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Highlights of moment connection detail test no. 1, December 1981.

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Fracture of Moment Connections



**HIGHLIGHTS OF
MOMENT CONNECTION DETAIL**

TEST NO. 1

FRITZ ENGINEERING
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by

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**Fritz
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1 ABSTRACT

A tension test of a simulated detail of a beam-to-column web moment connection was conducted as part of the experimental work in a study to determine methods to prevent fracture of this type of connection. Preliminary results including theoretical and experimental load-deflection curves and a description of a failure by fracture are described. The specimen successfully sustained loads up to approximately 1.35 times the nominal yield stress of the material and overall elongations of a simulated beam tension flange of approximately 6 times the nominal elongation to cause yield. A fracture completely severed the simulated beam tension flange after this load was achieved. Initiation of the fracture was at tack welds used to fasten a backing bar provided to permit groove-welding the simulated beam flange to a connection plate. The fracture suggests the need for further attention to details involving tack welds in connections where ductility is required such as in structures designed by plastic design, structures designed for earthquake resistance, or structural designs which take advantage of the redistribution of moment provisions of allowable stress design. A non-linear elastic-plastic analysis gave good agreement with the experimental load-deflection behavior up to the nominal yield stress of the specimen.

The nonlinear elastic-plastic analysis was conducted in preparation for fracture analyses of the connection which will constitute the main analytical part of this research. However, a nonlinear fracture analysis might be more suitable in this task.

Results of this and other tests plus a theoretical analysis of the connection detail will constitute a basis for guidelines and recommendations for future design of beam-to-column web moment connections.

2 INTRODUCTION

The concept for the current research program originated as a result of a series of tests performed to investigate the strength of full-scale beam-to-column web moment connections [Rentschler, 1980].

Two out of four specimens tested exhibited premature failure due to fracture. It was suspected that material defects or unsatisfactory welding were the cause of fracture. However, further metallurgical and chemical tests ruled out this possibility. The tests indicated that the material and welds were reasonably normal. It was concluded that the primary causes of the fractures were stress conditions which were due to restraint provided by fastening the tension flanges of the beams to the heavy column flanges [Driscoll, 1979]. The current research program was proposed to determine how to avoid fracture problems in beam-to-column web connections.

The research plan was to first isolate the tension flange connection detail and then apply the results to a study of the whole connection. A physical and mathematical model of the tension flange detail was formulated consisting of a short length of a column, a flange connection plate, and a plate simulating the tension flange of the beam. In this model, the principal variable was the flange connection plate. Variations in the shape, thickness, and pattern of welding to the column provided the range of different details studied. The connection plate and the simulated tension flange plate were to be fastened perpendicular to the web plane of a 4-foot long column stub at its center. This assemblage would be supported as a simple beam at the ends of the column stub. A tensile force which would create moment about the weak axis of the column would be applied on this tension plate. Effects of the beam web, beam shear connection, and the beam compression flange were neglected in the preliminary model. This assumption, with a rudimentary analytical justification, was made in order to achieve an economical model for both computational and experimental purposes.

A parametric study of elastic stress distributions of several details was carried out using a linear finite element program [Bathe, 1974]. The parametric analysis was conducted as a first step, to

evaluate the effect of different thicknesses and geometries of the flange connection plate on stress concentration. This analysis helped select appropriate details for a second step of analysis. A complete report on the elastic parametric study has been prepared [Shen, 981a].

An elastic-plastic step-by-step load-deflection analysis was carried out on a series of simulated tension flange connection details adopted from the results of the parametric analysis. Figure 1 shows layout and dimensions of the plates used in the elastic-plastic analysis. Case A represents most nearly one of the specimens that fractured in the full-scale beam-to-column web connection tests. Case B is the same detail, but with a 1-inch thick stiffener supplied. Both case A and B were analyzed with the joint of the simulated beam flange flush with the plane of the tips of the column flanges, although it would not be a suitable joint for welding. However, no specimens were fabricated this way. Case C has the connection plate tapered and extended 3 inches beyond the flange tips to move the joint to the beam flange away from the zone of restraint between the column flanges. Case D is essentially the same detail as case A, but a thicker 1-5/8 inch connection plate is used to reduce the stresses in the connection plate. Case E uses a 1-5/8 inch connection plate, but no weld is to be applied between the connection plate and the column web. Case F has the same detail as case C but has a thicker connection plate to investigate the effect of reducing the basic stresses. Case G was studied to observe the effect of an extended tapered connection plate not welded to the column web. The analysis of these details was carried out through a step-by-step procedure using the linear finite element program SAP4 [Pathe, 1974]. The results of these studies have been reported separately [Shen, 981b]. That report contains step-by-step load-deflection curves, stress distributions, and sketches of yielding patterns.

Based on the two analytical studies, a phase of design, fabrication, and testing of sample details was begun. This report covers the highlights of the first simulated connection detail test.

3 DESCRIPTION OF TEST

3.1 Test Program

Certain of the details previously analyzed were to be tested with and without backup stiffeners opposite the tension flange connection plate. An overall view of the test setup is shown in Fig. 2 and a list of tests with their schematic details has been summarized in Table 1. Specimens A and C represent the corresponding cases in Fig. 1. Specimen B represents case A, but with an extended connection plate (about 3 inches). Specimen D represents case E of Fig. 1 and specimen E represents case G.

The effects of different thicknesses and geometries and the contribution of the column web in transferring tensile force into the column flanges were considered in different specimens. The contribution of the column web in transferring stresses came into effect via the presence of a fillet weld along the flange connection plate and column web.

3.2 Specimens

Fig. 2 gives a schematic view of a typical specimen and its position in the test setup. The specimens were fabricated from ASTM-A572 grade 50 steel with a column size of W14x257. This column section was chosen to simulate the connection size which had failed in previous tests [Rentschler, 1980]. For the same reason, a 10-inch wide tension flange plate was adopted, simulating the flange of a beam. The length of the plate was 5.0 feet to provide adequate distance from the grips of the testing machine and to ensure that St. Venant's principle holds.

Two mechanical property tests have been carried out at this stage of testing. The yield stress level of the material was approximately 54.0 ksi. Fig. 3 shows the stress-strain curve which was obtained from one of these tensile tests.

Welding was in accordance with the AWS Building Code [American Welding Society, 1980]. All welds were fabricated using E-70XX electrodes. Fillet weld sizes were measured and checked against their nominal values and all were sound without cracks according to a visual

inspection.

ASTM-A490 bolts were used in assembling the bolted parts. All bolted joints were designed as bearing type and an allowable design stress of 40 ksi, in shear, was used.

Because of the relative strengths of the column member and the plate material, failure would occur in the tension plate, but not in the column. Because of the survival of the column section, each stub column could be used twice, once without a back-up stiffener and then with a back-up stiffener. In order to reuse the specimen, the flange connection plate would be burned off at the tip of the column flange and parallel to the column web to remove the remaining damaged parts of the first test. Then a fresh connection plate and a flange plate would be welded to the reverse side of the column, serving as a new test specimen. The burned-off connection plate from the first test would serve as the back-up stiffener.

3.3 Test Fixture

The test fixtures are also depicted schematically in Fig. 2. Two reusable bolted end plates were bolted to welded end plates on the specimen. The end plates were bolted in turn to a fixture nearly the same in appearance as a specimen, but much stronger in order that it would resist large forces and survive the full series of tests. By creating an axial alignment of specimen and test fixture, the benefits of symmetry would simplify testing. The same column size was used for both fixture and specimen. However, the column of the test fixture was oriented so that the applied tensile force would produce bending about the strong axis of the fixture. The tension plate of the fixture was 2.0 in. thick, twice as much as that of the specimen.

3.4 Test Setup

The test was carried out using the 5-million lb. Universal Testing Machine which furnished the required tensile loading, applied at the far ends of the tension flange plate and the fixture tension plate on a gripping length of two feet each. Strains and deflections were measured using electrical resistance wire strain rosettes and linear variable displacement transformers (LVDT's) at different locations shown in Fig.

4. The reference point for measuring the relative deflections between points a-b and a-c was fixed with respect to the testing machine columns. Data from strain and deflection gages was collected and printed automatically using a B&F Data Acquisition System at each loading step. This saved time while eliminating the ever-present possibility of error in transcription.

3.5 Details of Specimen (Test 1)

Test 1 was a control test and it had the most resemblance to one of the original full-scale connections that fractured during testing. Both the tension flange and the flange connection plate had a thickness of 1.0 in. The tension flange plate was groove-welded to the flange connection plate using a backing-bar tack-welded parallel to the groove weld at six different points. The flange connection plate was not exactly flush with the tip of the column flange but projected about 0.75 in. as shown in Fig. 5. The fillet welds between the flange connection plate and the column flange had end returns across the thickness of the flange connection plate at the tips of the column flanges. Finally, both returned welds intersected at top and bottom of the connection plate (see Fig. 5).

4 Test Results

A load-deflection curve of Test 1 is given in Fig. 6. Because of the uncertainty of the final outcome of this first test, load was applied in many small increments (many more than the number plotted). At the end of the first day's testing, the specimen was partially unloaded after having reached about 93% of the predicted full plastification load. Testing was resumed on the following day. After first unloading completely to permit adjustment of equipment, loading was resumed. At the time the indicated load approximated nominal yield stress across the entire flange plate, strain gages gave indications of local yielding at several places on the specimen. From that point on, load increased at nearly constant slope equivalent to about $0.62 E$ until a sudden fracture occurred.

The maximum load occurred at 730 kips, equivalent to 73 ksi ultimate stress, about 35 percent greater than the nominal yield stress. Fracture

occurred after a fairly extensive elongation over six times the elongation required for nominal yield of the tension flange, as shown in Fig. 6. The theoretical curve obtained from the elastic-plastic finite element analysis, computed up to the yield point, is plotted as a dashed line in the same figure. Whitewash had been applied to the specimen in order to observe flaking of mill scale caused by surface yielding of the specimen. However, very little whitewash flaking had been observed before the specimen failed abruptly with a loud noise. A close inspection of the fractured surface (as shown in Fig. 7) revealed that a crack had initiated at one of the outer tack welds and propagated rapidly across the flange tension plate. Fig. 8 shows the trace of the crack near the outer edge of the backing bar. The region near the tips of the column flanges where fracture had been presumed to be most probable, was not involved in the fracture.

5 Plans for Re-Testing

Since the fracture which occurred in Test 1 did not involve the highly-stressed region between the column flanges, it was decided to find if a connection which did not have the tack welds to hold the backing-bar for welding would perform as expected. Also it was desired to see if any imminent cracking had been started in that zone but not observed because of the premature fracture in the tension plate.

No cracks were discovered by the available non-destructive testing methods. The old groove weld, backing-bar, and a slice of the connection plate were removed by sawing (to avoid adding any new heat effects) and the specimen was prepared for re-test by welding on a new 10 by 1 inch tension flange plate. In this case, the groove weld was applied using a backing-bar tacked at the root of the weld, where the tack welds would be fused into the final groove weld.

Since five specimens had already been welded using the unfortunate arrangement of tack welds, it was decided to find if the effects of the tacks welds could be reduced by other means short of replacement of the entire tension plate and groove weld. Specimen B of Table 1 was selected because it might most closely approximate the detail of Test 1. The six tack welds were removed by grinding with a pencil grinder. This

procedure would not remove the residual stresses caused by the welding of the tack welds, but it could release some of the local restraint between the backing bar and the main plates.

Both of these specimens (Specimen A re-test and Specimen B) will be tested soon, and then further decisions will be made about the disposition of the remaining specimens in the series.

6 Conclusions

The specimen tested showed considerable ductility before fracturing. It reached the expected yield load at about the expected stress. It reached a ductility ratio of about 6. The maximum nominal stress of 73 ksi approached the expected tensile stress of the material used.

The fracture at the location of the tack welds was a surprise, although it should not be unexpected in view of experiences with fracture under conditions of fatigue. It should be noted that the backing-bar and tack welds used would be removed in bridge construction, although they are frequently left in place in building construction when they are used. The test has provided a valuable lesson that we can not be careless about details involving tack welds, even in statically-loaded construction.

The macroscopic appearance of the fractured surface indicated the existence of plane strain test conditions, hence, a low fracture toughness of the specimen, although not necessarily of the material. Crack initiation from a tack weld of a backing-bar indicates its deleterious effect in tensile stress zones due to the residual stresses left behind by welding. Once a crack started in the tension flange plate, it propagated all the way through the plate without an ability to follow a path perpendicular to the theoretical maximum tensile stress direction, a curved path which was observed in the full-scale connection which fractured in the prior test series.

One additional interesting observation may be made about the effects of preparing specimens as was done in this series. There is a noticeable difference in the nature of residual stresses in the tension plate of the detail specimen and a rolled beam flange. Namely, while residual stresses at the tip of rolled beam flanges are compressive stresses,

those of the tension plate are tensile stresses because of heat cutting. This introduces high stresses which may exhaust some of the ductility and may be considered responsible for crack initiation in the flange tension plate. It appears that there are certainly more things to be learned from this test program.

7 ACKNOWLEDGEMENTS

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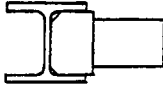
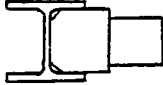
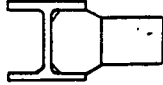


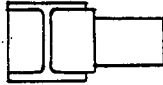
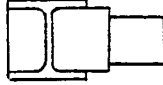

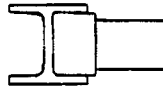

SPECIMEN NO.		TENS. FLG. P. THICKNESS (IN.)	CONN. P. THICKNESS (IN.)	STIFFENER THICKNESS (IN.)
A	 CONTROL TEST	1	1	—
B	 EXTENDED CONNECTION PLATE	1	1	—
C	 TAPERED CONNECTION PLATE	1	1	—
D	 UNWELDED COLUMN WEB	1	1 ⁵ / ₈	—
E	 //	1	1 ⁵ / ₈	—
A2	 CONTROL TEST WITH BACK-UP STIFFENER	1	1	1
B2		1	1	1
C2		1	1	1
D2	 WITHOUT BACK-UP STIFFENER	1	1 ⁵ / ₈	—
E2	 WITHOUT BACK-UP STIFFENER	1	1 ⁵ / ₈	—

Table 1: Test Specimen Configurations

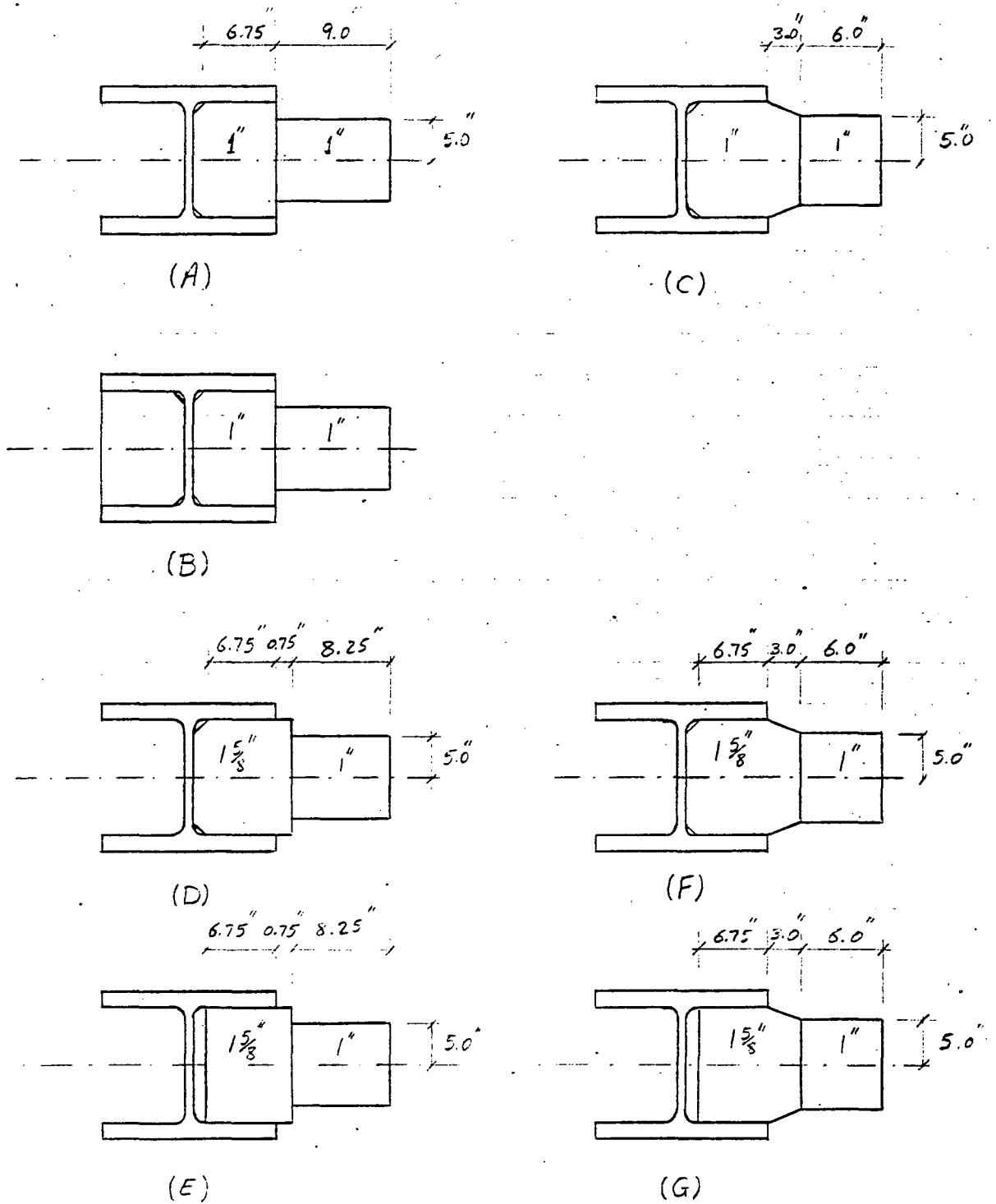


Figure 1: Dimensions of Connection Details Analyzed

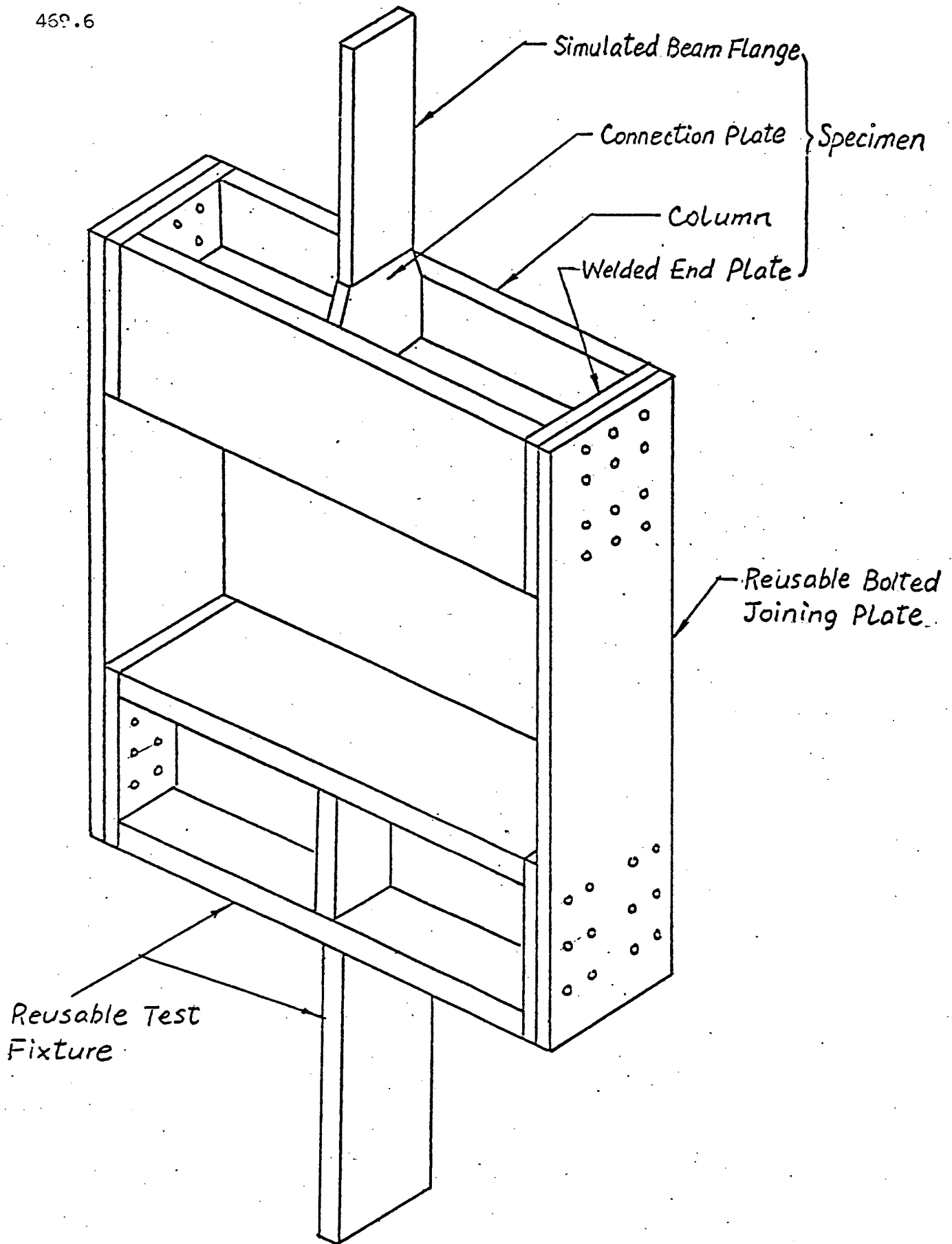
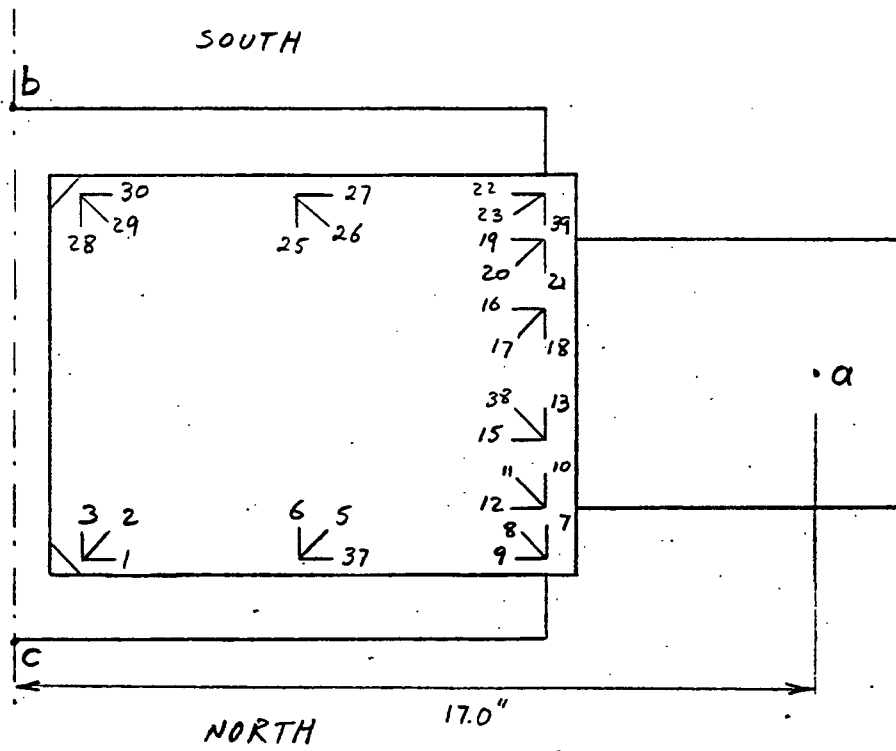
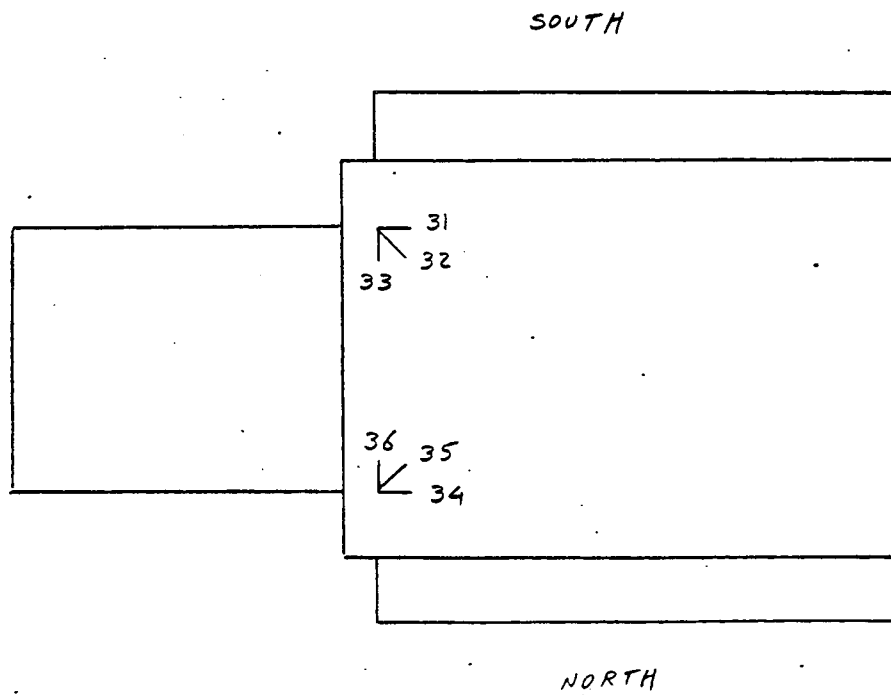


Figure 2: Test Setup



TOP VIEW



BOTTOM VIEW

Figure 4: Layout of Rosettes

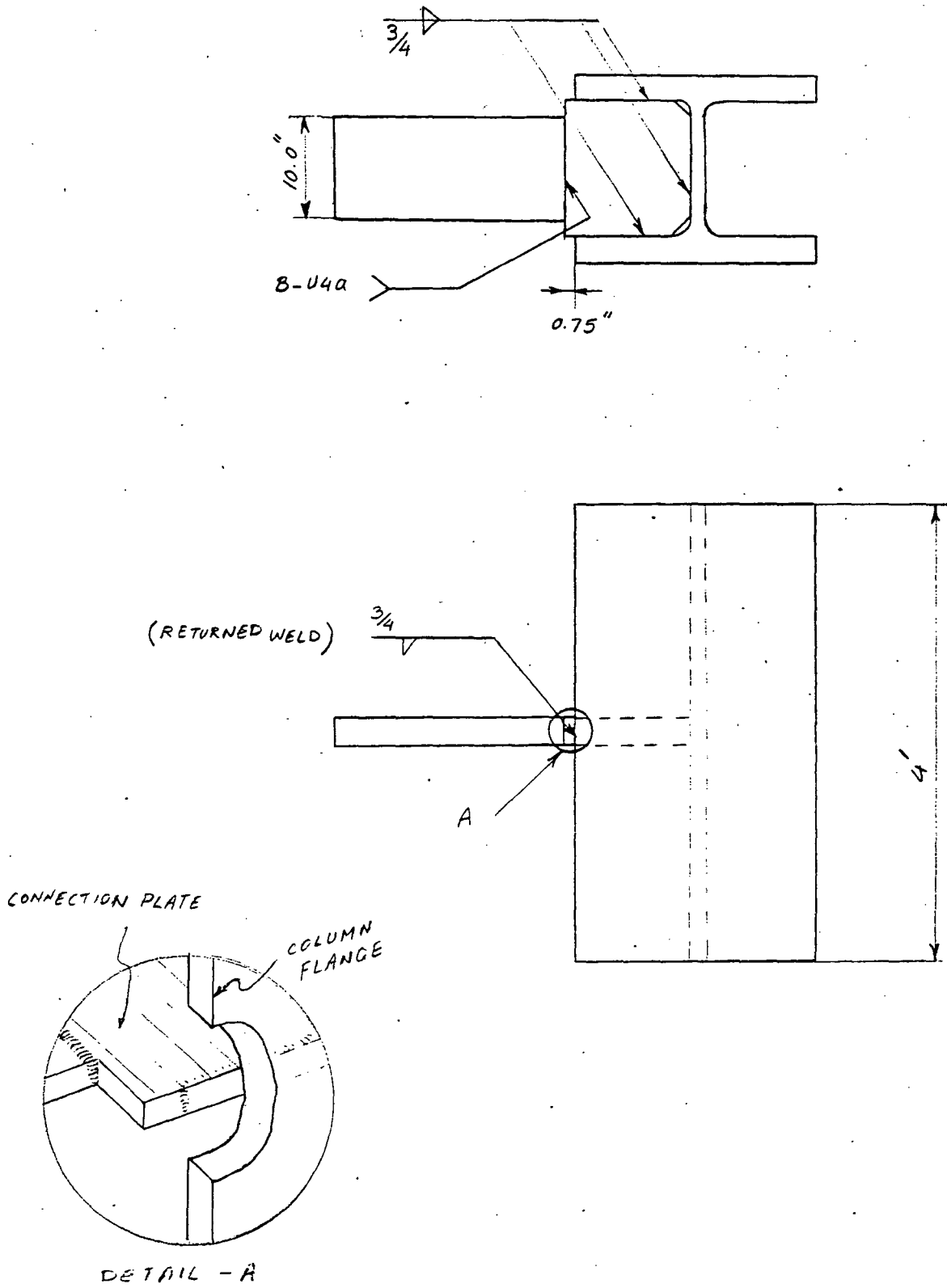


Figure 5: Connection Plate Weld Details

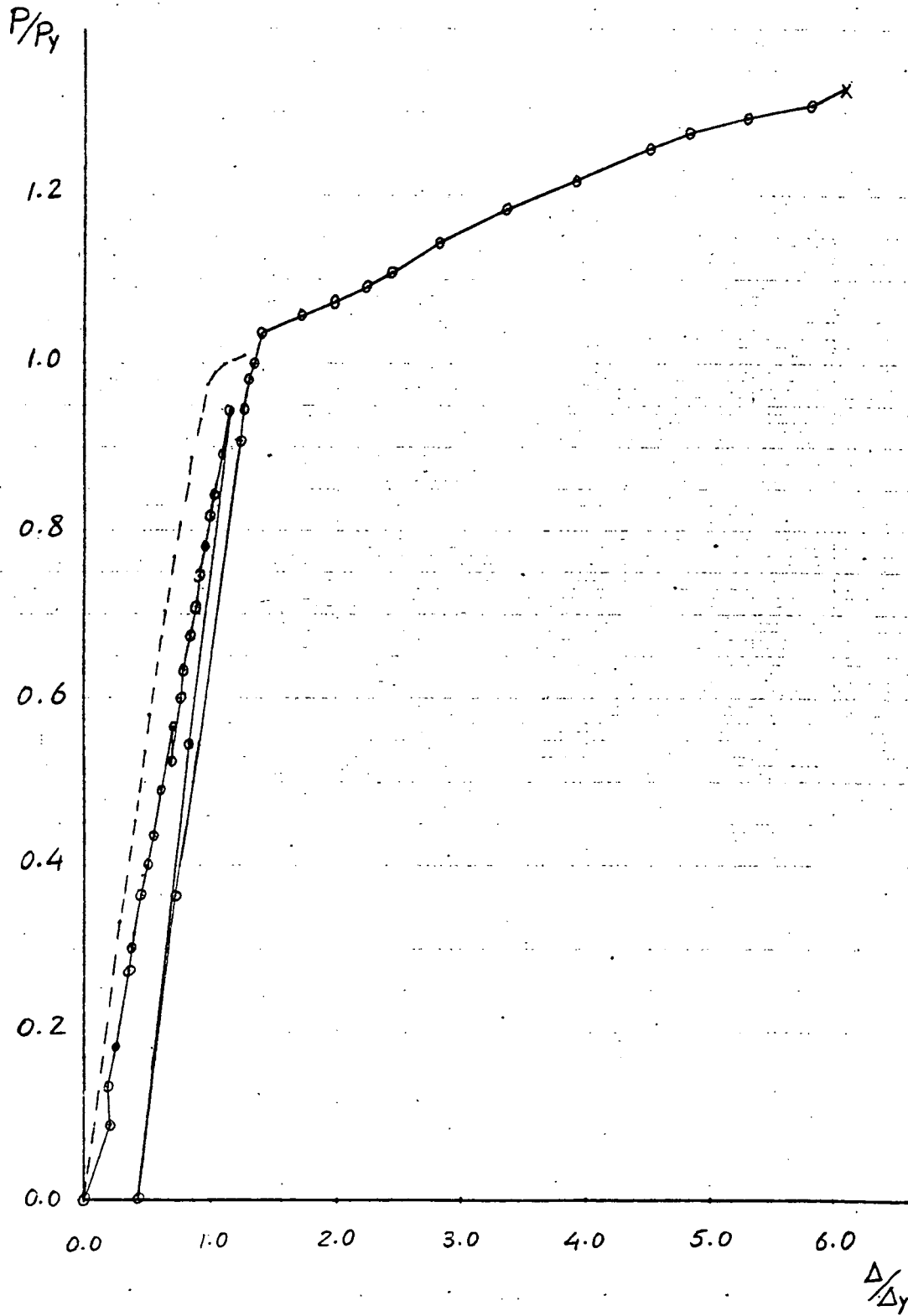


Figure 6: Load-Displacement Curve of Test 1

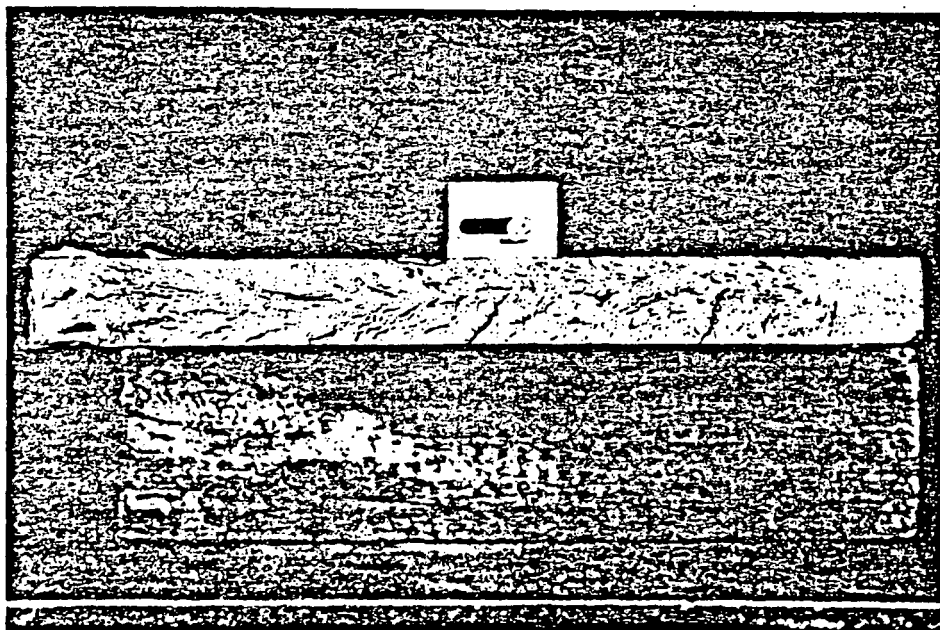


Figure 7: Front Views of Fractured Surface

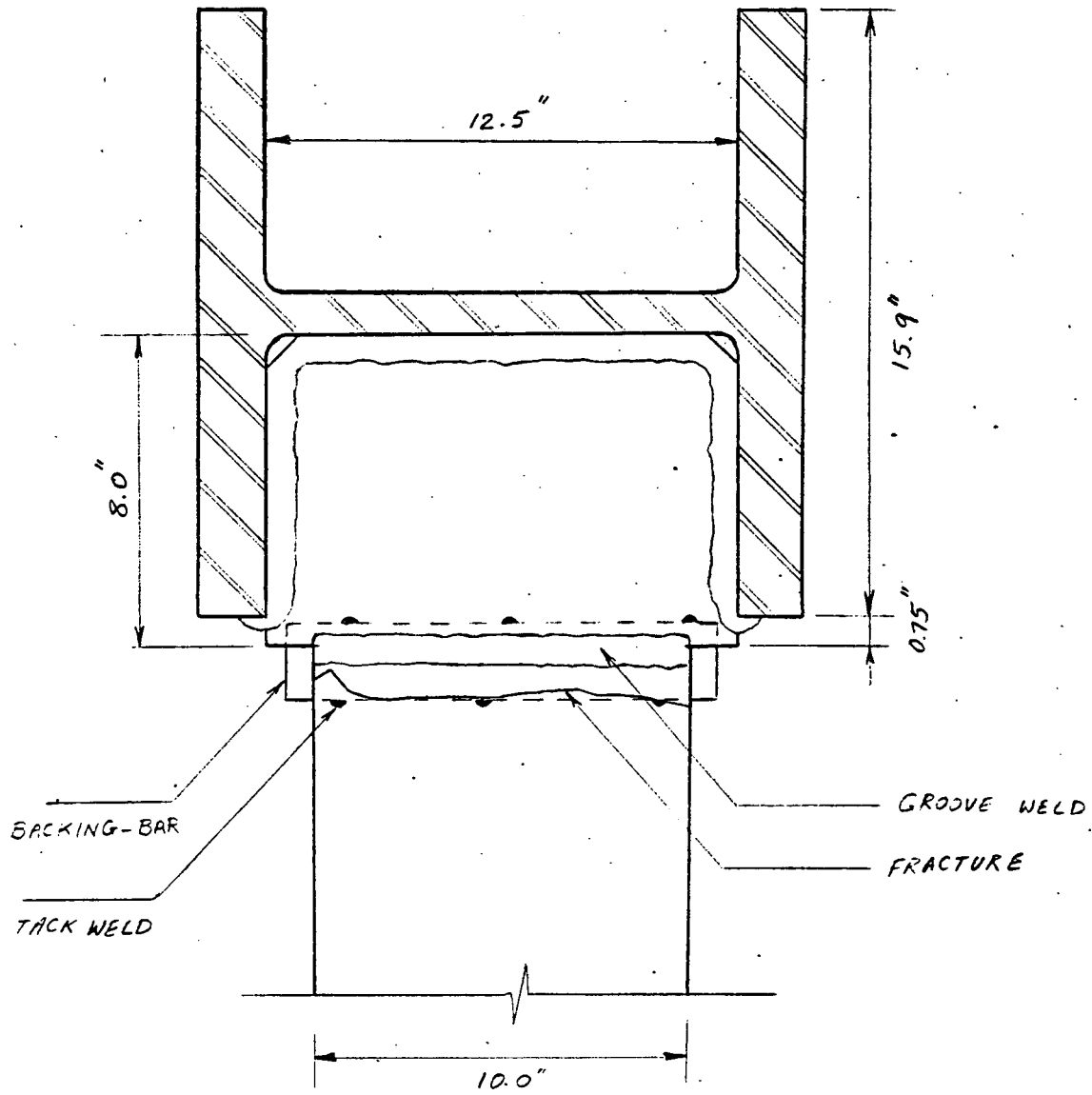


Figure 8: Location of Fracture

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