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George C. Driscoll

Beam to Column Connections

FRACTURES OF BEAM-TO-COLUMN WEB MOMENT CONNECTIONS

by

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FRACTURES OF BEAM-TO-COLUMN WEB MOMENT CONNECTIONS

KEY WORDS: Bolts, Buildings, Connections, Fracture, Moment-Resisting, Steel, Structural Engineering, Welding

Supplemental tests and analyses were made to determine the cause of fractures in two beam-to-column web moment connections. Fractures had occurred in tension flange connection plates during static monotonic bending tests of subassemblages containing the connections.

Metallurgical tests made included photomicrographs, chemical analyses, hardness tests, and Charpy V-notch impact tests. Tests showed that the connection plate material met the chemical analysis and hardness ranges typical of the material specified. The rolling direction of the connection was transverse to the direction of applied stress. Both tension flange connection plates failed in a predominantly brittle manner, although significant ductility was present on both fracture surfaces as indicated by shear lips or crack arrest markings. The fractures initiated at weld details, although flaws present in the welds were within normal standards.

It was concluded that the fractures occurred due to very large strains concentrated at design details. It was also concluded that it is possible that other assemblies with similar design details would fail in a substantially brittle manner if strained to a large amount.

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INTRODUCTION

In a structural test program on beam-to-column web connections, two unexpected fractures occurred in connection plates. Each connection plate joined the tension flange of a wide flange beam to the flanges and web of a wide flange column loaded so that it was bent about its weak axis. The beams were loaded to cause plastic moment at the tip of column flanges simultaneously with about 80 percent of full plastic shear.

Both connections reached a load value higher than that required to reach beam M_p based on calculations using the centerline of columns and beams in determining spans of members. Both connections also resulted in beam deflections exceeding 3 times the theoretical elastic deflection required to reach a plastic moment.

However, the fact that fractures occurred was disturbing to the investigators and the members of their advisory committee, the Task Group on Beam-to-Column Connections of the Structural Steel Committee of Welding Research Council. A limited additional study was authorized to search for causes of fractures of the connections. Metallurgical and chemical analyses of the fractured regions were proposed as well as hardness tests and Charpy V-notch toughness tests. This report describes the additional study. Reference will be made to three reports on these specimens: (1) Highlights report presented at the 1977 task committee meeting (405.6), (2) Preprint of paper at 1978 Pittsburgh ASCE meeting (405.9), (3) Report of the metallurgists (APPENDIX).

DESCRIPTION OF SPECIMENS

Connection 14-1

A sketch of the fabrication drawing of Connection 14-1 is given in Fig. 1. This connection was planned to provide a field bolted connection of the beam web to a shear plate welded to the column web. Field welded groove welds were planned to connect the beam flanges to connection plates welded to the column web and flanges and to the shear plates.

The load-deflection curve of this specimen is given in Fig. 2. Photographs after test are given in Fig. 8 and 9 of the preprint and pertinent statistics are given in Table 1 of the preprint.

Figure 3 shows the location and extent of the crack which occurred during the connection test. Also shown is a smooth radius gratuitously formed by the fabricator in the connection plate between the point where the column flange ends and the beam flange begins. This detail may be compared with the squared off connection plate backing bar and beam flange of Fig. 1.

Connection 14-3

Fabrication details of connection 14-3 are given in Fig. 4. This connection was planned for field bolting of both flanges and web of the beam to flange plates and web plates shop welded to the column.

The load-deflection curve of this specimen is given in Fig. 5. The initial slope of the experimental curve shows fair agreement with the slope of an idealized theoretical curve considering linear effects of bending, shear, and axial deformation.

The path of the failure crack is plotted in Fig. 6. The figure also shows the location of cosmetic weld passes provided along the thickness of the connection plate at the tips of the column flanges.

Since numerous previous series of beam-to-column and corner connections had been tested well into the inelastic range without fractures, a small investigation was conducted to determine if there were flaws in the materials or welds which could have caused the fractures.

SUPPLEMENTAL STUDIES

Several metallurgical and physical tests were performed with two main purposes:

(1) To determine if there were material or fabrication flaws which could have caused abnormal behavior.

(2) To determine the identity of anonymous steel taken from fabricators floor stock and provided as the connection plates of the two connections.

The fractured surfaces were sprayed with a protective coating to prevent corrosion shortly after the fractures occurred.

The supplemental studies performed were as follows:

(1) The fracture surfaces were examined visually and photographically to determine the site and mode of initiation of fracture and to detect any flaws.

(2) Chemical analyses were performed on the connection plate material to determine its type.

(3) Etching procedures were used to determine the extent of the heat-affected zone of the transverse full-penetration butt weld joining the beam tension flange of connection 14-1 to the flange connection plate which fractured. Etching was also used to determine the rolling direction of the steel used in connection plates.

(4) Rockwell hardness tests were run on the connection plates.

(5) Charpy V-notch toughness specimens were machined from the connection plates and tested. A transition temperature curve was prepared for each of the two connection plates. The locations and orientation of the Charpy specimens is indicated in Fig. 3 and Fig. 6.

RESULTS OF METALLURGISTS STUDY

Metallurgical studies were performed under direction of Dr. John D. Wood in consultation with Dr. Alan Pense, both of the Department of Metallurgy and Materials Science, Lehigh University. Their report is included in the Appendix. A summary of the findings follows.

(1) Material Identification

The 3/4 inch material of the tension flange connection plate of connection 14-1 was found by chemical analysis to be within the requirements of ASTM A572-Grade 50. The hardness was HRB-90. The rolling direction was perpendicular to the direction of applied tensile stress.

The 1 inch material of the tension flange connection plate of Connection 14-3 was not within the limits of ASTM A572 Grade 50 but it met A588B according to the chemical analysis. The hardness was HRB-84. The rolling direction was perpendicular to the direction of applied tensile stress.

(2) Discussion of Fractures

The fracture of Connection 14-1 initiated at the edge of the butt weld joining the beam tension flange to the connection plate. The point of crack initiation is indicated by the point of the arrow labelled "crack" in Fig. 3. The metallurgist noted that this point would have a strain concentration due to the reduction in plate width at the end of the radius. He also noted that weld irregularities could have concentrated to the strain concentration. The fracture was predominantly brittle in nature although there is evidence of several crack arrests which indicates some ductility in the region. There is no evidence of defects on the fracture which could have contributed to the failure.

Metallographic examination of the weld indicated that defects (voids and fusion line cracks up to 1/10 inch in maximum dimension) were present. It was noted that these defects were not greater in size than what might be typically found in structural welds.

The metallurgist concluded that this connection failed in a brittle manner due to the large applied tensile strain which was concentrated at the design detail. The defects present in the weldment were not identified as the primary cause.

The fracture of Connection 14-3 initiated at the end of the welding joining the tension flange connection plate to the column flange. The fracture propagated across the tension plate normal to the applied tensile strain. The initiation site appears to be at a cosmetic welding pass between the edge of the column flange and the side of the connection plate. The point of crack initiation is indicated by the arrow labelled "crack" in Fig. 6. The fracture was predominantly brittle, but shear lips are present indicating some ductility in the connection plate. There is no evidence of any defects on the fracture surface. (Note: The "cosmetic welding pass" might also be called a seal weld.)

The metallurgist concluded that Connection 14-3 failed when a large amount of strain was applied and was concentrated at a design detail (the edge of the column flange and tension plate) which was made worse by the use of a cosmetic edge welding pass.

(3) Conclusion

The metallurgists overall conclusion was:

> "Since it is possible for acceptable steel to fracture in a predominantly brittle manner if strained significantly and strain concentrators are present, it is not surprising that these connector-beam assemblies failed as they did. It is very possible that other assemblies with similar design details would fail in a substantially brittle manner if strained to a large amount."

RESULTS OF CHARPY TESTS

Since there were no extra pieces of connection plate steel available, specimens for standard Charpy V-notch tests were taken from the actual plates which failed. Figures 3 and 6 show the locations of the Charpy specimens. Fifteen TL specimens were obtained from Connection 14-1 as indicated in Fig. 3. Three additional transverse LT specimens were obtained from the same specimen.

Only a small piece was available from Connection 14-3 because part of the material was mislaid in storage. Charpy specimens were cut from two layers of the connection plate steel. Eight TL specimens were obtained from one layer and four more TL specimens were obtained from the lower layer along with three transverse LT specimens. The location is sketched in Fig. 6.

The fifteen TL specimens of Connection 14-1 were tested as planned, 3 each at temperatures of -40°F, 0, +40, +70, and +100. Since 40°F appeared to be the temperature at which 15 ft-lb of energy was absorbed, the three LT specimens were tested at +40°F. The test results were evaluated using a Metallurgy Department computer program which averages, curve fits, and plots the results. The resulting plot is given in Fig. 7 and the numerical results are given in Table 1. Based on an impact energy criterion, the steel was able to absorb 15 ft-lb at a temperature of 27.0°F. This is better than the AASHTO fracture-toughness specifications for A572 steel to be used in Zone 2 with minimum service temperature from -1 to -30°F (15 ft-lb at 40 F).

The three LT specimens for Connection 14-1 were expected to be better and they were. They gave values of 36, 19, and 39 ft-lb at 40°F, for an average of 31 ft-lb (see Table 3).

Testing of the Charpy specimens from Connection 14-3 required a different series of temperatures. The first three TL specimens were tested at 0°F and gave results averaging 94 ft-lb. Therefore the next three were tested at

-40 F resulting in values averaging less than 15 ft-lb. Henceforth, three specimens were tested at -30°F and three at -20°F. The test results are plotted in Fig. 8 and the numerical results are given in Table 2. Based on the impact energy criterion, the steel was able to absorb 16 ft-lb at a temperature of -49°F. This is better than the AASHTO fracture toughness specifications for A588 steel to be used under the most severe Zone 3 requirements with minimum service temperature from -31 to -60°F (15 ft-lb at 10°F).

The three LT specimens for Connection 14-3 gave values of 14, 10, and 12 ft-lb at -30 F° (see Table 3). These values were lower than the values of the perpendicular specimens at the same temperature. These results are adequate compared to the A572 steel, although they place suspicion on the accuracy of determination of the rolling direction. The metallurgists had indicated that they were not absolutely sure of the rolling direction for this one specimen.

COMPARISON WITH RESULTS OF OTHER RESEARCH

There have been few previous tests of beam-to-column web connections and those which were conducted used much smaller structural members.

Three prior tests at Lehigh were conducted by Jensen et al. on four-way connections.(5) Those connections used 12WF65 and 12WF40 columns along with 16WF36 and 12WF27 beams. Because the connections were four-way, the column web always had stiffening opposite the beam tension flange. None of these tests experienced cracking.

Tests conducted by Popov included web connections with W8x20 beams framed to W8x48 columns.(4) These connections were one-sided connections with stiffener plates on the column web opposite the flange connection plates. The testing included cyclic reversed repeated loading applied to determine hysteresis loops in an earthquake study. The Popov connections experienced cracking similar to that in the current study. The investigators stated that the propensity for crack initiation in the web-connected specimens appears to be greater than in the flange-connected type.

In interpreting these results, further insight may be gained from references on fracture mechanics. It has been found that cracking may initiate in a ductile steel at points where a triaxial state of stress exists with high strains. Also it has been found that thick steel plates (over about one inch thick) are subject to a condition of plane strain which causes a triaxial state-of-stress and reduces the apparent ductility of the steel.[6]

The present series of beam-to-column web connection tests is the only one to use such heavy members. However, a previous corner connection test was made on a joint using W36x230 beam and column, both bent about their strong axis.[8] The joint was first subjected to a closing corner bending condition up to about 10 times the yield rotation and then later subjected to an opening corner bending condition. A complete fracture of the tension flange occurred after the plastic moment had been reached and deformations about 8 times the yield deformation had been reached. The configuration of intersecting flanges and stiffeners at the reentrant corner would be expected to result in a triaxial stress state. Thus this serves as one precedent to the brittle fracture of a highly strained member of ductile steel. The behavior was not present in similar joints using W30x108, W24x100, and W14x30 members, all having less thickness.

ANALYTICAL STUDY

Partial results are available from an analytical study made as part of a PhD dissertation by Rentschler.[8] The study was made using elastic range finite element calculations. Some of the results allow comparison with experimental observations, and some of the results are predictions of the effects of different possible variations of details in connections. The connections studied were essentially the same as 14-1 with members ranging from W14x455 column and W30x132 beam down to W12x40 column and W12x22 beam. Solutions were made with and without fitted stiffeners opposite the beam flange connection plates and with and without welding of the flange connection plates to the column web. Also the effect was studied of extending the beam flange connection plates a couple of inches out past the ends of the column flange tips before connecting the beam flanges.

Pertinent findings of the analytical studies are as follows:

(1) Beam tension flange stresses at the outer edges of beam flanges near the region of transition from beam flange width to connection plate width range from 2.6 to 5.5 times the nominal Mc/I stress.

(2) The most severe concentrations occur when the beam flange connection plate is not connected to the column web. They range from 3.0 to 5.5.

(3) If the flange connection plate is welded to the column web, the stress concentration factor at the beam flange tips is reduced somewhat so it ranges from 3.0 to 5.3.

(4) If a fully-welded stiffener is provided for the column web in line with the beam flange connection plate, the stress concentration at the beam flange edges is 2.6 to 4.2.

(5) Doubling the area of the stiffener changes the stress distribution only modestly. (The investigator did not report on other ways of increasing stiffness of stiffeners.)

(6) Extending the flange connection plate two inches past the tips of the column flanges causes about 10 percent reduction in the computed stress concentration factor when the beam flange is as wide as 0.8 of the connection plate width. There is negligible reduction when the beam flange is as narrow as 0.4 times the connection plate width.

(7) The largest contribution of extending the flange connection plate is reduction of the stress concentration in the connection plate at the plane of the column flange tips. With wide beam flanges as wide as 0.8 of the connection plate width, the stress concentration factor is about 0.8 of the value with no plate extension. With a narrow beam flange of about 0.5 of the connection plate width, the concentration factor is about 0.25 of its value with no plate extension.

The overall impression gained from the analytical study is that stress concentration factors of 3 or 4 can not be avoided at the edges of beam flanges or connection plates. This concentration can cause early inelastic behavior and possible exhaustion of ductility before redistribution averages out the stresses across the whole plate.

SUMMARY AND CONCLUSIONS

Supplemental tests and analyses were made to determine the cause of fractures in beam-to-column web moment connections 14-1 and 14-3.

It was concluded from the studies that the steel material and welds were of normal soundness and quality and that the fractures occurred due to large strains at details sustaining triaxial stress conditions under a combination of high bending and shear loading.

The findings leading to these conclusions are as follows:

(1) Chemical analyses of the tension flange connection plate materials showed that they were in the composition range of ASTM A572 and A588 for Connection 14-1 and 14-3 respectively, (Appendix).

(2) Rockwell hardness readings of B-90 and B-84 were consistent with the same steel grades, (Appendix).

(3) Charpy V-notch toughness tests showed that the connection plate material is able to absorb 15 ft-lb or more at 29°F and at -40°F respectively, (Table 1, Table 2).

(4) Metallographic examination of welds showed defects no larger than might be found in typical structural welds.

(5) Fracture initiated at the edge of the tension flange butt weld in Connection 14-1 and progressed through the connection plate. The fracture was predominantly brittle in nature with evidence of several crack arrests indicating some ductility in the region.

(6) There were no evidences of defects on the fracture surface of either connection.

(7) The fracture of Connection 14-3 initiated at the end of the weld joining the tension flange connection plate to the column flange. The fracture was predominantly brittle, but shear lips indicate some ductility in the connection plate. The fracture arrested when load was removed.

(8) Although the tension flange connection plate material was aligned so the major tensile stress was perpendicular to the rolling direction, it is believed that this had no effect on the fractures.

(9) References on fracture mechanics identify a limiting thickness of about one inch for change in behavior of steel plates from a plane stress state to a plane strain state. The plane strain state leads to triaxial stresses and an apparent reduction in ductility of the steel.

(10) A previous Lehigh investigation on four-way beam-to-column connections had some joint details similar to Connection 14-1. No cracking occurred. Members were much thinner than one inch, thus avoiding the plane strain condition.

(11) Web type connections tested under cyclic reversed repeated loading by Popov were found to be more susceptible to cracking than flange type connections. The members were small and probably not subject to plane strain.

(12) One previous Lehigh test which probably was affected by a plane strain condition was a tension test (opening corner) of a 36WF230 corner connection. This joint having a flange thickness over one inch fractured at room temperature after sustaining very large inelastic deformations in both compression and tension.

(13) Elastic finite element analyses performed to provide theoretical values of load-deflection behavior gave results of stress concentrations consistent in magnitude and location with those measured by strain gages. They support the metallurgical observation of high strains at the points of fracture initiation.

Because of the widespread use of connections of the types studied and because of their susceptibility to brittle fracture when highly overstressed, it is recommended that appropriate design information be included in design guides.

ACKNOWLEDGEMENTS

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The metallurgical testing and evaluation was performed by Dr. John D. Wood and Mr. Robert A. McDemus under the direction of Dr. Alan W. Pense in the Department of Metallurgy and Materials Engineering. Charpy tests were performed by Mr. Robert R. Dales. Dr. John W. Fisher provided advice on fracture mechanics.

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February 21, 1979

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From: Dr. John D. Wood
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Subject: AISI Engineering Subcommittee
Structural Steel and Plate Producers Committee
Project 192 - Beam to Column Web Connections
Fractures in Connections 14-1 and 14-3.

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SUMMARY

Both connector-beam assemblies failed in a predominately brittle manner although significant ductility was present on both fracture surfaces as indicated by shear lips or crack arrest markings. The factors contributing to these fractures are:

1) A large amount of plastic strain was imposed on the assemblies.
2) The strain was concentrated at design details (radius and/or welding passes).
3) The connector plates were oriented so that the applied strain was in the least fracture resistant direction (the transverse direction in the plates).

Since it is possible for acceptable steel to fracture in a predominately brittle manner if strained significantly and strain concentrators are present, it is not surprising that these connector-beam assemblies failed as they did. It is very possible that other assemblies with similar design details would fail in a substantially brittle manner if strained to a large amount.

DISCUSSION OF FRACTURES

I. Connection 14-1

This fracture initiated at the edge of the weldment (on the beam flange of the connector, roughly parallel to the weldment (perpendicular to the applied strain). The initiation site appears to be at the edge of the weldment where the design detail created a strain concentration when the flange of the connector was reduced in width by a radius to join the narrower beam flange (also weld irregularities could have contributed to the strain concentration).

The fracture was predominately brittle in nature although there is evidence of several crack arrests which indicates some ductility in the region. There is no evidence of defects on the fracture which could have contributed to the failure.

The composition of the tension flange of the connector was within the requirements of ASTM A572-Grade 50 (see Table 1). The hardness of the flange is HRB-90. The rolling direction of the connector flange was perpendicular to the applied tensile strain (i.e. rolling direction was parallel to the fractured plane). Metallographic examination of the weldment indicated that defects (voids and fusion line cracks up to 1/10 of an inch in maximum dimension) were present; however, these defects were not greater in size that what might be typically found in structural welds.

Although some defects were present in the weldment, it is felt that this connection fractured in a brittle manner due to the large applied tensile strain which was concentrated at the design detail.

II. Connection 14-3

This fracture initiated at the end of the connector tensile flange where it was welded to the flange of the column and propagated across the tensile flange of the connector normal to the applied tensile strain. The initiation site appears to be at a cosmetic welding pass between the edge of the column flange and the side of the connector plate.

The fracture was predominately brittle, but shear lips are present indicating some ductility in the connector tensile flange. There is no evidence of any defects on the fracture surface. The composition of the connector flange was not within the limits of ASTM A572 Grade 50 but meets the requirements of ASTM A588B (see Table 1). The hardness of this flange is HRB-84. The rolling direction of the connector flange was normal to the applied strain, so that the applied strain was in the transverse direction of the plate.

This connector appears to have failed when a large amount of strain was applied and the strain was concentrated at a design detail (the edge of the column flange and the connector flange) which was made worse by the use of a cosmetic edge welding pass.

TABLE 1-- APPENDIX

Chemical Analysis of Connection Tensile Flanges

	14-1	14-3
Carbon	0.22	0.13
Sulfur	0.16	0.019
Phosphorous	0.020	0.015
Silicon	0.02	0.19
Manganese	1.042	0.906
Chromium	0.020	0.556
Copper	0.257	0.312
Nickel	0.009	0.276
Molybdenum	0.002	0.047
Vanadium	0.040	0.020
Aluminum	NIL	0.059

TABLE 1

CHARPY TEST RESULTS FOR CONNECTION 14-1

CHARPY IMPACT SERIES A572

DESCRIPTION PROJECT 405

CONNECTION 14-1
TENSION PLATE
~~3/4 INCH THICK~~

TEST RESULTS

SPEC NO	TEST TEMP (DEG F)	ENERGY (FT-LB)	LAT EXP (MILS)	PERCENT FIB FRACT
1	-40.0	3.0	2.0	1.0
2	-40.0	3.0	3.0	1.0
3	-40.0	3.0	1.0	1.0
4	0.0	9.0	5.0	1.0
5	0.0	10.0	4.0	1.0
6	0.0	14.0	7.0	1.0
7	40.0	17.0	12.0	10.0
8	40.0	12.0	9.0	10.0
9	40.0	17.0	9.0	10.0
10	70.0	21.0	11.0	10.0
11	70.0	22.0	11.0	20.0
12	70.0	25.0	14.0	30.0
13	100.0	27.0	14.0	40.0
14	100.0	26.0	15.0	40.0
15	100.0	28.0	16.0	30.0

THE NUMBER OF SPECIMENS IN THIS SERIES WAS 15

CHARPY IMPACT SERIES A572

TRANSITION TEMPERATURES

IMPACT ENERGY CRITERION

ENERGY	TEMPERATURE	
10 FT-LB	2.7 DEG F	
15 FT-LB	27.0 DEG F	
20 FT-LB	53.9 DEG F	
25 FT-LB	101.5 DEG F	
26 FT-LB	128.5 DEG F	
26 FT-LB	128.5 DEG F	
26 FT-LB	128.5 DEG F	
26 FT-LB	128.5 DEG F	
26 FT-LB	128.5 DEG F	
26 FT-LB	128.5 DEG F	
26 FT-LB	128.5 DEG F	
15 FT-LB	28.5 DEG F	(EU-EL)/2 + EL
13 FT-LB	18.4 DEG F	INFLECTION

TABLE 2

CHARPY TEST RESULTS FOR CONNECTION 14-3

CHARPY IMPACT SERIES A588

DESCRIPTION PROJECT 405

CONNECTION 14-3
TENSION PLATE
1 INCH THICK

TEST RESULTS

SPEC NO	TEST TEMP (DEG F)	ENERGY (FT-LB)	LAT EXP (MILS)	PERCENT FIB FRACT
1	-40.0	5.0	3.0	1.0
2	-40.0	32.0	20.0	1.0
3	-40.0	7.0	4.0	1.0
4	0.0	109.0	45.0	40.0
5	0.0	81.0	36.0	20.0
6	0.0	91.0	40.0	30.0
7	-30.0	74.0	33.0	30.0
8	-30.0	28.0	13.0	1.0
9	-30.0	39.0	18.0	10.0
10	-20.0	84.0	37.0	20.0
11	-20.0	77.0	34.0	20.0
12	-20.0	60.0	27.0	10.0

THE NUMBER OF SPECIMENS IN THIS SERIES WAS 12

CHARPY IMPACT SERIES A588

TRANSITION TEMPERATURES

IMPACT ENERGY CRITERION

ENERGY	TEMPERATURE	
16 FT-LB	-49.6 DEG F	
16 FT-LB	-49.6 DEG F	
20 FT-LB	-43.6 DEG F	
25 FT-LB	-39.7 DEG F	
30 FT-LB	-36.8 DEG F	
35 FT-LB	-34.3 DEG F	
40 FT-LB	-32.0 DEG F	
45 FT-LB	-29.8 DEG F	
50 FT-LB	-27.7 DEG F	
55 FT-LB	-25.7 DEG F	
60 FT-LB	-23.6 DEG F	
62 FT-LB	-22.6 DEG F	(EU-EL)/2 + EL
54 FT-LB	-25.7 DEG F	INFLECTION

TABLE 3

TRANSVERSE CHARPY TEST RESULTS

LT Specimens Transverse to Loading Direction

CONNECTION	SPEC NO	TEST TEMP (DEG F)	ENERGY (FT-LB)	LAT EXP (MILS)	PERCENT FIB FRACT
14-1	16	40.	36.	16.	1.
	17	40.	19.	10.	10.
	18	40.	39.	16.	10.
14-3	13	-30.	14.	8.	1.
	14	-30.	10.	6.	1.
	15	-30.	12.	8.	1.

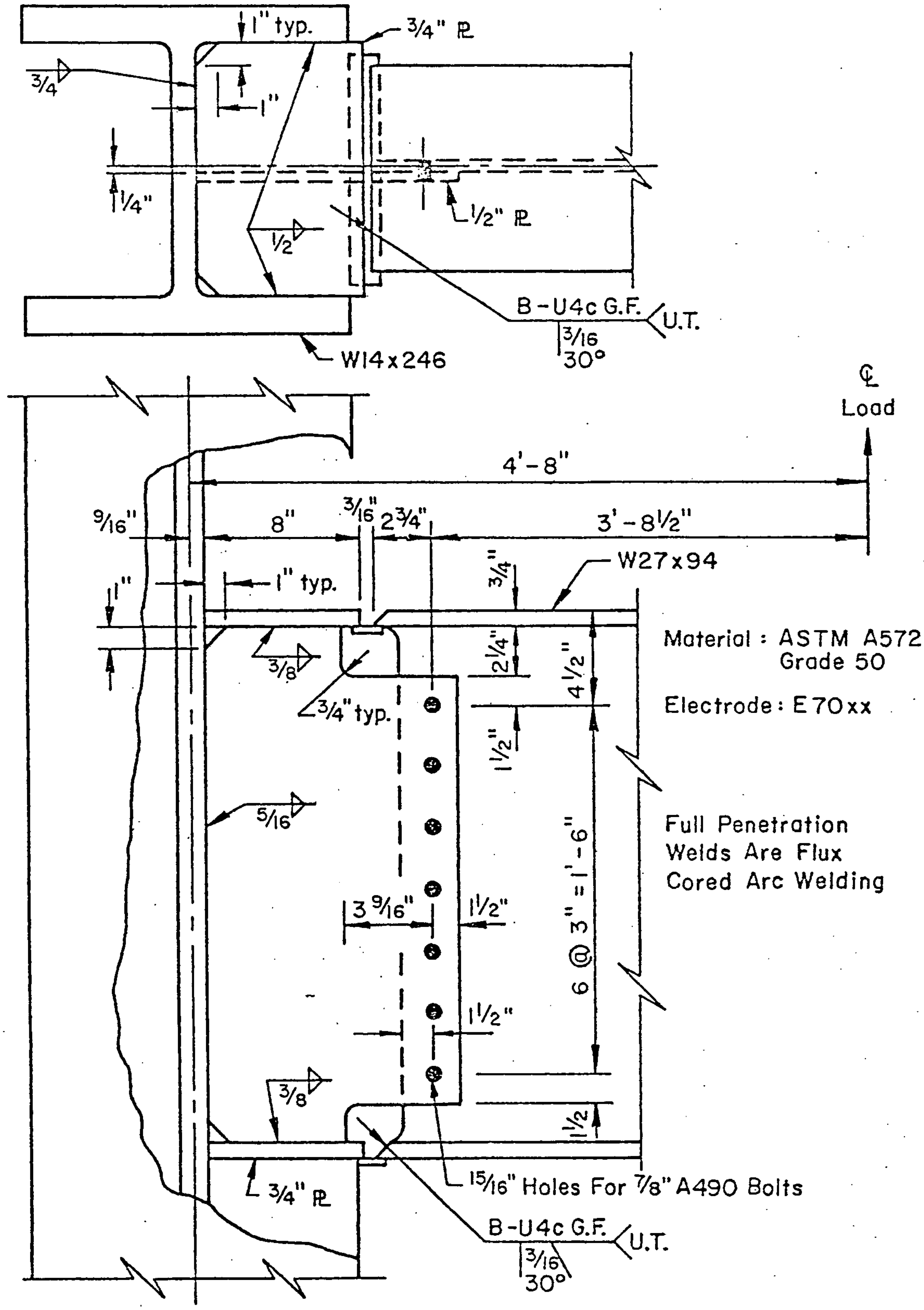

Fig. 1 Details of Connection 14-1

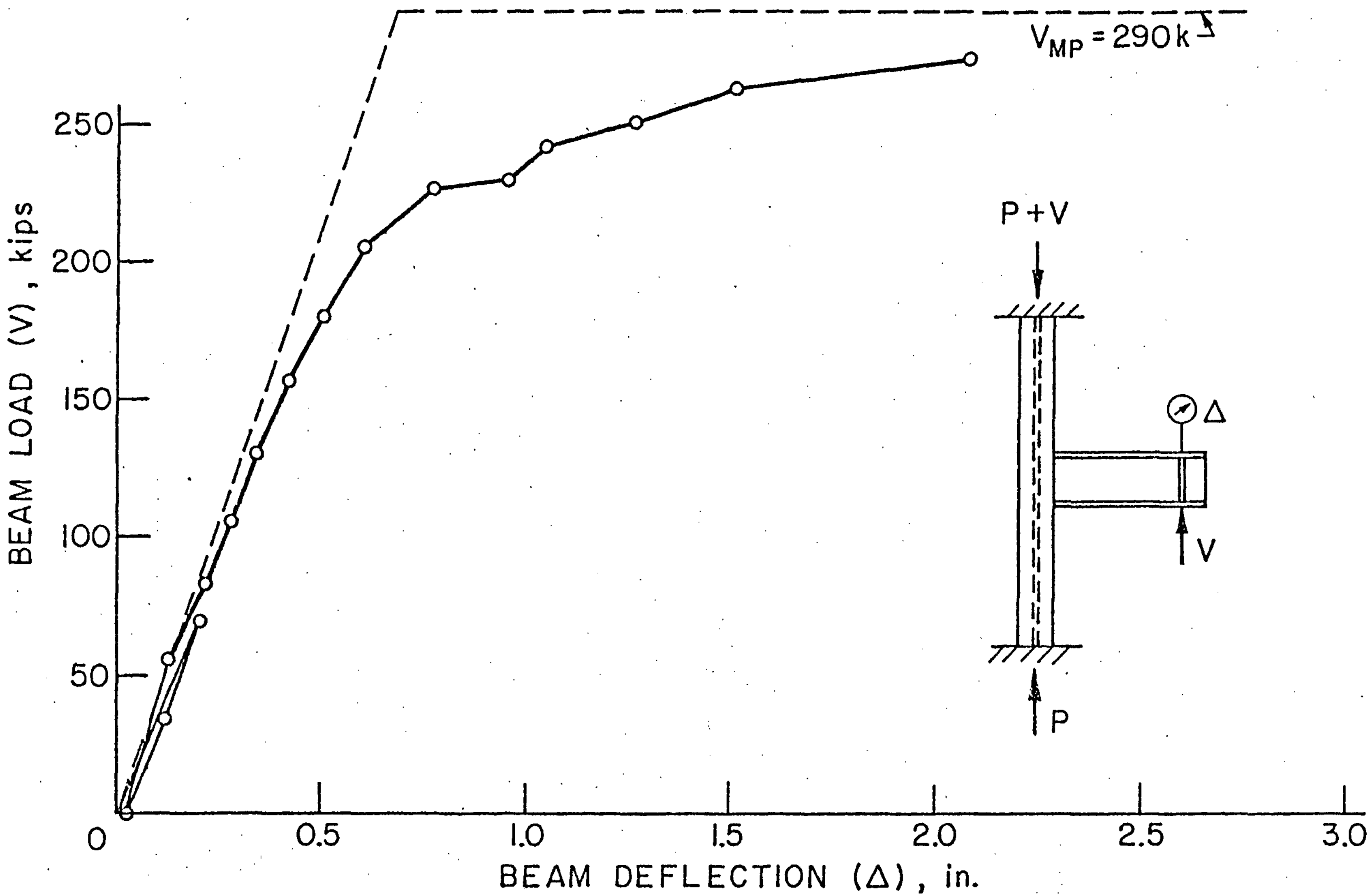

Fig. 2 Load-Deflection Curve of Test 14-1

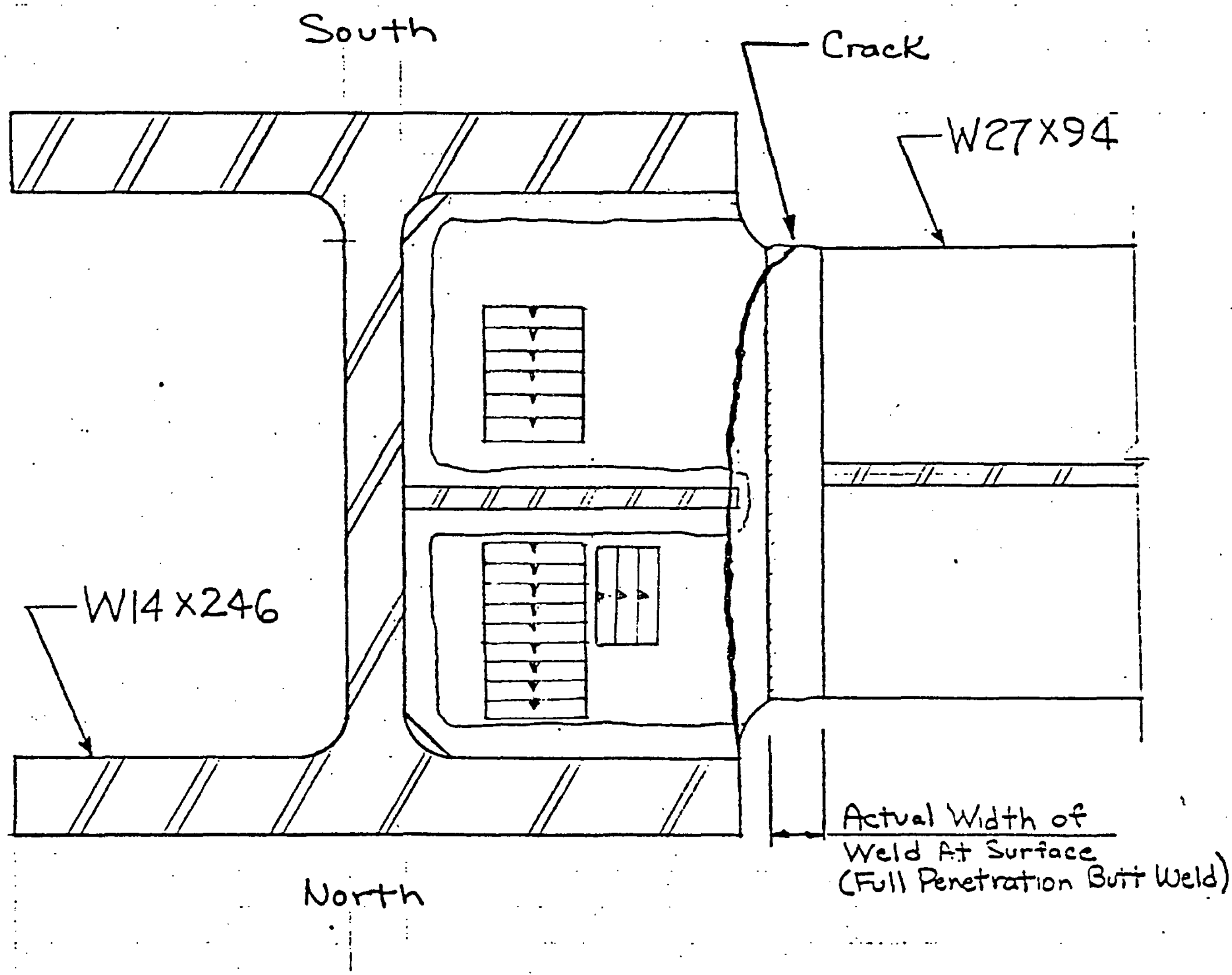

Fig. 3 Failure Location in Test 14-1

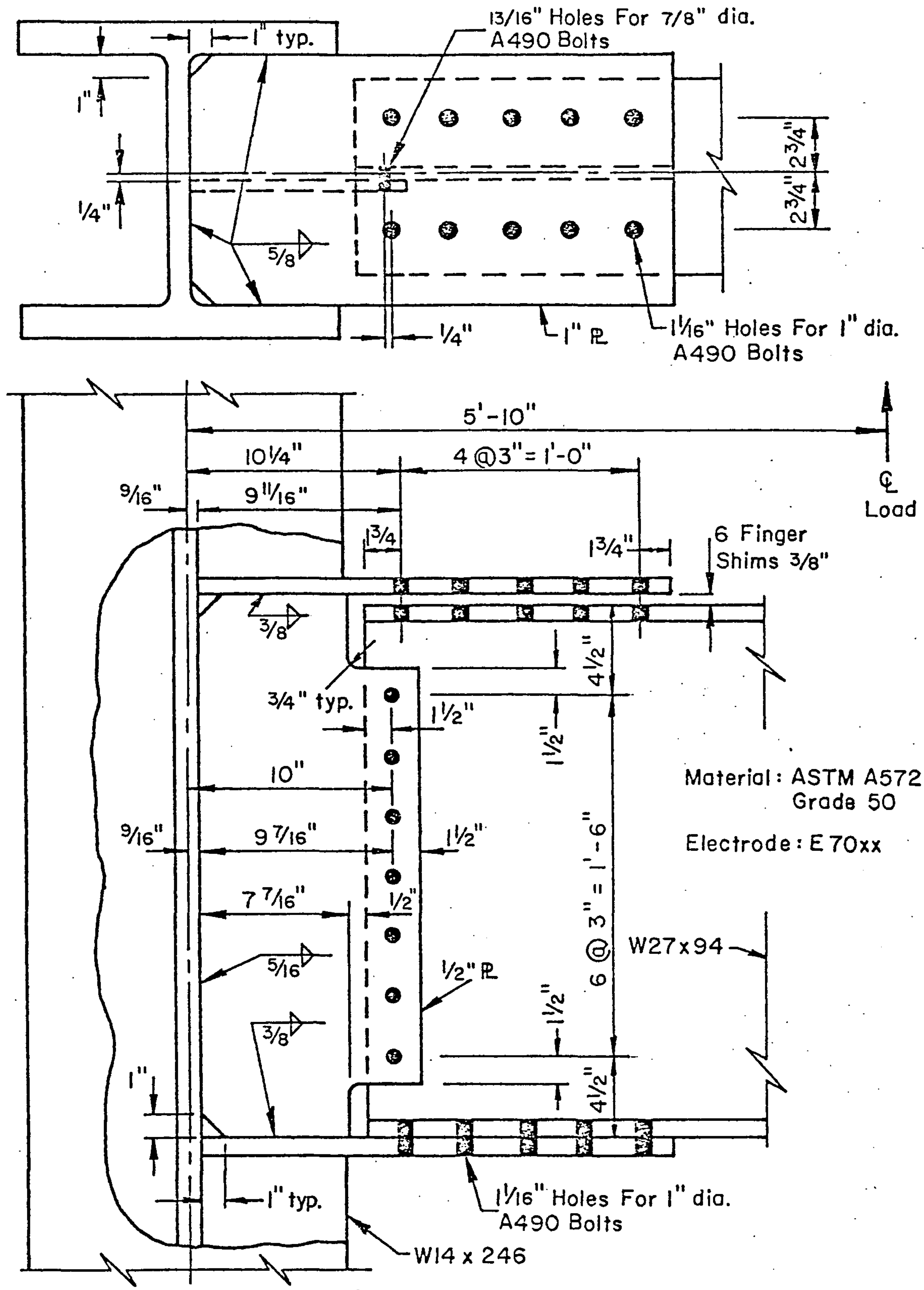

Fig. 4 Details of Connection 14-3

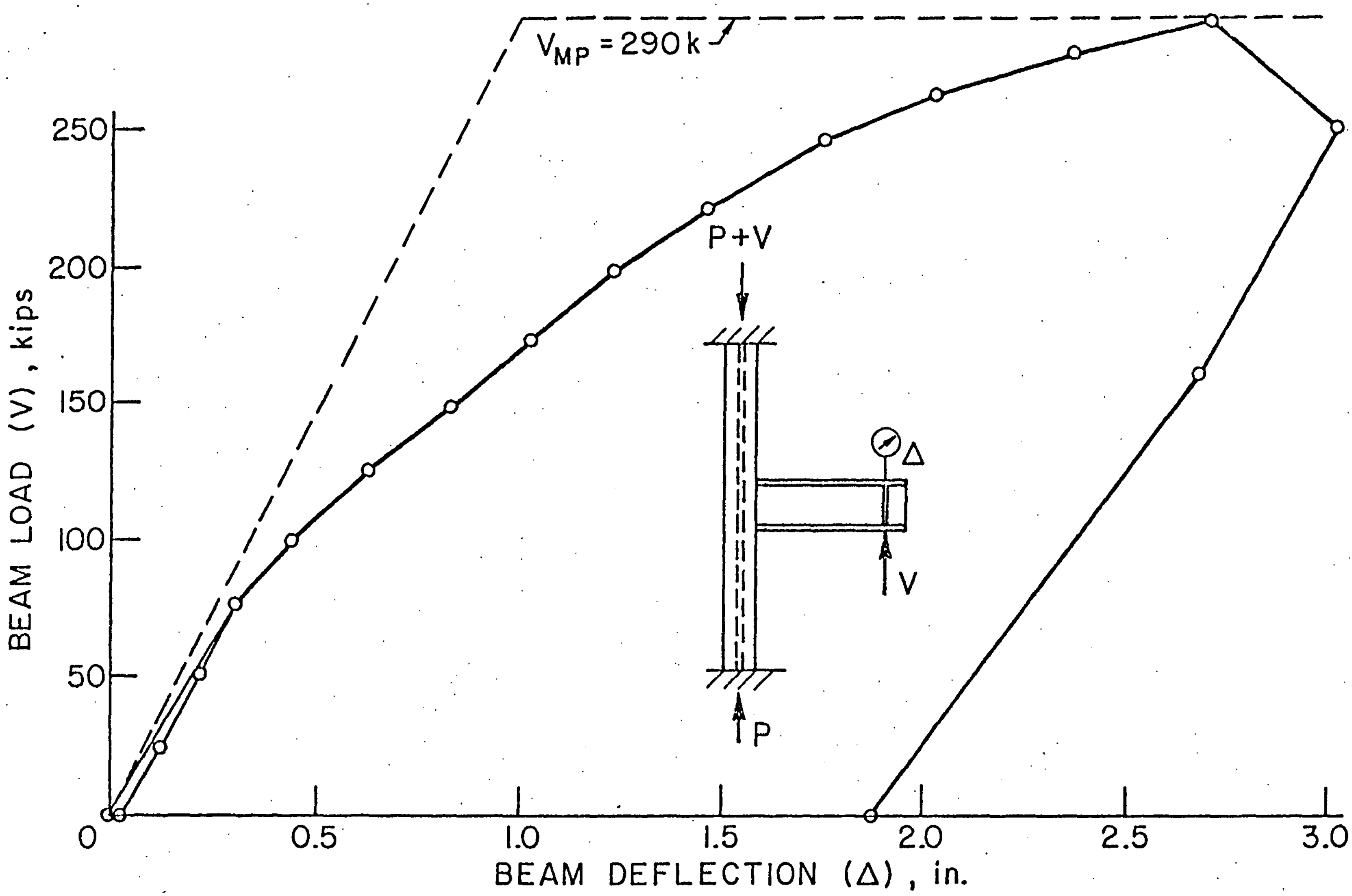

Fig. 5 Load-Deflection Curve of Test 14-3

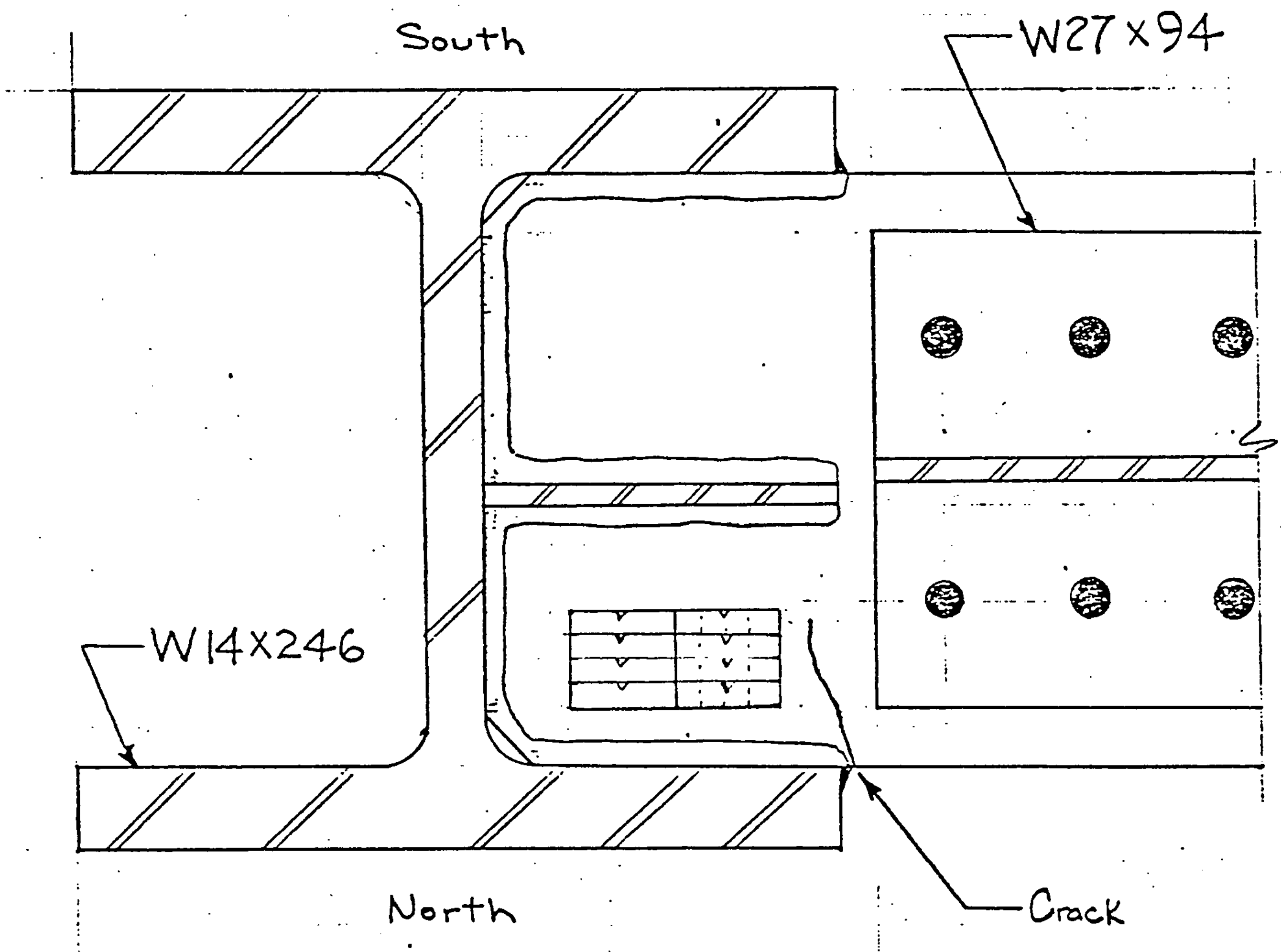

Fig. 6 Failure Location of Test 14-3

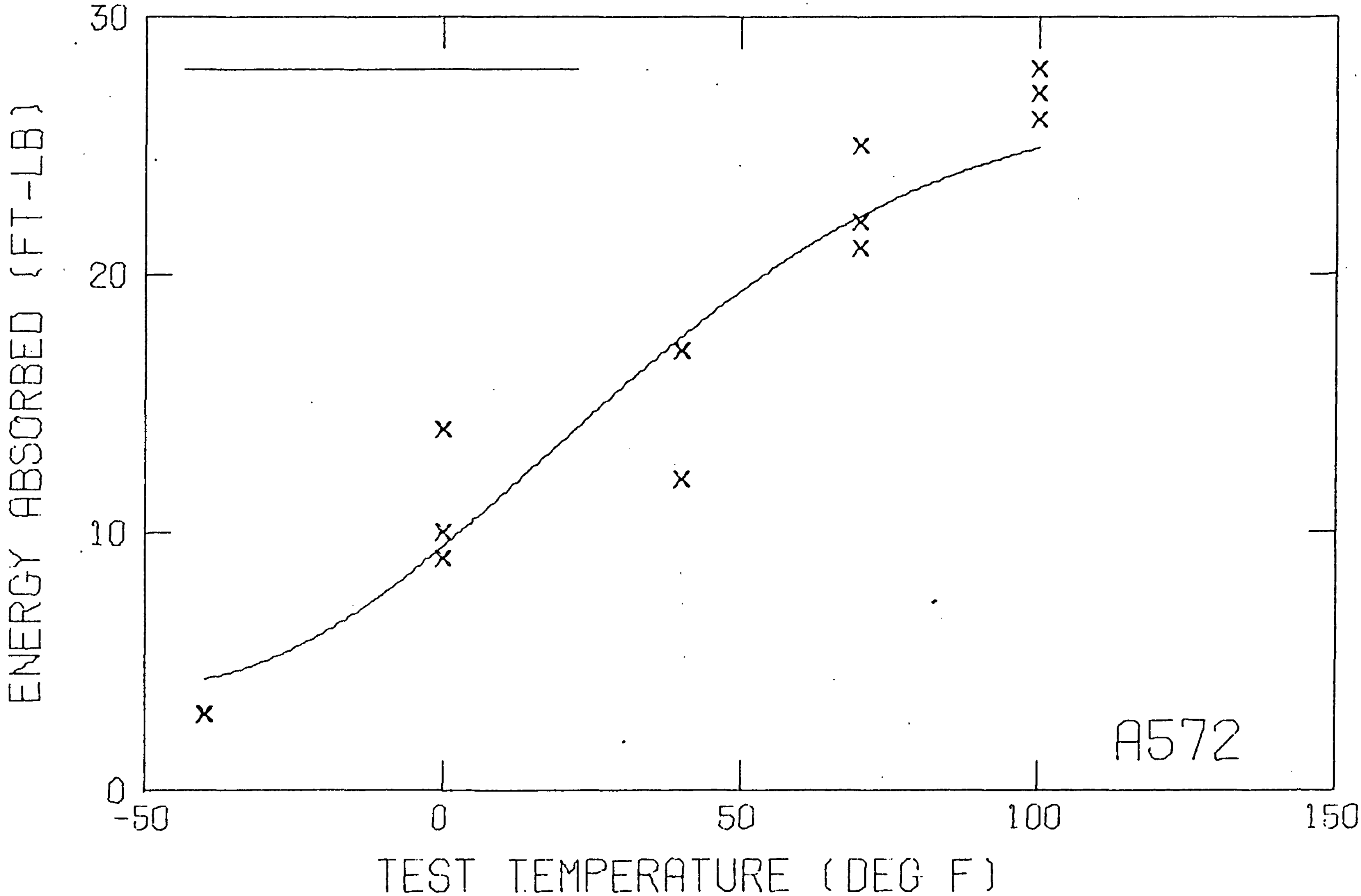

Fig. 7 Charpy Test Results for Connection 14-1

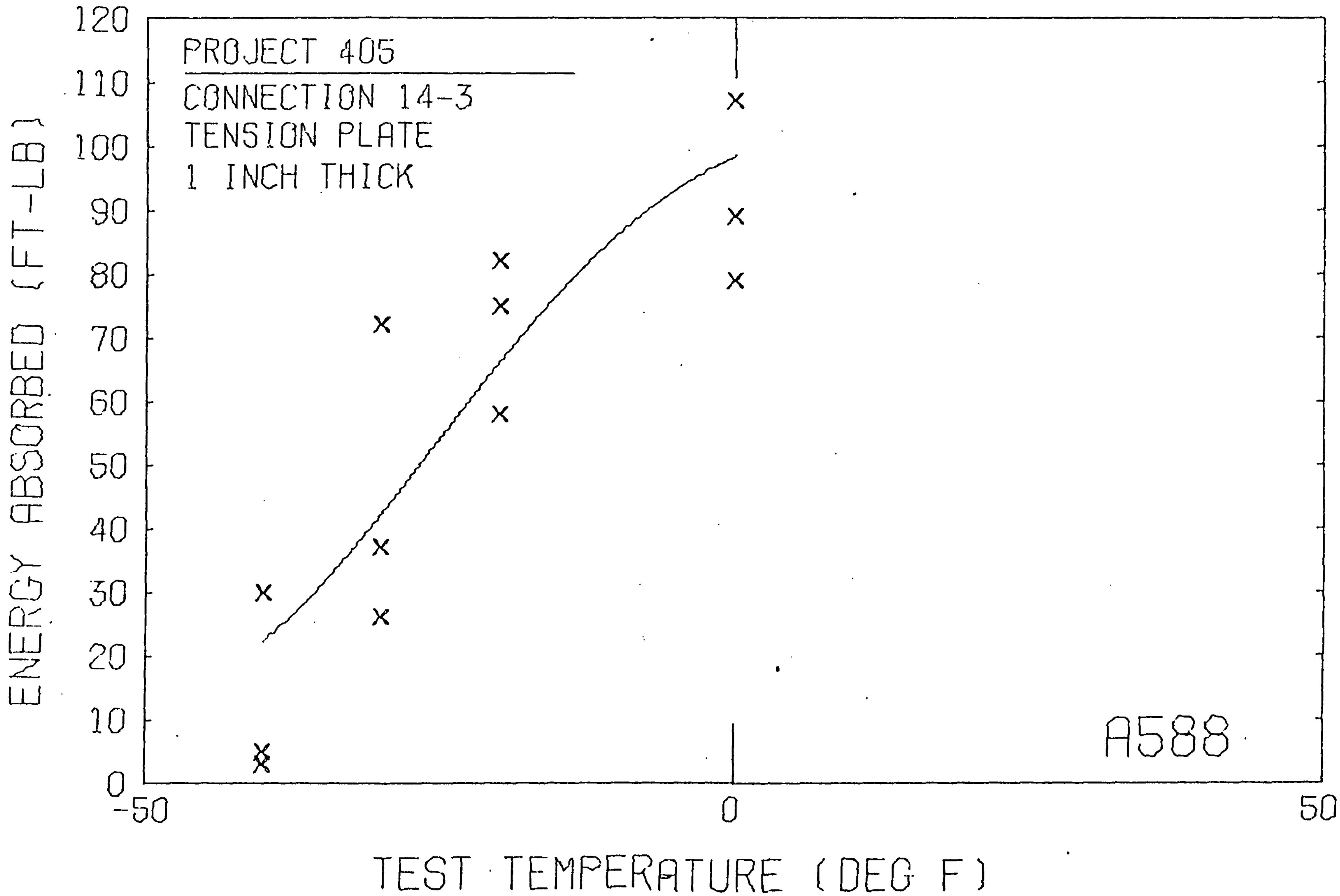

Fig. 8 Charpy Test Results for Connection 14-3

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If retyped, indent this column of names

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TO: Members, WRC Task Group on Beam-to-Column Connections

J. A. Gilligan (Ch) ✓	N. W. Edwards ✓	H. A. Krentz ✓
V. V. Bertero ✓	W. E. Edwards ✓	W. A. Milek, Jr. ✓
O. W. Blodgett ✓	H. J. Engstrom, Jr. ✓	C. W. Pinkham ✓
V. R. Cartelli ✓	E. R. Estes, Jr. ✓	F. Schneller ✓
H. C. Crick ✓	T. R. Higgins ✓	A. N. Sherbourne ✓
C. F. Diefenderfer ✓	I. M. Hooper ✓	F. W. Stockwell, Jr. ✓
		N. W. Young ✓

FROM: G. C. Driscoll

SUBJECT: FRACTURES OF BEAM-TO-COLUMN WEB MOMENT CONNECTIONS

At our last meeting on July 20, 1977, there was discussed a need for studies of the causes of fractures of the connections 14-1 and 14-3 in the series on beam-to-column web moment connections.

Subsequently, funding was received ~~from AISI and ASCE~~ for the specific purpose of conducting an ~~a further brief~~ investigation into the fractures, including the quality of the plate involved. The investigation was conducted at Fritz Laboratory and in the Department of Metallurgy and Materials Engineering at Lehigh.

The results of the fracture investigation are presented in ~~Eventually~~ the report entitled FRACTURES OF BEAM-TO-COLUMN WEB MOMENT CONNECTIONS by George C. Driscoll, Fritz Engineering Laboratory Report No. 405.10, March 1979. The report includes and the metallurgists' report. ~~It has been previously sent to J. A. Gilligan. He has now approved its distribution~~ to the Committee. (A)

indicated that the report can be sent

PS. The results of the original connection tests are presented in the May 1980 ASCE ST5 in a report entitled TESTS OF BEAM-TO-COLUMN WEB MOMENT CONNECTIONS, by Glenn P. Rentschler, Wai F. Chen, and George C. Driscoll. This replaces the preprint listed in 405.10. make this a PS?

Copies of Report No. 405.10 are enclosed for your information and discussion. We are hoping eventually to publish this work. We will be contacting you later about this. Meanwhile we need your comments.

George C. Driscoll

GCD:esn

Enclosure

cc: L. S. Beedle ✓
A. C. Kuentz ✓
L. W. Lu ✓
C. R. Felmley Jr. ✓

~~(A) Maybe delete after all. Doesn't make anybody look too good.~~

5-20-80

I want the references to show both the Preprint and the final publication with the asterisk, because the text refers to the "preprint".

We will reproduce the report and letter with no cover.

Sufficient copies for committee and project staff with a few left over.

To be mailed.