a view of the panel zone of the connection showing the extent of yielding at the conclusion of testing.

The elastic theoretical slope shown in the graph in Fig. 7, and later graphs, for comparison, is based upon accounting for beam bending, beam shear deformations and joint rotation and does not include items such as loss of column stiffness due to axial load, or the effect of small end rotations at the top of the column. The actual test curve is very close to the theoretical stiffness in the elastic range. The theoretical horizontal line is the beam shear required to cause the plastic moment M_p in the beam at the critical section and is based on nominal yield strength values (for the four connections, the yield strengths of material from all beams were well above nominal).

Connection 14-2

Figure 10 shows the beam load vs. beam deflection plot for connection 14-2. The curve has a definite linear elastic V-Δ slope up to a load of approximately 100 kips. The effect of yielding of connection components is indicated by the nonlinear behavior at higher loads. This nonlinear behavior was primarily due to yielding of the column web under the action of the beam flange forces, the column web alone having to resist the beam bending forces because the beam flanges were not attached to the column flanges.

The maximum load in this specimen was 205.4 kips which was 71 percent of V_{mp} at the critical section, which in this case is taken to be the centerline of the column web. The failure of this specimen was indicated by two related events. First, at a beam load of 195 kips, the column web fractured on one side of the beam tension flange where the beam was welded to the column web. The fracture did not completely penetrate the column web but caused a redistribution of stress in the beam tension flange. The fracture caused an increase in stress (and related strain) on the portion of the beam still intact with the column web. Ultimate failure then occurred at a load of 201.9 kips when a portion of the weld still connecting the beam flange to the column web fractured. Since the fracture did not proceed across the entire beam flange, the load did not drop off completely, but no further loading was attempted. The maximum beam deflection attained was 1.58 inches.

Shown in Fig. 11 is a view of the beam tension flange showing the fracture in the region of the beam flange-to-column web groove weld. The severe deformations to which the column web was subjected are visible in Fig. 12 which is a view of the opposite side of the column. The beam tension flange is in the lower part of the photo.

The elastic stiffness of the tested connection does not compare well to the theoretical stiffness, calculated as described earlier. The major difference between the two curves is the deformation of the column web. Without stiffening present on the opposite side of the column, the flexible column web deforms significantly out-of-plane due to the action of the beam flange forces.

Connection 14-3

The beam load V vs. beam deflection Δ for connection 14-3 is given in Fig. 13. The plot shows an initial linear elastic slope up to approximately 90 kips and then a secondary linear slope up to a load of 200 kips. This general type of behavior of two distinct slopes agrees quite favorably with the results of tests on bolted connections recently conducted at Lehigh University (Ref. 12). The second linear slope is due to many minor slips of the bolted flange plates into bearing. There was no one major slip during the test of this connection or during previous beam-to-column connection tests. The load-deflection curve then gradually loses stiffness due to yielding of elements within the assemblage. The maximum load attained on this test was 289 kips which is 100 percent of the beam load required to cause M_p at the critical section (145% if the critical section is taken as the centerline of the column web). During the next load interval, a tear developed in the tension flange connection plate as shown in Fig. 14 and the load dropped to 249 kips. The load reached a value of approximately 300 kips before the tear occurred. No further loading was attempted and the connection was completely unloaded.

As in 14-1, the initial elastic slope compares favorably to the theoretical slope. However, the secondary elastic slope due to beam flange plate bolt slip shows considerable reduction in stiffness. The fact that this stiffness reduction occurs in what would be the working load range is of considerable importance.

Connection 14-4

A curve depicting the beam load V vs. beam deflection Δ behavior is given in Fig. 16. The connection assemblage exhibits a linear load-deflection slope up to a beam load of approximately 150 kips at which time the stiffness is reduced due to local yielding. Again, the elastic slope deviates slightly from the predicted stiffness.

The maximum loading on the specimen was 303.5 kips which is 105 percent of the plastic moment producing beam load at the critical section (122 percent if the critical section were considered to be the centerline of the column). The testing was terminated when, due to the large beam deflection and other deformations, no further purpose would be served by continuing to load. The load started to fall off from its peak value due to out-of-plane deformation of the beam compression flange and the vertical web connection plate. The beam deflection at the end of testing was 3.22 inches. A photo of Connection 14-4 at the conclusion of testing is provided in Fig. 15.

Presented in Table 1 is a summary of maximum beam loads and the maximum load as a percentage of the plastic moment load computed at both the critical section and at the centerline of column. Also given is the maximum deflection in inches and as a ratio of the theoretical deflection (Δ_p) at the start of plastic beam behavior as well as failure mode of the four connections.

CONCLUSIONS

A series of four full-scale beam-to-column moment-resisting web connection assemblages have been tested to observe their behavior under simulated static loading. This testing program in combination with a previous testing program and future theoretical work will provide a thorough understanding of such connections and will lead to recommendations and guidelines for those involved in their design.

The following conclusions can be made regarding the test results of these four assemblages.

1. When considering the maximum beam load evaluated at the column centerline, Connections 14-1, 14-3, and 14-4 all achieved load levels beyond the plastic moment load.
2. The maximum load level of Connection 14-2 was only 71 percent of the plastic moment load.

3. Connections 14-1, 14-2 and 14-4 all exhibited a linear elastic stiffness followed by gradual plastification of subassemblage elements.
4. Connection 14-3 exhibited two linear elastic slopes prior to the start of local connection yielding. The secondary elastic slope was due to minor slips of the bolted flange plates into bearing.
5. The deviation of the actual initial elastic slope from that predicted can be accounted for by the out-of-plane movement of the column web under the action of beam flange forces.
6. The out-of-plane movement of the column web on Connection 14-2 and the resultant reduction in connection stiffness is quite significant and could be an important factor when such connections are used in design.
7. If it is desired to limit the out-of-plane deformation of the column web, especially on the type of connection where the beam is attached only to the column web as in 14-2, column stiffening must be examined.
8. Although bolted connections such as 14-3 exhibit very good strength, the reduction of stiffness in the working load range due to bolt slippage must be considered when such connections are used.
9. The failure of 14-1, 14-2, 14-3 was by fracture of connection material with the fractures in 14-1 and 14-3 occurring after the connection was loaded well into the plastic range. The fractures of all three connections can be related to high stress concentrations.
10. The use of column stiffening as a means of reducing stress concentrations is a definite possibility and should be examined, especially in connections similar to 14-2.

ACKNOWLEDGMENTS

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TABLE 1

	14-1	14-2	14-3	14-4
Max. Beam Load (kips)	273	205.4	289	303.5
% of V_p (at critical section)	94	71	100	105
% of V_p (at centerline of col.)	109	71	145	122
Max. Beam Deflection Δ_{max} (in)	2.07	1.58	3.02	3.22
$\frac{\Delta_{max}}{\Delta_p}$	3.09	3.16	3.02	4.81
Failure Mode	Fracture	Fracture	Fracture	Large Deformations

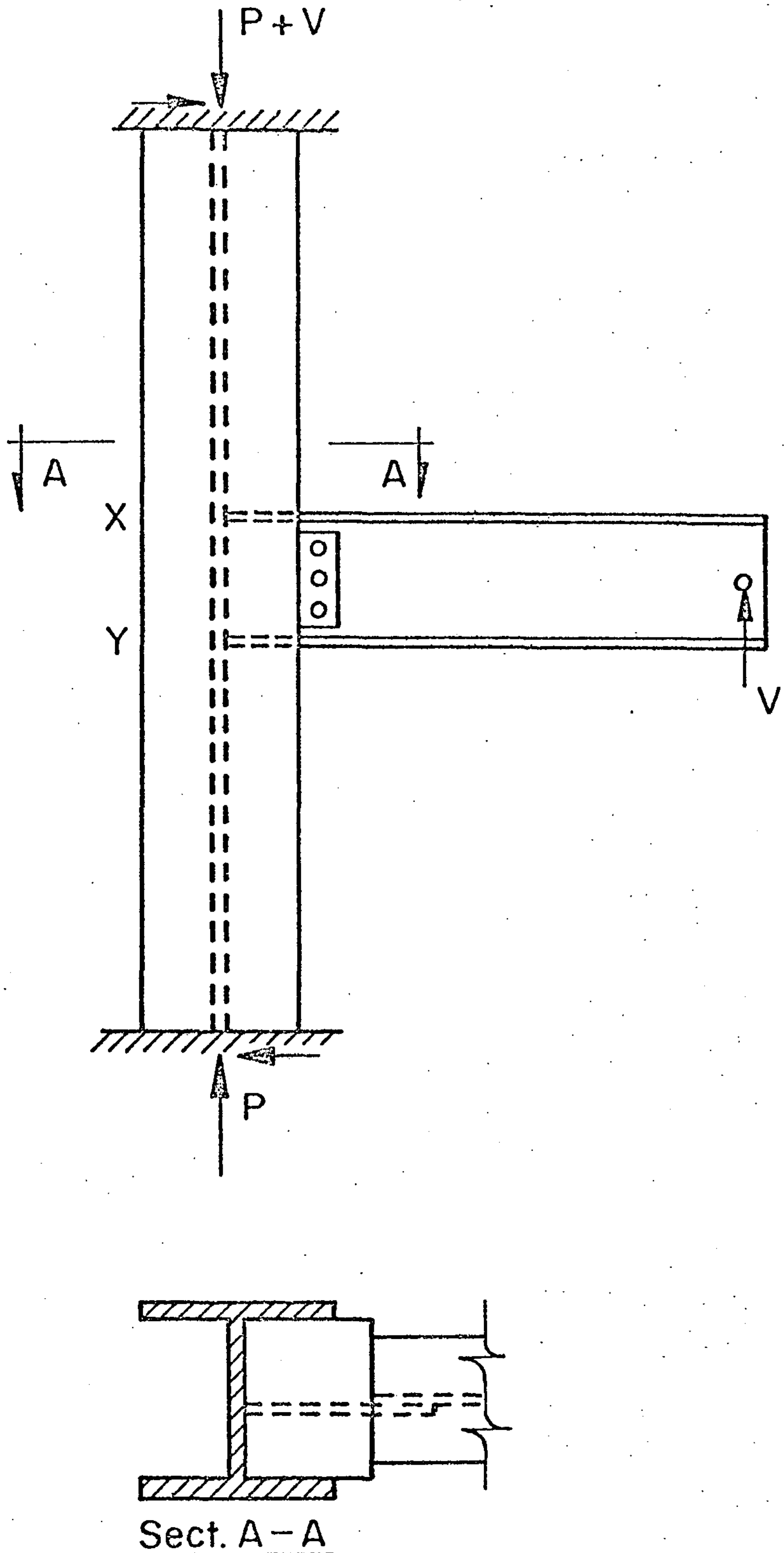

Fig. 1 Web Connection Assemblage

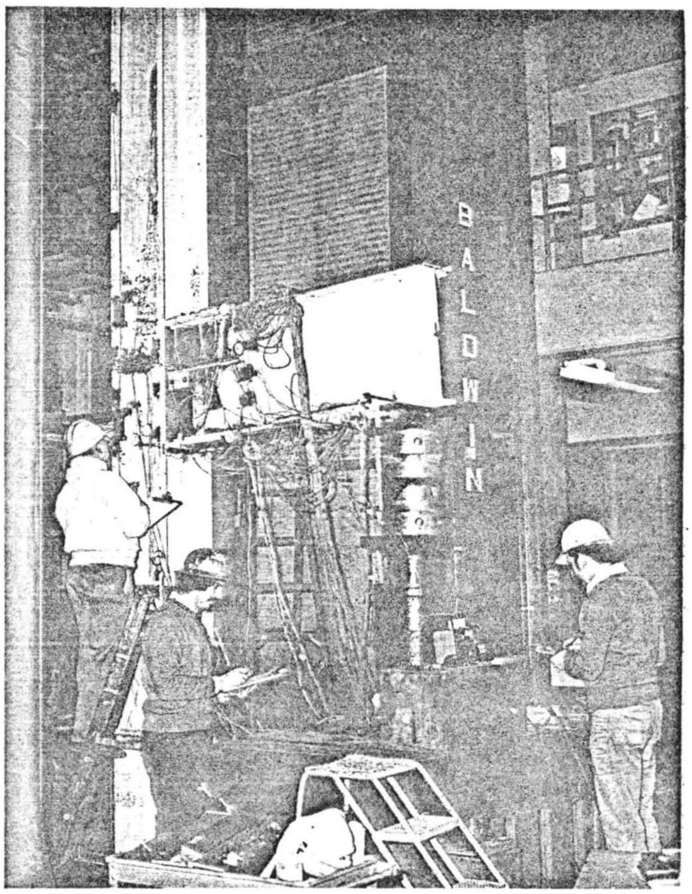

Fig. 2 Test Setup

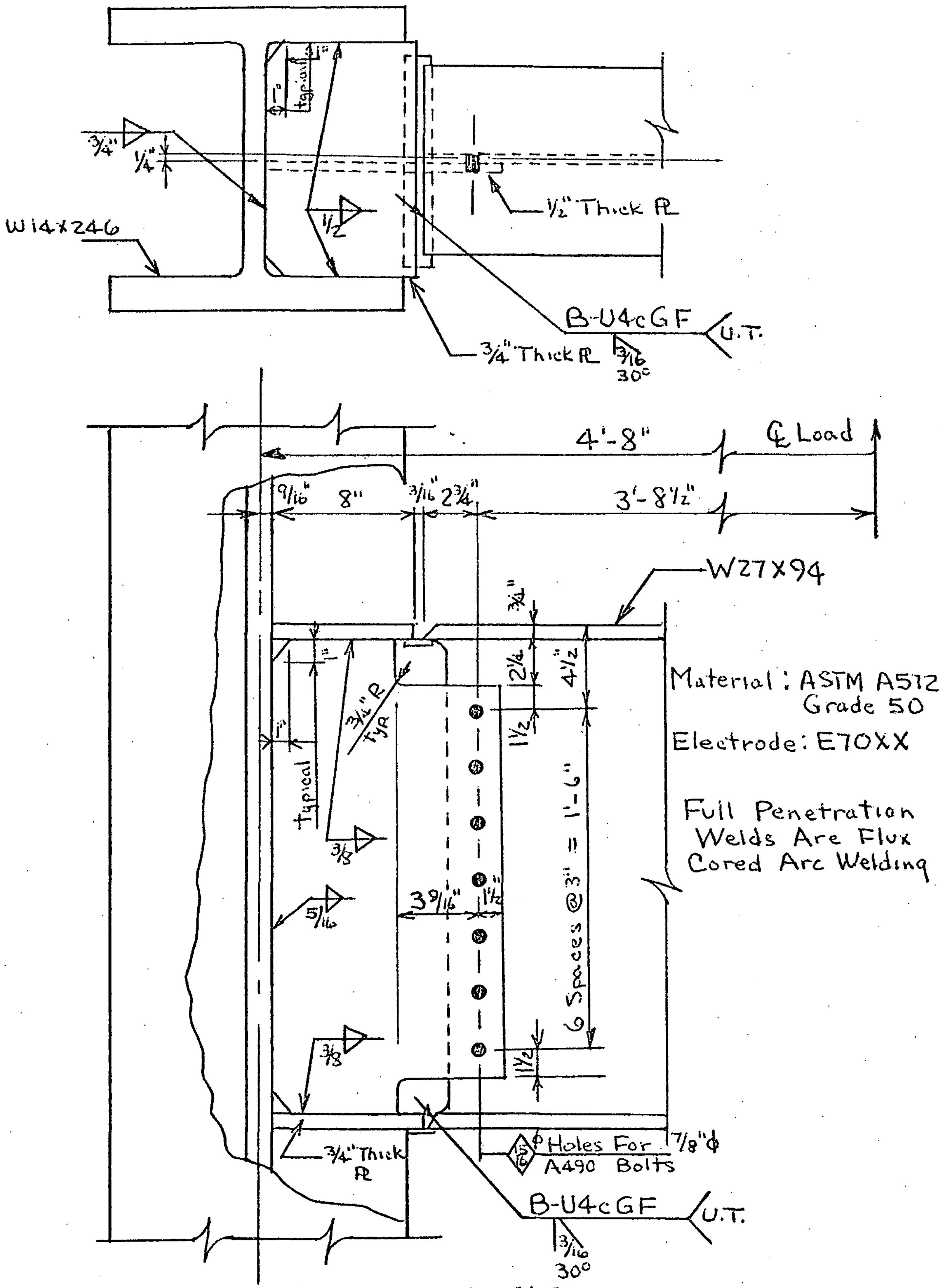

Fig. 3 Connection 14-1

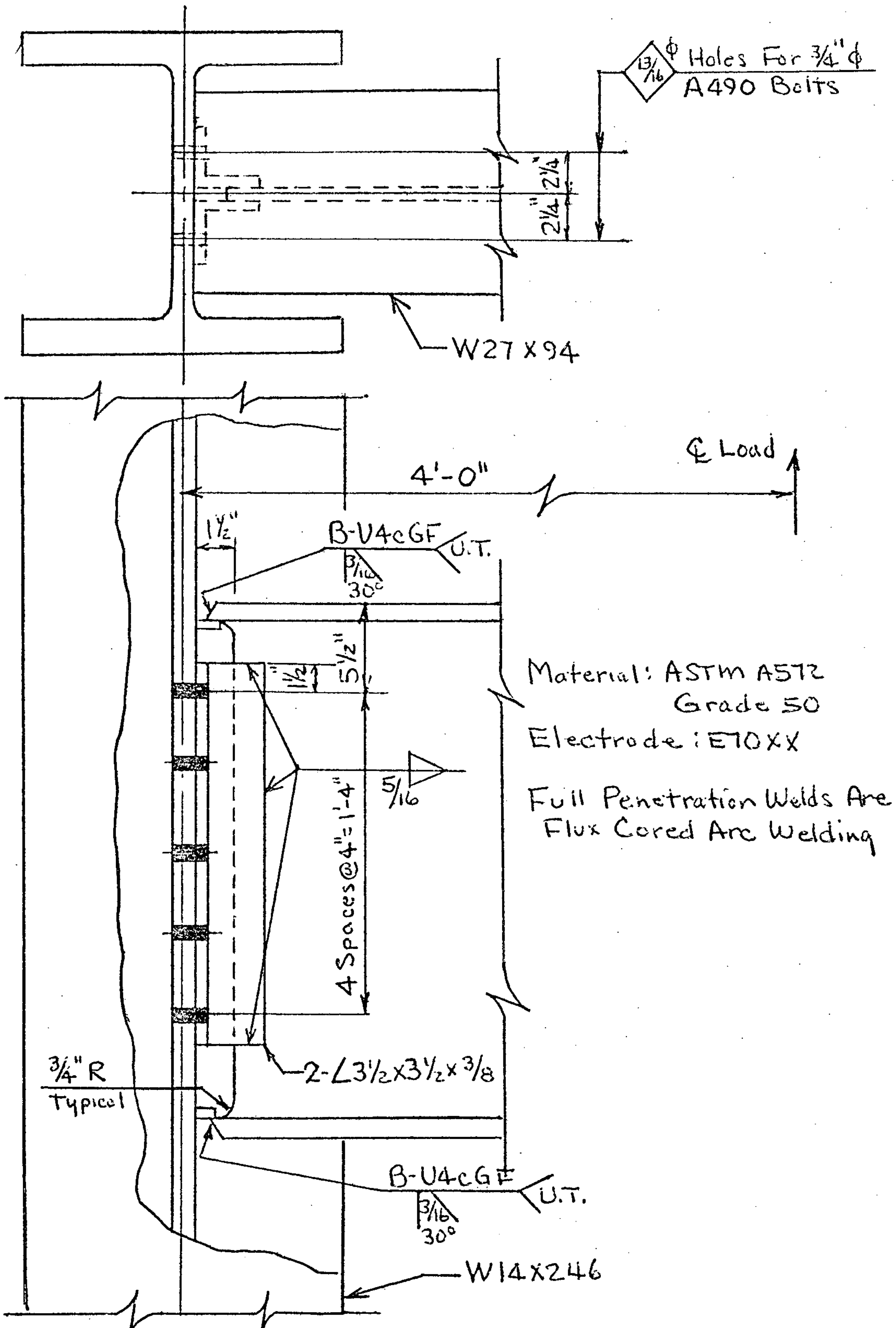

Fig. 4 Connection 14-2

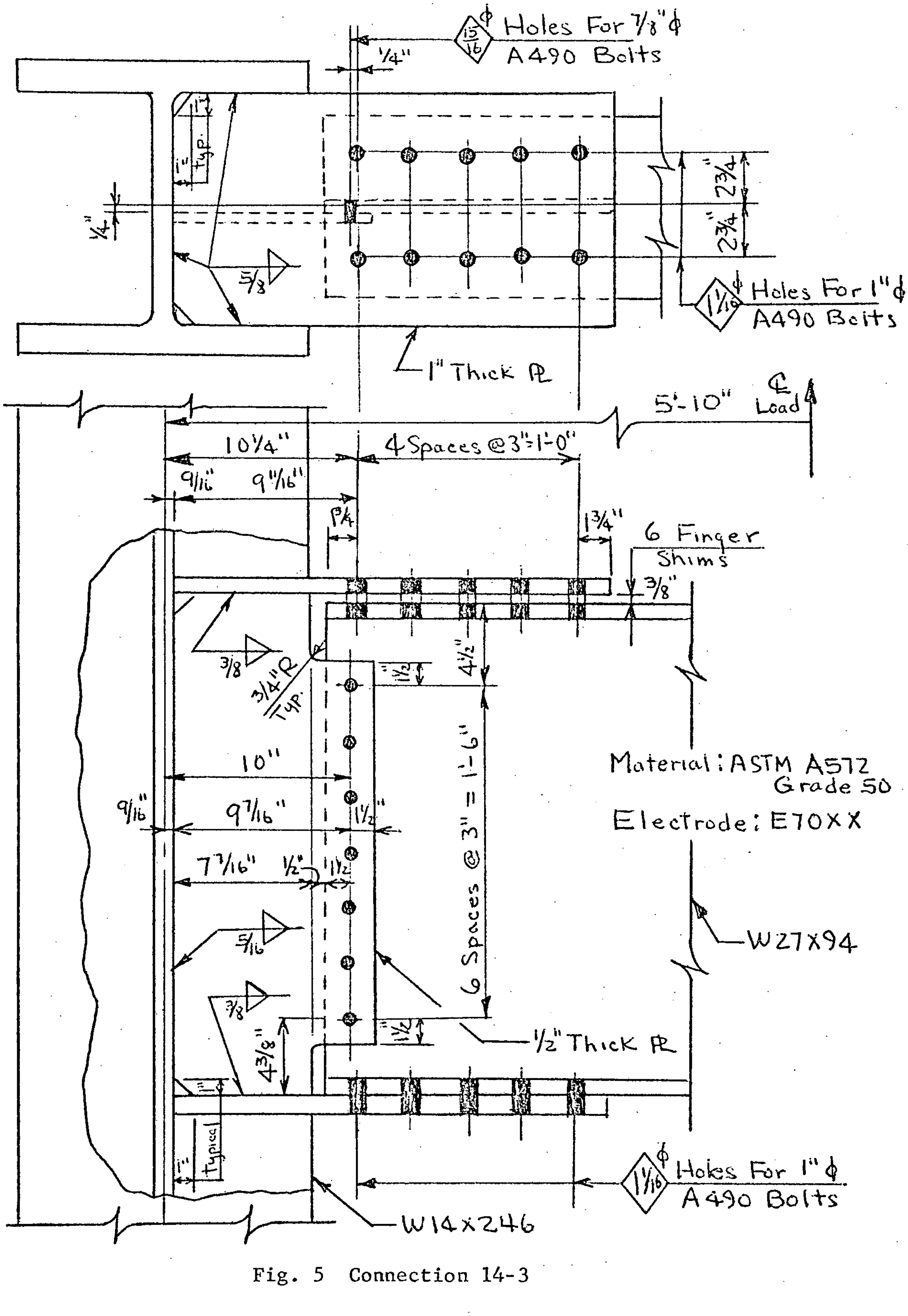

Fig. 5 Connection 14-3

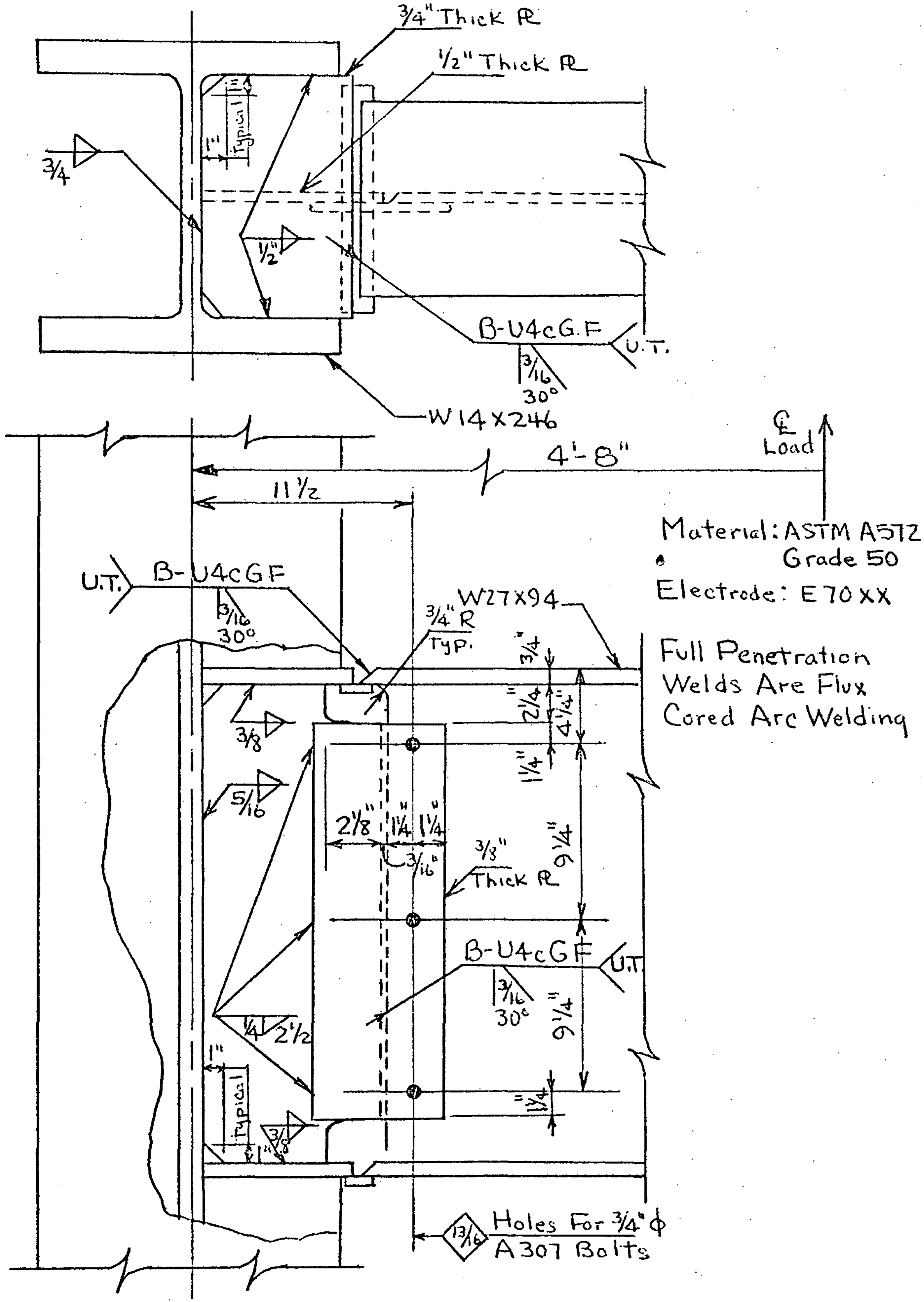

Fig. 6 Connection 14-4

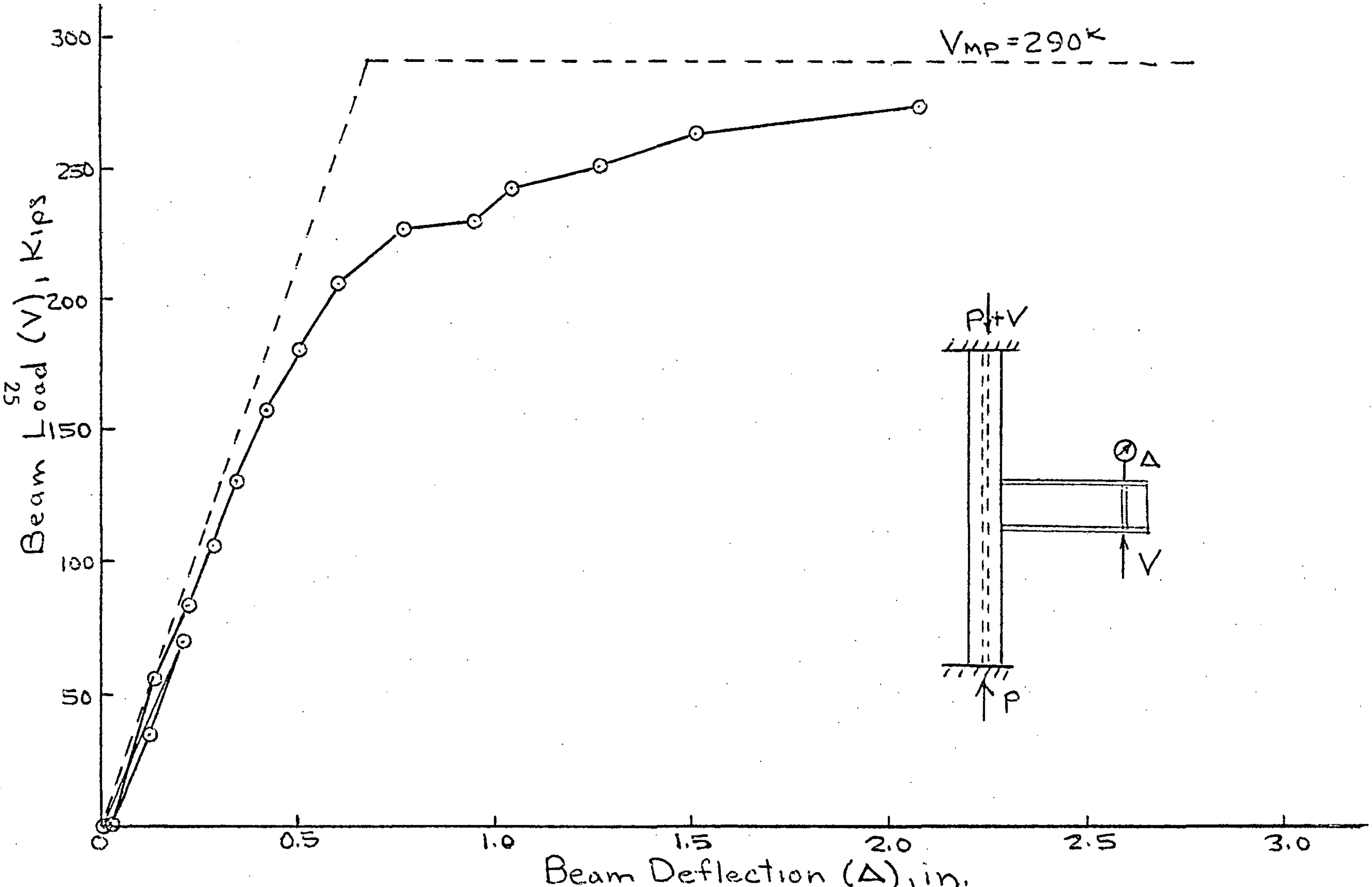

Fig. 7 Connection 14-1 Load-Deflection Curve

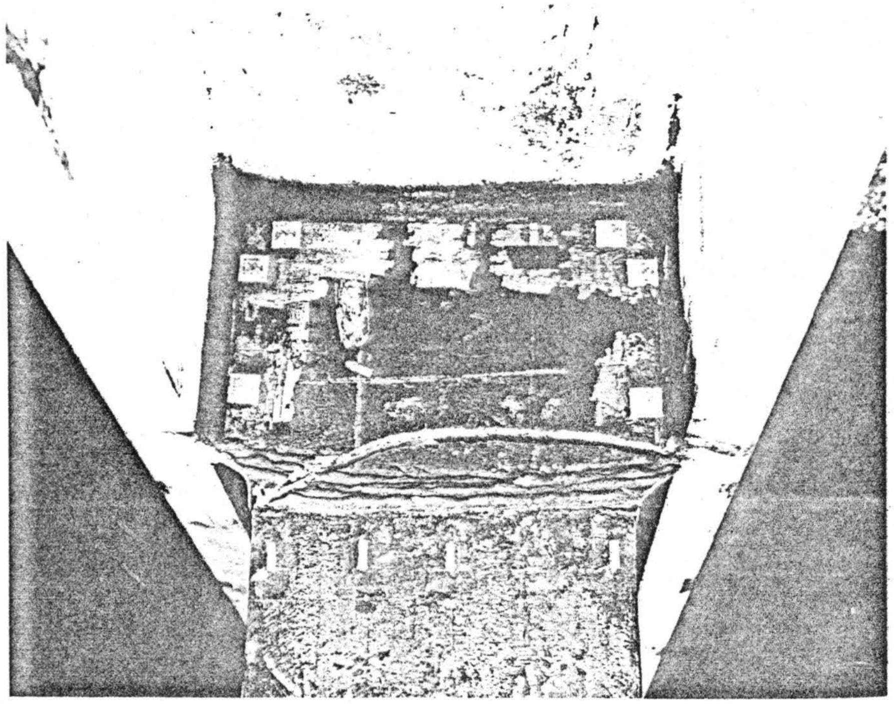

Fig. 8 Fracture of Connection 14-1

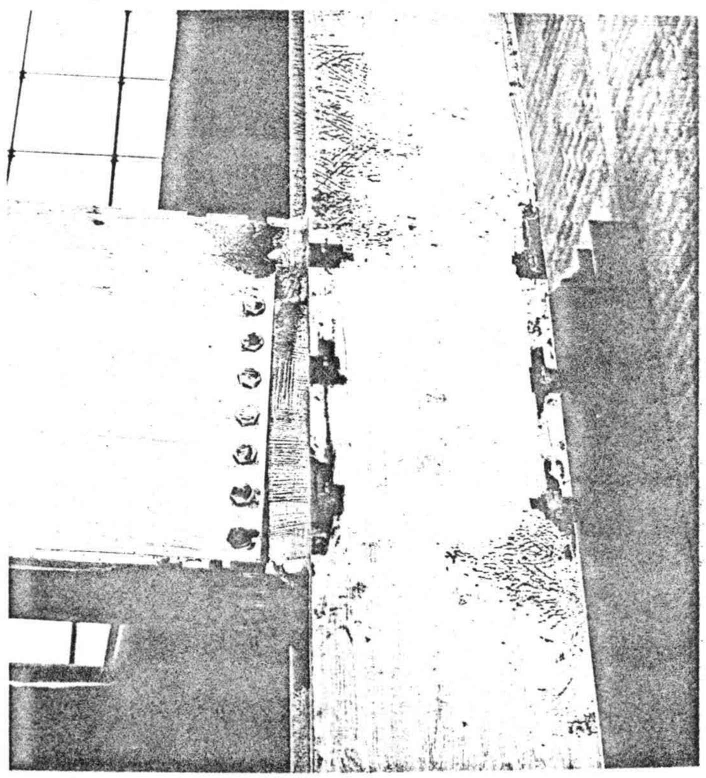

Fig. 9 Connection 14-1 after Testing

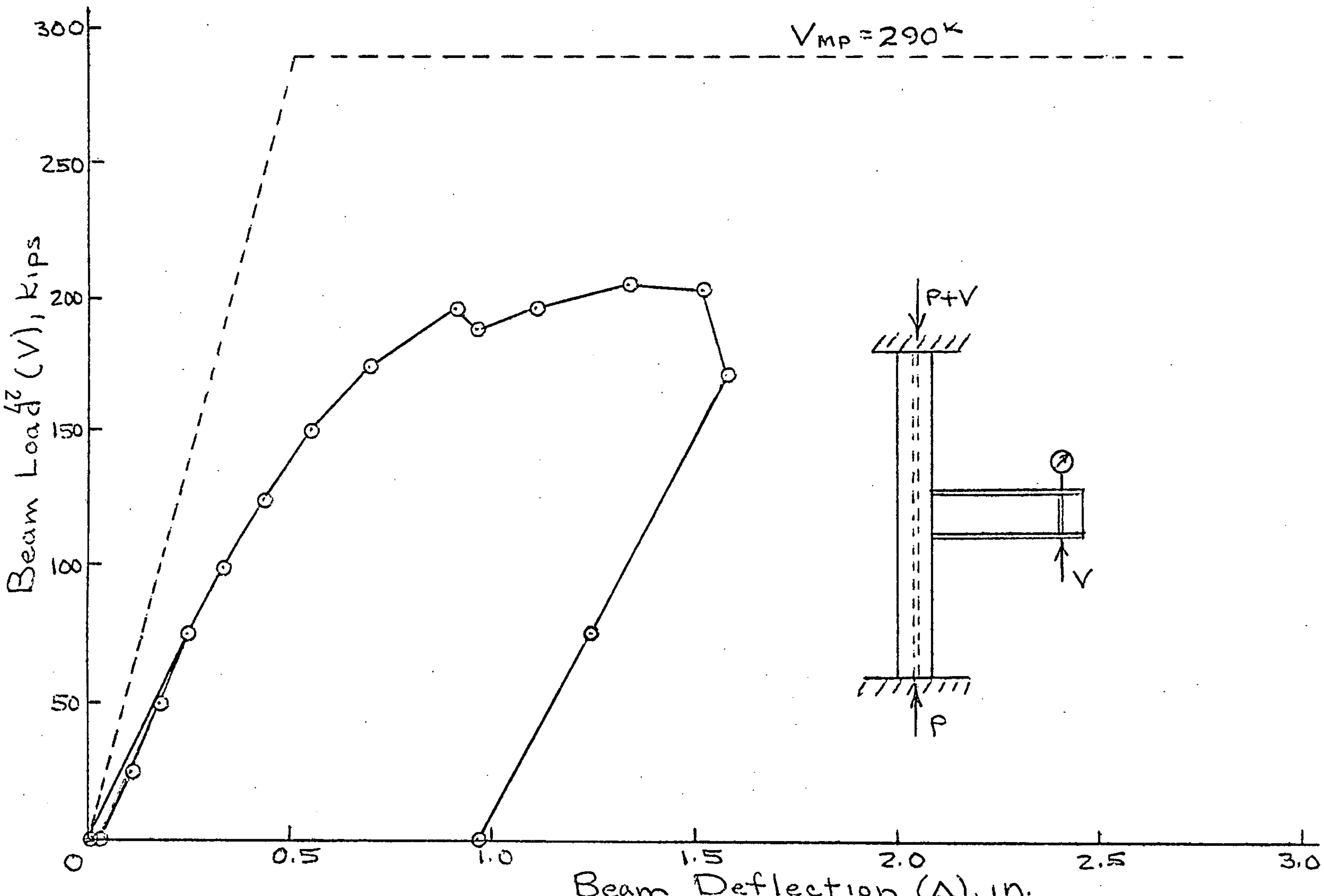

Fig. 10 Connection 14-2 Load-Deflection Curve

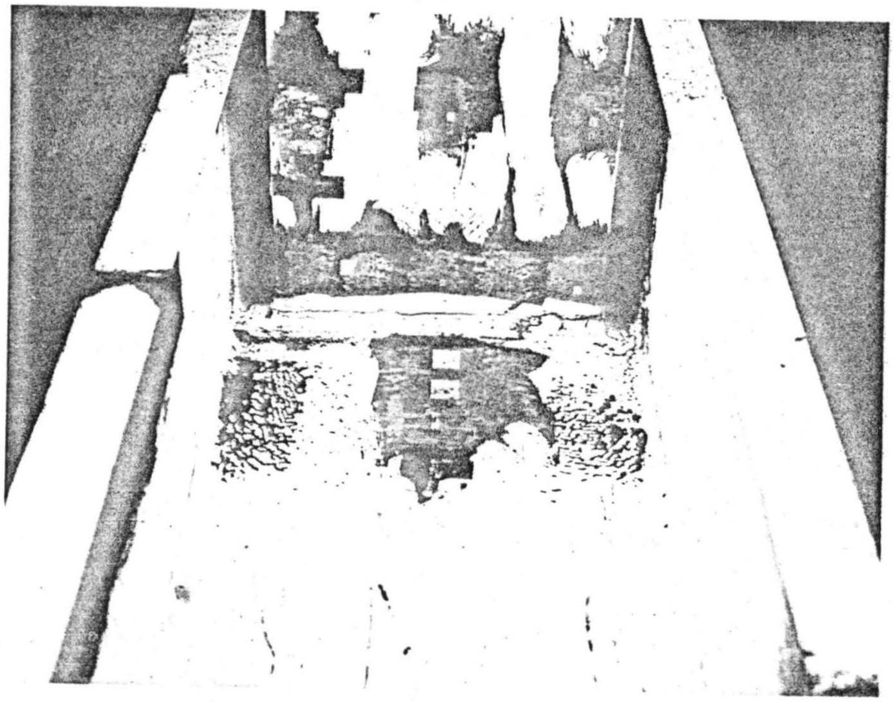

Fig. 11 Tearing of Connection 14-2 at Tension Flange-Column Web Junction

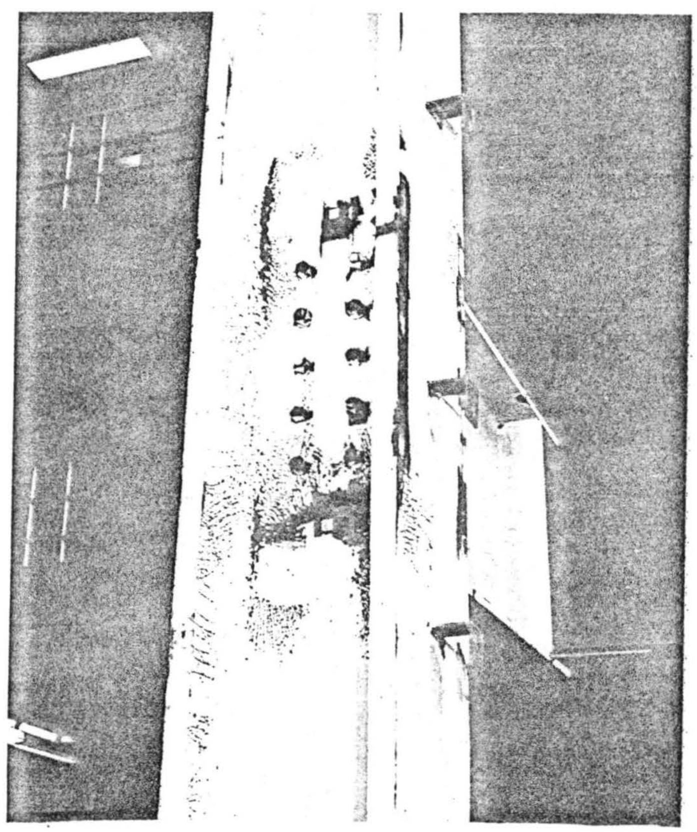

Fig. 12 Connection 14-2 Column Web Yielding

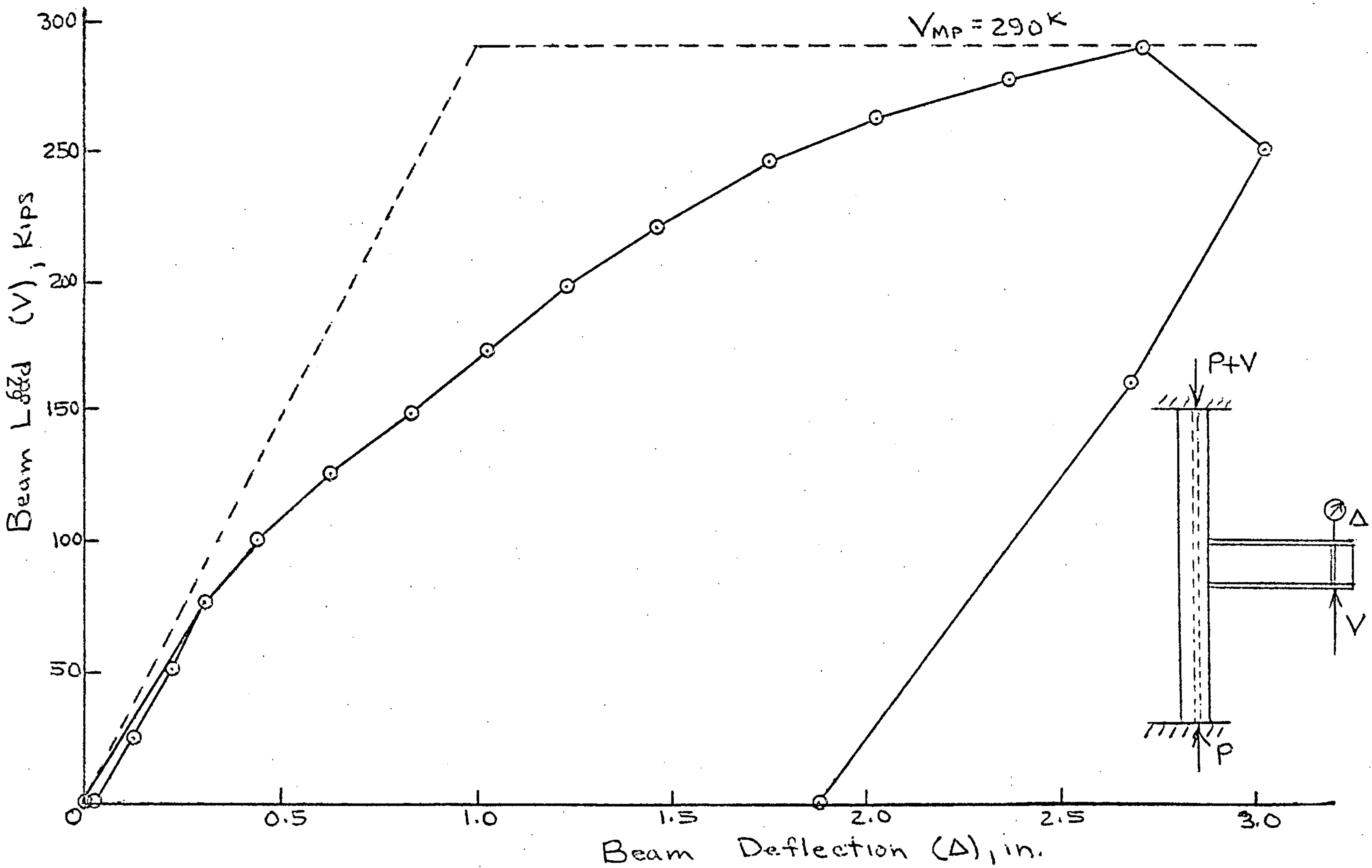

Fig. 13 Connection 14-3 Load-Deflection Curve

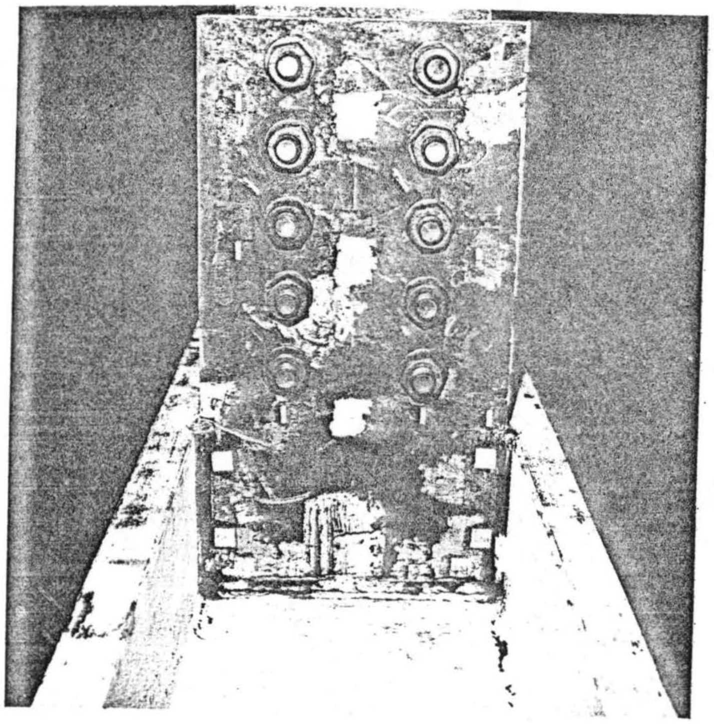

Fig. 14 Tearing of Connection 14-3 Flange Plate

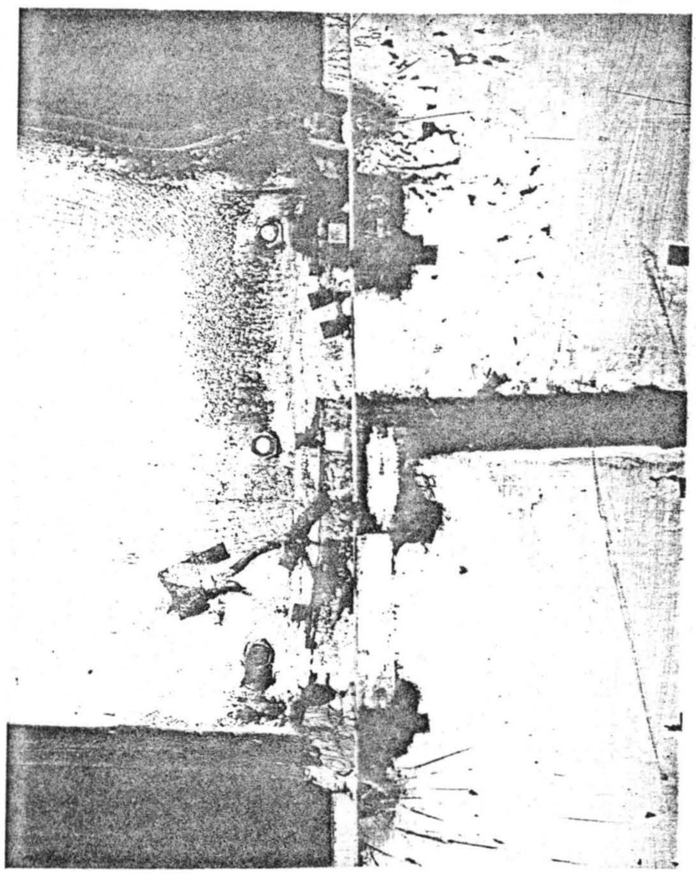

Fig. 15 Connection 14-4 after Testing

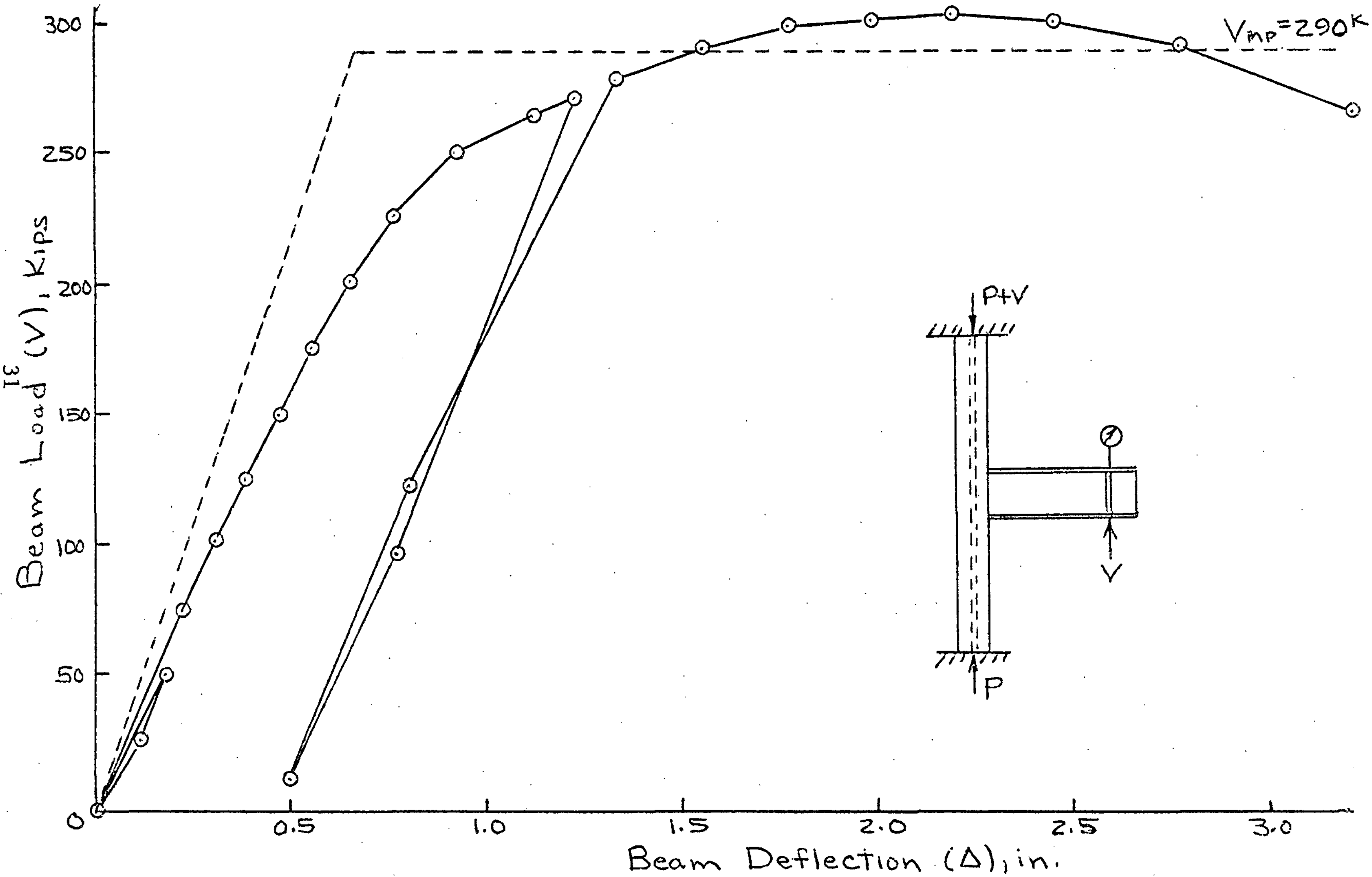

Fig. 16 Connection 14-4 Load-Deflection Curve