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Beam-to-Column Connections

HIGHLIGHTS OF LEHIGH CONNECTION TESTS--
WEB CONNECTION TESTS 14-2 AND 14-4

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Highlights of Connection Tests 14-2 and 14-4

The testing of two beam-to-column web connections was recently completed. The specimens tested were 14-2, a web connection where the beam flanges were welded directly to the column web and where the beam web was bolted to the column web and, 14-4, a fully welded connection where the beam flanges were attached to the column by means of flange connection plates which were welded to both the column web and flanges. This completes the testing of the four web connection assemblages planned in the experimental portion of the study of beam-to-column web connection behavior.

Test 14-2 failed when, first, the column web fractured due to the load on the beam tension flange and secondly, when a fracture occurred in the transverse direction on the beam tension flange adjacent to the butt weld attaching the beam flange to the column web. This occurred at a beam load which was approximately 70% of the load required to cause M_p in the beam. Test 14-4 performed well, reaching the M_p load and exhibiting significant ductility.

Prior to further discussion of these tests, a brief description of the testing procedure will be given. After being 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, which is at a different location in each test. This value of 290 kips represents a value of V/V_p of 0.80 where V_p is defined as the beam load required to cause shear yielding of the beam web. Both P and V values calculated above are based on using nominal yield stress values for the column and beam material, respectively. The material used for both beams and columns of the tests was A572 Grade 50.

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 plastic moment of the beam had been attained 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 exceeceeded 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 in the column below that level relative to the column above. Shown in Fig. 1 is a drawing of the test assemblage and loading scheme.

Connection 14-2

Connection 14-2 shown in Fig. 2 was a flange-welded, web-bolted connection. The beam flanges were welded directly to the column web by means of a full penetration groove weld. The beam web was connected to the column web by means of two structural angles which were welded to the beam web and then bolted to the column web by eight 3/4 in. dia. A490 bolts in 13/16" dia. holes. There was no attachment of the beam to the column flanges.

The plot of beam load V vs. beam deflection Δ is given in Fig. 4. This curve has a definite linear elastic V-Δ slope up to a load of approximately 100 kips. The effect of yielding of the assemblage is indicated by the nonlinear behavior at higher loads. The nonlinear behavior was primarily due to bending of the column web under the action of the beam flange forces.

The maximum load on this specimen was 205.4 kips which is 71 percent of the beam load needed to cause M_p 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 at the north side of the beam tension flange where the beam was welded to the column web. This is shown in Fig. 3. 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 the portion of the weld still connecting the beam flange to the column web fractured as shown in Fig. 3. Since the fracture did not proceed across the entire section, the load did not drop off completely. However, no further loading was attempted, and the specimen was then unloaded in two increments. The maximum beam deflection was 1.58 inches.

The elastic theoretical slope shown in the graph in Fig. 4 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 the axial load, or the effect of small end rotations at the top of the column. The theoretical horizontal line is the beam shear required to cause M_p in the beam based on nominal steel yield strength. This value was calculated for a critical beam section located at the column centerline.

Connection 14-4

Connection 14-4 shown in Fig. 5 was a fully-welded connection. The beam flanges were connected to the column by means of flange connection plates. These plates, equal in thickness to the beam flange, were connected to the column flanges and web by fillet welds and to the beam flange by full penetration butt welds. The web of the beam was attached to a web connection plate by a vertical butt weld. Prior to welding, the beam web was attached to the web connection plate by a vertical plate fillet welded to the web connection plate and bolted to the beam web to simulate a temporary erection scheme used in the field. This vertical plate then served as the back-up strip for the vertical butt weld.

The plot of beam load V vs. beam deflection Δ is given in Fig. 7. This connection had a definite linear elastic slope up to a beam load of approximately 150 kips at which time the stiffness started to reduce due to local yielding in the assemblage.

The maximum load on this specimen was 303.5 kips which is 105 percent of the beam load needed to cause M_p at the critical section, which in this case is taken to be at a point which is eight inches from the column centerline. 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 beam deflection at the end of testing was 3.64 inches. At the conclusion of the test, the beam compression flange where it was connected to the flange connection plate had buckled vertically out of plane and the web connection plate adjacent to the compression beam flange connection plate had buckled laterally. These were the reasons why the load started to fall off from its peak value. During the test at a beam load of 290 kips on the ascending portion of the load-deflection curve, vertical cracks were noted in the area of the groove weld connecting the tension flange to the flange connection plate in the region where the flange widens to a width equal to the distance between the inside faces of the column flanges. This area is shown in Fig. 6. These cracks when first noticed were of a length equal to about 1/4 of the plate thickness and at the conclusion of testing they were 3/4 of the length of the plate thickness and propagated laterally for a small distance along the back-up bar adjacent to the beam flange groove weld. The unloading and reloading portion of the load deflection curve was necessitated by problems with the jack applying the beam load.

The elastic theoretical slope shown in the graph in Fig. 7 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 the axial load, or the effect of small end rotations at the top of the column. The theoretical horizontal line is the beam shear required to cause M_p in the beam based on nominal steel yield strength. This value is calculated for a critical beam section located at the column flange tips.

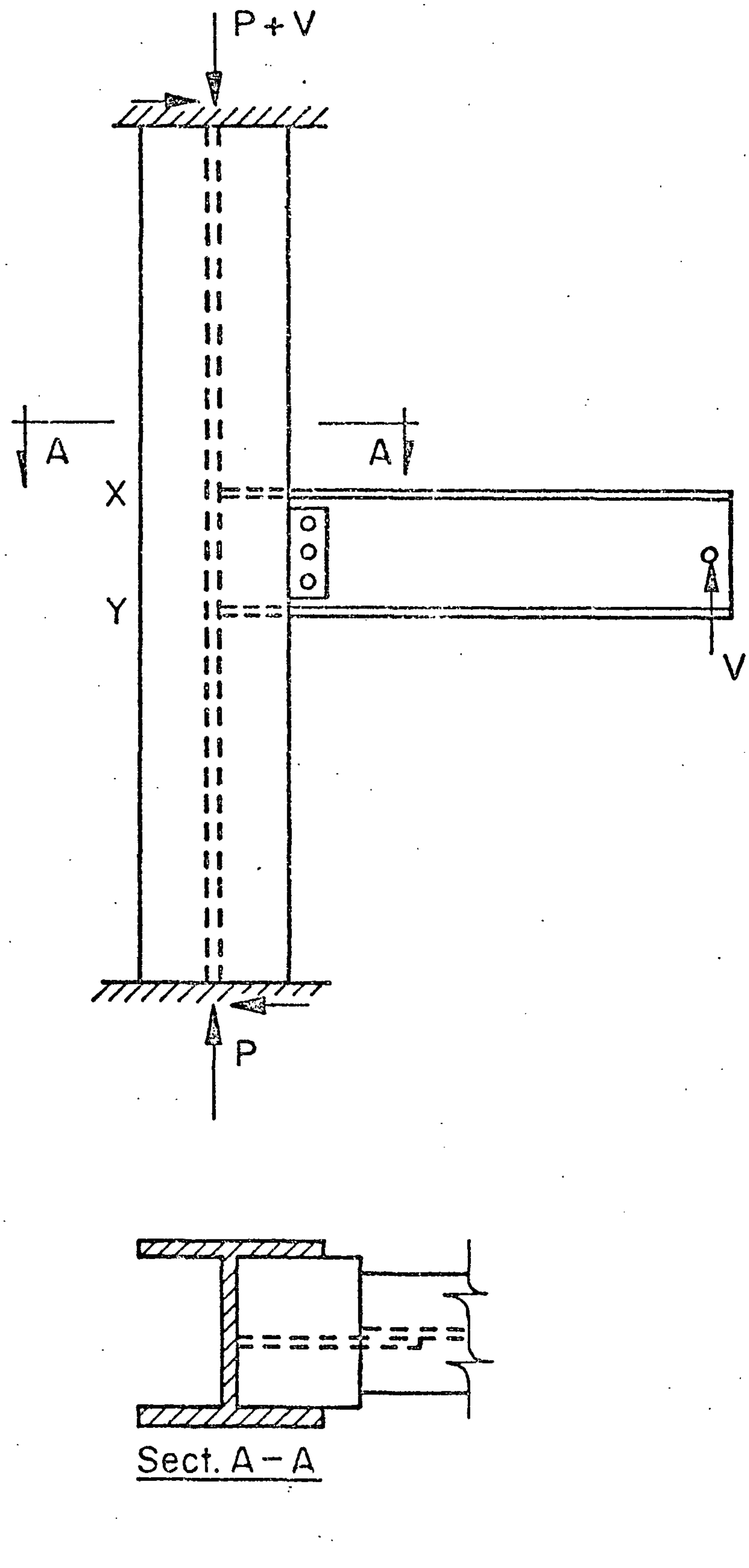


Fig. 1 Connection Assemblage and Loading Scheme

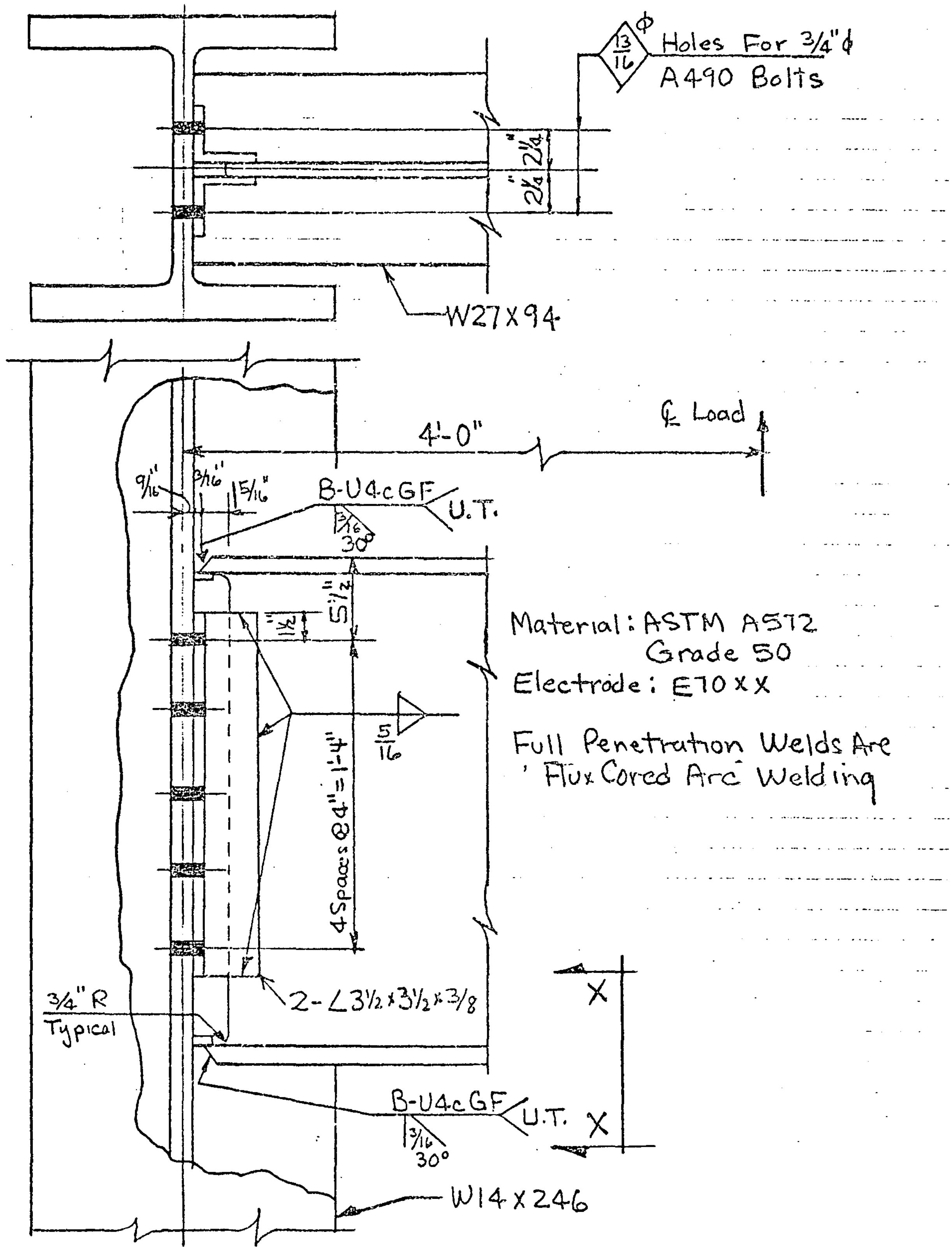
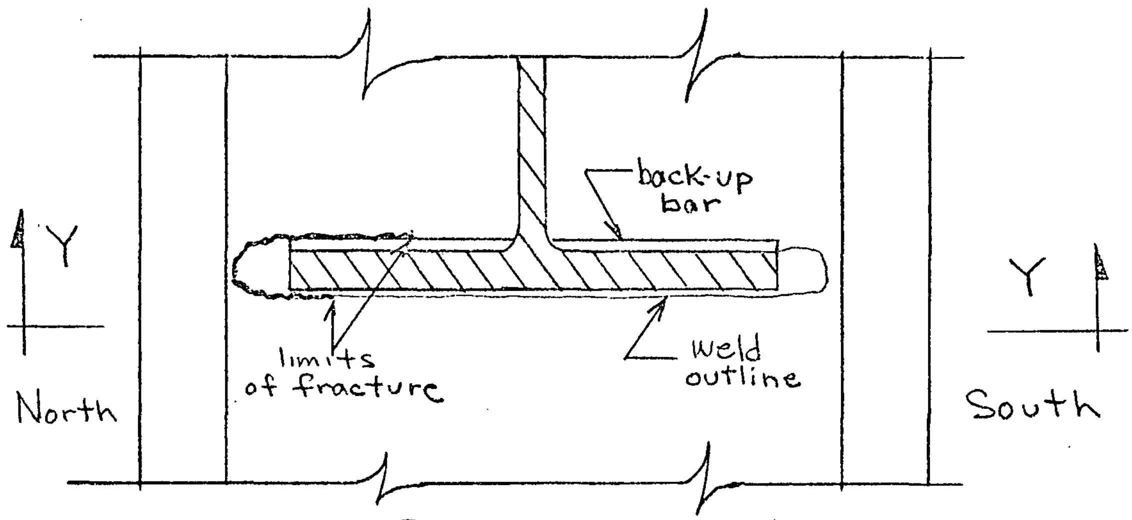
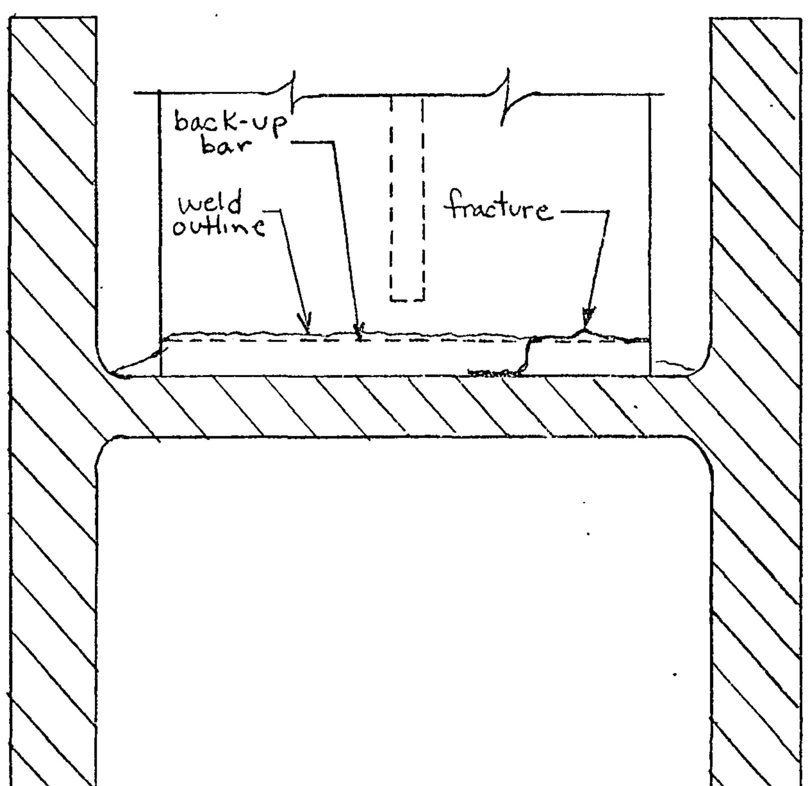


Fig. 2 Connection 14-2



Section X-X

Scale: 1/4" = 1"



Section Y-Y

Scale: 1/4" = 1"

Fig 3 View Showing Failure of Connection 14-2

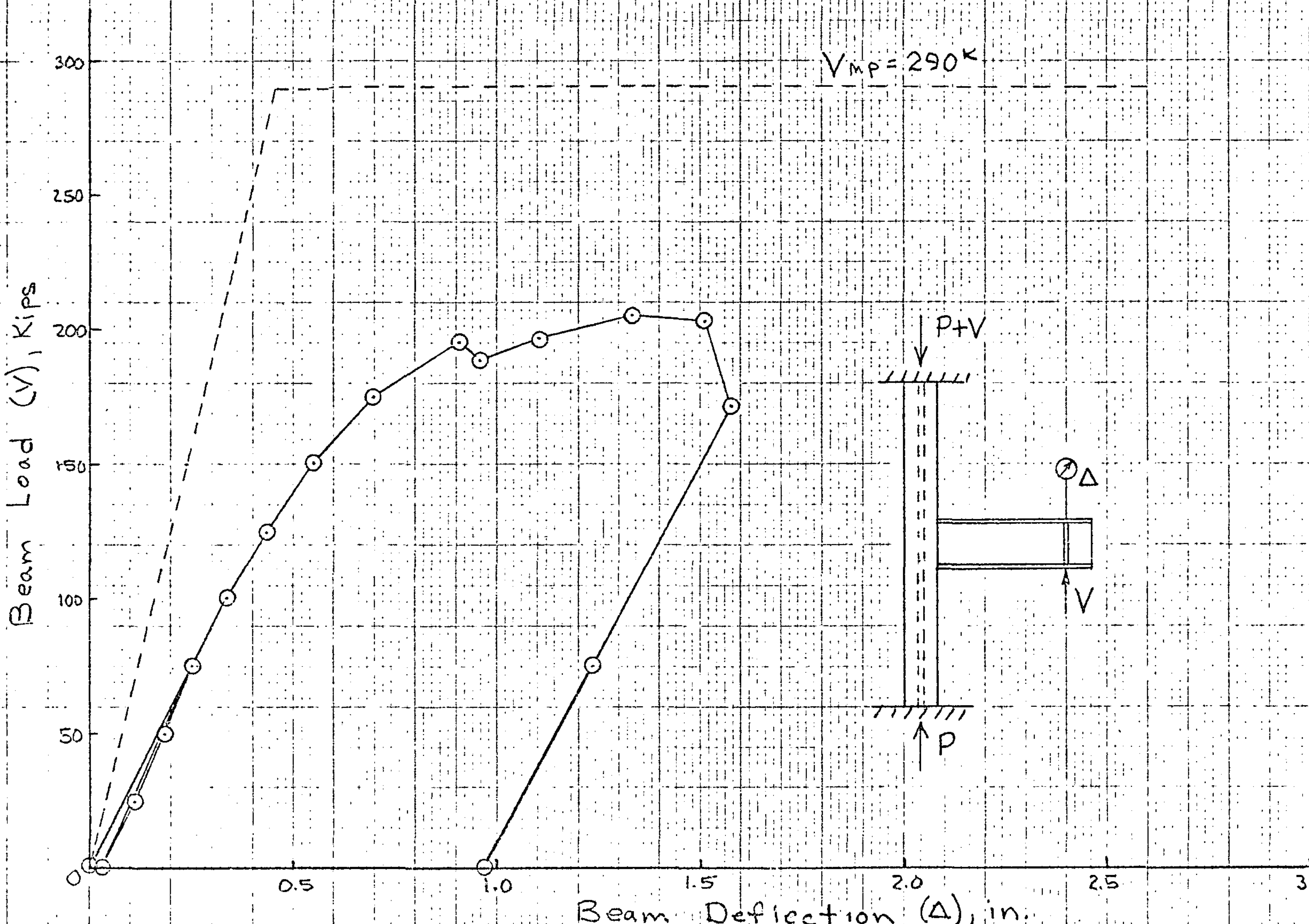


Fig 4 Beam Load Vs. Beam Deflection For Test 14-2

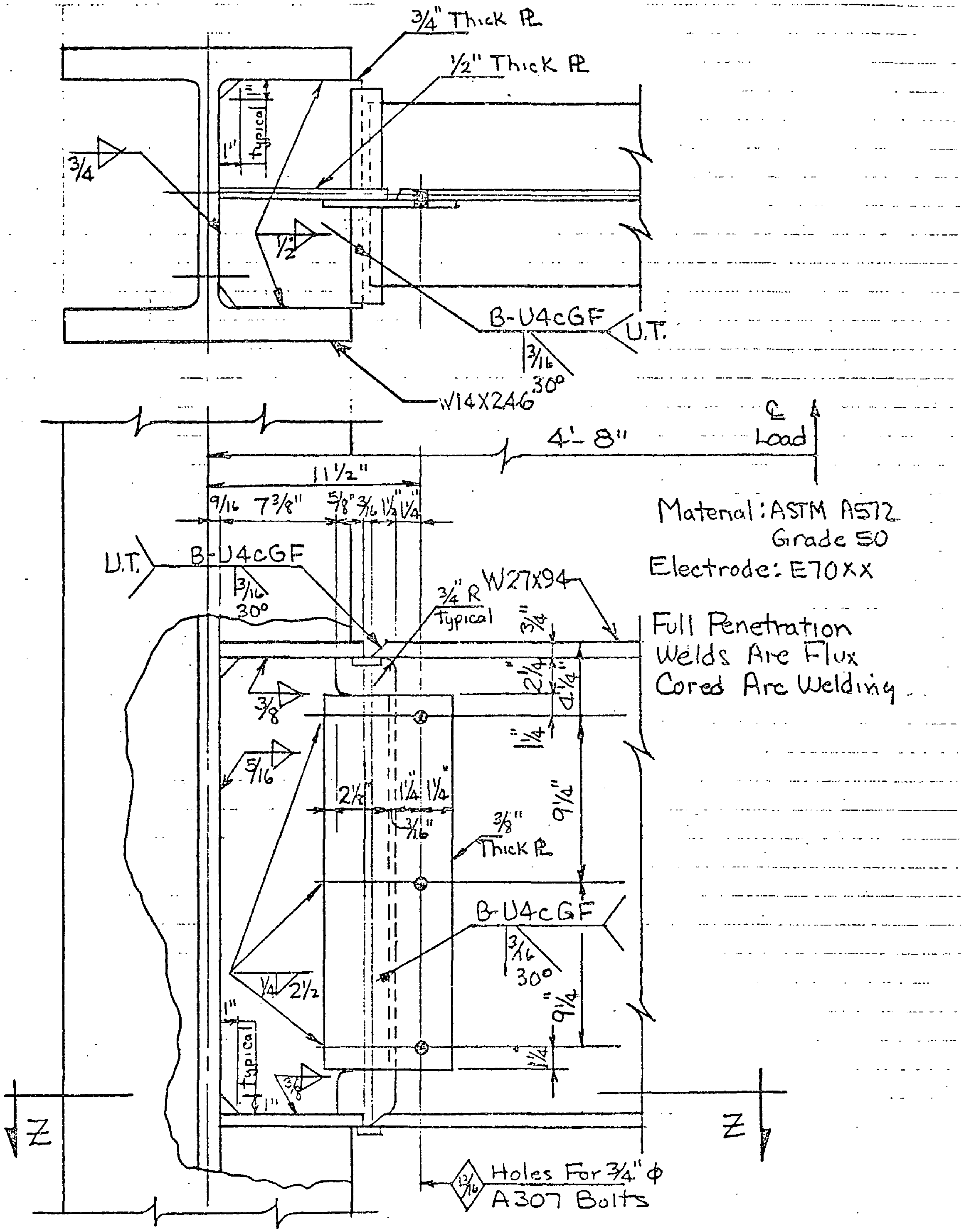


Fig. 5 Connection 14-4

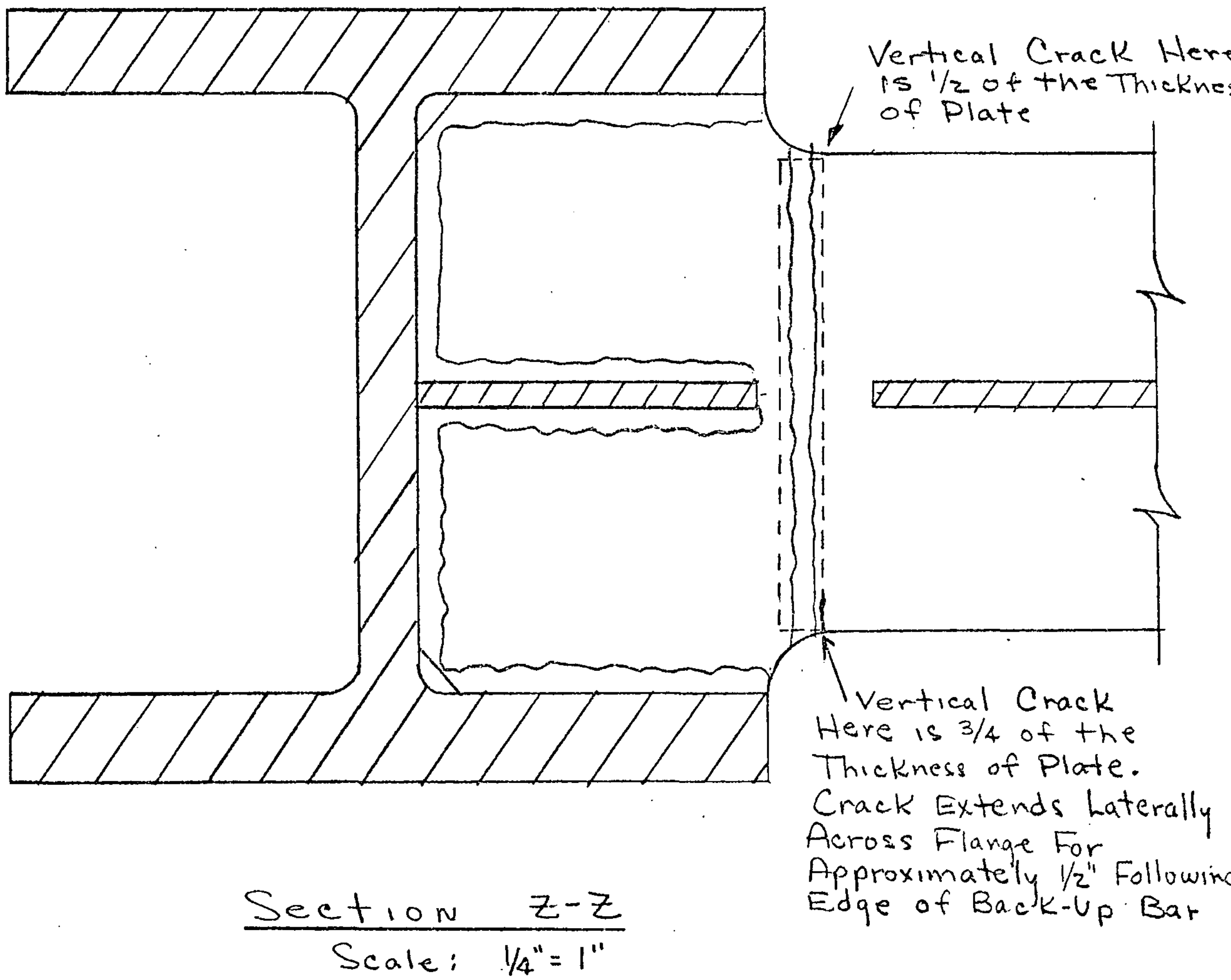


Fig 6 View Showing Cracks In Connection 14-4

Fig 7 Beam Load Vs Beam Deflection For Test 14-4