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STEEL BEAM-TO-COLUMN MOMENT CONNECTIONS

By Joseph S. Huang[1], A.M. ASCE and Wai F. Chen[2], M. ASCE

1. INTRODUCTION

One of the determining factors of economy in structural steel design
is the moment-resisting beam-to-column connections. The selection of connec-
tions is often based upon simplicity, duplication and ease of erection. The
designer should avoid complicated and costly fabrication. Welded connections
providing full continuity are commonly used in plastically designed structures.
In recent years, A325 and A490 high-strength bolts have become the most com-
monly used fasteners in field construction. Connections, which require a com-
bination of welding and bolting, are also used in plastically designed struc-
tures. They are considered to be very economical.

1.1 PREVIOUS RESEARCH

Previous investigations on moment-resisting beam-to-column connec-
tions conducted at Cambridge University, Cornell University, and Lehigh Univer-
sity were summarized and discussed in Ref. 4. The types of connections studied
are: fully welded connections, welded top plate and angle seat connections,
bolted top plate and angle seat connections, end plate connections, and T-
stub connections. In addition, the behavior of welded corner connections, bolted
lap splices in beams, and end plate type beam splices was discussed. The
connecting media for these specimens were welding, riveting, and bolting. Only
A325 high-strength bolts were used. The most important result of these tests
is that for all properly designed and detailed welded and bolted moment connec-
tions, the plastic moment of the adjoining member was reached and the connec-
tions were able to develop large plastic rotation capacity. There were no pre-
mature failures except those which could have been predicted and prevented.

Recently, a series of eight tests of full-size steel beam-to-column

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Pa.

connections was carried out at the University of California (8). The connections were subjected to cyclic loading simulating earthquake effects on a building frame. Among those connections tested were two fully welded connections, five flange-welded web-bolted connections, and one flange-welded connection. A325 bolts were used in fastening the web shear plates. Beam sections used were W18x50 and W24x76; column sections were W12x106. The connection specimens were made of ASTM A36 steel. All connections had horizontal stiffeners which were connected to the columns by groove welds. Results of this series of tests show that the hysteresis loops in all cases were stable in shape under repeated loading cycles. The failure of connections was due to either local buckling of beam flanges or weld fracture, and occurred only after many cycles of loading beyond yield.

1.2 STATEMENT OF THE PROBLEM

Currently, a test program consisting of twelve full-size beam-to-column connections is underway at Lehigh University under the guidance of the Welding Research Council (6). The types of connections being studied are: (1) flange-welded web-bolted connections; (2) stiffened seated beam connections; (3) only flange-welded connections; and (4) bolted moment plate connections. These connection specimens were made of ASTM A572 Gr. 55 steel, fastened with A325 and A490 bolts utilizing higher allowable shear stresses, namely 30 ksi for A325 and 40 ksi for A490, for bolts in bearing-type connections.

1.3 OBJECTIVE OF RESEARCH

The objective of this study is to investigate the performance of those connections that are subjected to symmetrical loading conditions. Primary attention is focused upon the strength, deformation capacity and over-all stiffness in the elastic range. The results of these tests will be utilized to formulate design procedures for safe, efficient, and economical beam-to-column connections.

2. T E S T   P R O G R A M

2.1 DESIGN AND FABRICATION OF SPECIMENS

The specimens were designed according to Sec. 2.8, Connections, of the AISC Specification (1). The connections were proportioned to resist the plastic moment of the beam section. 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 A325 and A490 high-strength bolts in bearing-type connections the design shear stresses used were equal to 1.7 times 30 ksi and 40 ksi, respectively, instead of 22 ksi and 32 ksi suggested in current Specification. The concept of this procedure will be discussed later.

2.1.1 Member Size and Beam Span

The connection specimens were chosen to have an appropriate combination of a beam section and a column section which represented the real interior beam-to-column connections in a multi-story frame. Three different sizes of specimens consisting of W14x74, W24x61, and W27x94 beams connected to W10x 60, W14x136, and W14x176 columns, respectively, were used in this test program.

All of the beam sections are plastic design sections which satisfy the requirements of Sec. 2.7, Minimum Thickness (Width-Thickness Ratios), of the AISC Specification. Both W24x61 and W27x94 shapes are the lightest in weight in each particular group as given in the Plastic Design Selection Table of the AISC Manual.

Another factor considered in selecting beam sizes is the way a wide-flange shape resists bending moment and shear force. It is well known that the flanges resist most of the bending moment, and the web almost entirely carries the shear force. The ratio of flange area to web area furnishes an index to the amount of moment carried by the web, which must be transferred to the flanges at the connections since the shear connections have negligible

moment-resisting capacity. The ratios of one flange area to web area, A_f/A_w, for W14x74, W24x61, and W27x94 sections are 1.39, 0.44, and 0.60, respectively. The behavior of these sections should be representative of a wide range of wide-flange sections.

It was desired to use column sections which did not need horizontal stiffeners. In each case the columns selected were the least column size based upon the AISC tension or compression flange criterion.

The specimens were proportioned in such a way that at the beam-to-column juncture the plastic moment and the factored shear capacity of single shear bolts in beam web would be reached concurrently. Beam span then was simply the ratio of moment to shear values.

All test specimens were fabricated from wide-flange shapes made of ASTM A572 Gr. 55 steel. A detailed report of material properties is given in Ref. 10.

2.1.2 Fasteners and Holes

ASTM A325 and A490 bolts were used to assemble the joints. In bearing-type connections, the allowable shear stresses used in design for A325 and A490 bolts were 30 ksi and 40 ksi, respectively. The use of higher allowable shear stresses reflects the logical design criterion which would result if an adequate factor of safety were applied against the shear strength of the fasteners. This design criterion is based upon the results of a study of A7 and A440 steel lap and butt joints fastened with A325 bolts, and A440 steel joints connected with A490 bolts (5). Tests have been subsequently carried out to substantiate the suggested design criterion, especially the use of A490 bolts in A440 and A514 steel joints (12,7).

Since both oversize holes and slotted holes are desirable to facilitate erection adjustments, and slotted holes may better facilitate the assumed distribution of shear and moment at the connections, experimental justification is required for beam-to-column connections assembled with high-strength bolts with enlarged

and with slotted holes. Previous research has indicated that oversize holes, sized according to bolt diameter, do not adversely affect the slip behavior of friction-type joints or cause undesirable bolt-tension losses (2). It was also observed that slotted holes did not affect the strength of bearing-type joints.

In the test program, 1-1/4 in. round holes were used in top and bottom moment plates fastened with 1 in. diameter A490 bolts and designed as a friction-type connection (Test C8 in Fig. 9). The use of 1/4 in. oversize holes is the maximum size permitted in the current Specification (9). Slotted holes were used in one-sided shear plates fastened with either A490-X bolts (C3 in Fig. 4) or A325-X bolts (C8 and C9). The remaining joints had round holes 1/16 in. larger than the nominal diameter of the bolt.

Both A325 and A490 bolts were installed by the turn-of-nut method. Washers were not used for A325 bolts. In bearing-type connections, A490 bolts had a hardened washer under the element (nut or bolt head) turned in tightening. In the friction-type connection (C8), a hardened washer was inserted under both the head and nut. Nut rotation from snug tight condition was 1/2 turn as required by the Specification (9). All bolts were calibrated and installed in the Fritz Laboratory.

2.1.3 Welds

The connection specimens were welded according to the AWS Building Code (3). The welding process used for groove welds was the innershield procedure; the electrodes were E70TG (flux cored arc welding with no auxiliary gas shielding). The type of filler metal for beam flange groove welds in the flat position and for moment plate groove welds in the horizontal position was NR-311; NR-202 was used for beam web groove welds in the vertical-up position. The electrodes for fillet welds were E7028. In determining the size of fillet weld, the allowable shear stress on the effective throat was 21 ksi.

Nondestructive testing methods were employed to inspect the welds before testing of the specimens. Groove welds were inspected by ultrasonic

testing and fillet welds by magnetic particle. Results of weld inspection
were evaluated according to the AWS Code. Those rejected welds were repaired
and subsequently inspected prior to testing.

2.2 DESCRIPTION OF SPECIMENS

The test program, designated as C-series, is summarized in Table 1.
Detailed descriptions of each test are given as follows.

The joint detail of Test C1 is shown in Fig. 1. Beam flanges were
directly welded to the column flanges providing for plastic moment capacity.
A one-sided shear plate fastened with three 1 in. diameter A490-X bolts was
used to resist vertical shear. The fillet weld connecting the shear plate to
the column flange was sized for vertical shear only; the moment due to the
eccentricity of the applied load was neglected. Horizontal stiffeners were
designed according to Sec. 1.15.5 of the AISC Specification. Since the con-
nection panel zone was under symmetrical loads, a clearance of $\frac{1}{2}$ in. was
provided between horizontal stiffeners and column web. The size of fillet
welds for horizontal stiffeners was determined by computing the force taken
by the stiffeners when plastic moment was attained at the beam-to-column juncture.

Figure 2 shows Test C10 which is a fully-welded connection and was
used as a control test. Beam flanges and beam web were connected to the
column flanges by groove welds. An erection plate was tack welded to the col-
umn flange, and was used as the backing strip for the beam web groove weld.

Test C2 is shown in Fig. 3. Its connection type is similar to Test
C1, the only difference being that horizontal stiffeners are not required.

Test C3 is identical with Test C2 except that the one-sided shear
plate of Test C3, shown in Fig. 4, has slotted holes. The dimensions of
these slots conform to provisions in the current Specification (9). A con-
tinuous bar 5/16 inch in thickness and having a width equal to the length of
the slot was attached on the side of the slotted shear plate. (The addition
of continuous bars for single shear connections was approved by the Research

Council on Riveted and Bolted Structural Joints at its annual meeting on May 12, 1971).

Figure 5 shows the joint detail of Test C4. Moment capacity was provided through direct groove welding of the beam flanges to the column flanges. Vertical shear was resisted by a two-plate welded stiffener seat which was designed according to Table VIII of the AISC Manual. The strength of this connection should be greater than that of Test C5 shown in Fig. 6.

In the case of Test C5, the beam was connected to the column by groove welds only. It had neither an erection seat nor an erection clip. The purpose was to determine the actual capacity of the beam flange groove welds.

Figure 7 shows bolted top and bottom moment plate connection Test C6. The plastic moment was carried by flange plates which were fastened with 1 in. diameter A490-X bolts. The design procedure follows the example given on page 4-92 of the AISC Manual. The bracket stiffeners were designed with the aid of Table VIII.

Test C7 is shown in Fig. 8. The vertical shear is resisted by a one-sided shear plate connected to the beam web by three 1 in. diameter A490-X bolts. Tests C1, C6, and C7 were designed for the same amount of moment and shear, and, therefore, their behavior should be comparable.

Test C8 in Fig. 9 was designed as a friction-type connection having oversize holes in moment plates. The use of 1-1/4 in. round holes for 1 in. diameter A490-F bolts is permitted by the Specification (9). There is no reduction in slip resistance of the joint. The one-sided shear plates had slotted holes and were designed as bearing-type connections. A continuous bar was also attached on the side of the shear plate as shown in Fig. 9.

Test C9 (Fig. 10) is similar to Test C8. For the purpose of comparison, the moment plates of Test C9 were designed as bearing-type connections having round holes 1/16 in. in excess of the nominal bolt diameter.

For the purpose of rating the performance of those previously de-scribed connections, three fully-welded connections are included in this test program. Test C10 is shown in Fig. 2; Tests C11 and C12 are shown in Figs. 11 and 12, respectively.

2.3 TEST SETUP

The test setup is shown in Fig. 13. The axial load in the column was applied by a 5,000,000 pound-capacity hydraulic universal testing machine. The crosshead of the testing machine is shown. The beams were supported by two pedestals resting on the floor. Rollers were used to simulate simply supported end conditions. Because the combination of the short span of the beam and the size of shapes resulted in a compact setup, no lateral bracing was needed to provide stability.

3. TEST RESULTS

Eight tests have been completed to date. The experimental results and reference values are given in Table 2. The strength of connections is indicated by the maximum load to predicted plastic limit load ratio P_m/P_p. The deformation capacity is measured by the ratio of total deflection to the predicted deflection at plastic limit load Δ_m/Δ_p which is defined as the ductility factor μ. Table 3 (on page 17) summarizes the descriptions of failure of connections.

The load-deflection curves of connections consisting of W27x94 beam and W14x176 column are indicated in Fig. 14. Both C2 and C3 showed adequate stiffness under working load. Slip occurred above the working load and the A490 bolts eventually went into bearing against the sides of the holes. In addition, A490 bolts were able to deform to permit distribution of forces at maximum load. The ductility factors of C2 and C3 are 9.7 and 15.4, respec-tively; the deformation capacities of C2 and C3 are adequate for design. The minimum required ductility factor has been recommended to be 4. The joint of C2 at plastic limit load is shown in Fig. 15. The failure of C2 was due to the

tearing of the column web along the web-to-flange juncture. Figure 16 shows an over-all view of C2 after testing.

The unloading of C3 was initiated by local buckling of the compression flange of the beam. The panel zone and joints of C3 after testing are shown in Fig. 17. The testing of C3 was terminated when fracture occurred at the heat-affected zone of the weld at the tension flange (Fig. 18).

C4 also exhibited sufficient stiffness under working load. The unloading was due to the buckling of the beam web above the stiffened seat. It finally failed by excessive buckling of the beam web and fracture of the beam web which initiated at the cope hole as shown in Fig. 19. An over-all view after testing is shown in Fig. 20.

C5 attained 51 per cent of its predicted plastic limit load based upon whole section, and showed substantial deformation. The unloading was due to a beam web fracture which initiated at the cope hole. Testing was terminated when the tearing of the beam web became excessive (Fig. 21) and the beam web buckled (Fig. 22).

C12 is a fully-welded connection. The cause for unloading was buck-ling of the column web in the compression region. Testing was concluded due to a combination of excessive column web deformation in the compression region and fracture at the tension flange groove weld (Fig. 23) and along the beam web groove weld which occurred simultaneously. Fracture occurred by ripping out of column flange material around weld, and not fracture of actual weld itself. Figure 24 shows an over-all view of C12 after testing. A detailed report of Test C12 is given in Ref. 11.

Tests C1 and C10 consist of W14x74 beam and W10x60 column. Their behavior is shown by the load-deflection curves in Fig. 25. The deviation of C1 from C10, that occurred above the working load, is due to slip of the joints. The panel zone of C1 at plastic limit load is shown in Fig. 26. C1 failed when fracture occurred at the fillet weld connecting the tension stiffener to

the column flange (Fig. 27). Connection C10 after testing is shown in Fig. 28. The compression horizontal stiffeners behaved satisfactorily; they buckled after the attainment of the predicted plastic limit load. The failure of C10 was also due to a fracture occurring at the fillet weld of the tension stiffener (Fig. 29).

The load-deflection curve of C11 is shown in Fig. 30. This connection was designed for a shear capacity at factored load of 52.5% V_p which simulates the loading condition in a real building. The maximum load is 25 per cent greater than the predicted plastic limit load. This substantial increase in load-carrying capacity is attributed to the forming of plastic hinges at the joints (Fig. 31), and therefore, strain hardening sets in quickly in localized zones. The testing of C11 was terminated when fracture occurred at the heat-affected zone of the groove weld at the tension flange (Fig. 32).

4. CONCLUSIONS

On the basis of the results of these completed tests, the following conclusions, which are valuable to the fabricated structural steel industry, have been reached.

1. Flange-welded web-bolted connections have fulfilled design requirements, showing sufficient stiffness in the elastic range under working load, attaining plastic limit load, and exhibiting a very large amount of deformation capacity near maximum load. This type of connection may be used in rigid frame construction (AISC Type 1).

2. A490 high-strength bolts performed satisfactorily. It is appropriate to design the one-sided web shear plate as a bearing-type joint using an allowable shear stress of 40 ksi.

3. The fillet welds connecting the shear plate to the column flange may be sized for vertical shear only; the moment due to the eccentricity of the applied load may be neglected.

4. Slotted holes may be used in one-sided shear plates that are designed as

-10-

bearing-type joints.

5. Fillet welds may be used in lieu of groove welds in connecting horizontal stiffeners to column flanges.

6. Welds approved by ultrasonic inspection were satisfactory. A careful weld inspection during fabrication was necessary to ensure the adequate performance of connections.

5. ACKNOWLEDGMENTS

TABLE 1   TEST PROGRAM OF C-SERIES CONNECTIONS

| Test (1) | Beam Size (2) | Column Size (3) | Factored Load | | Beam Span L (6) |
			Moment (4)	Shear (5)	
C1	W14x74	W10x60	M_p=6930K-in Beam flange groove weld	V=161K(88.5%V_p) Shear plate, 3-1"ϕ A490-Xin 1-1/16" round holes	3'-7"
C2	W27x94	W14x176	M_p=15290K-in Beam flange groove weld	V=374K(94.7%V_p) Shear plate, 7-1"ϕA490-X in 1-1/16" rnd. holes	3'-5"
C3	W27x94	W14x176	DO	DO Shear plate has slotted holes	3'-5"
C4	W27x94	W14x176	DO	V=374K(94.7%V_p) Two-plate welded stiffened seat	3'-5"
C5	W27x94	W14x176	Beam flange groove weld	Beam flange groove weld	3'-5"
C6	W14x74	W10x60	M_p=6930K-in Moment plates 8-1"ϕA490-Xin 1-1/16" round holes	V=161K(88.5%V_p) Stiffener plate 1"x5"x11"	3'-7"
C7	W14x74	W10x60	DO	V=161K(88.5%V_p) Shear plate, 3-1"ϕA490-X in 1-1/16" rnd. holes	3'-7"
C8	W24x61	W14x136	M_p=8360K-in Moment plates 14-1"ϕA490-Fin 1-1/4" round holes	V=157.5K(52.5%V_p) Shear plate, 7-3/4"ϕA325-X in slotted holes	4'-5"
C9	W24x61	W14x136	M_p=8360 K-in Moment plates 6-1"ϕA490-Xin 1-1/16" round holes	DO	4'-5"
C10	W14x74	W10x60	M_p=6930 K-in Beam flange groove weld	V=161K(88.5%V_p) Beam web groove weld	3'-7"
C11	W24x61	W14x136	M_p=8360K-in Beam flange groove weld	V=157.5K(52.5%V_p) Beam web groove weld	4'-5"
C12	W27x94	W14x176	M_p=15290K-in Beam flange groove weld	V=374K(94.7%V_p) Beam web groove weld	3'-5"

a.   All specimens were made of ASTM A572 Grade 55 steel.
b. h_a for all specimens is 2'-6". h_b is 1'-3".
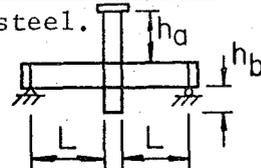
TABLE 2 TEST RESULTS

Test (1)	Experimental		Reference				$\dfrac{P_m}{P_p}$	$\dfrac{P_m}{P_{ps}}$	$\dfrac{P_m}{P_{pr}}$	$\dfrac{\Delta_m}{\Delta_p}$
	P_m (2)	Δ_m (3)	P_p (4)	P_{ps} (5)	P_{pr} (6)	Δ_p (7)	(8)	(9)	(10)	(11)
C1	357.5	1.50	322	286	271	0.361	1.11	1.25	1.32	4.2
C2	826	2.67	748	590	522	0.276	1.10	1.40	1.58	9.7
C3	818	4.26	748	590	522	0.276	1.09	1.39	1.57	15.4
C4	660	0.57	748	590	522	0.276	0.88	1.12	1.26	2.1
C5	380	1.72	748	370	522	---	0.51	1.03	0.73	--
C10	364.5	1.57	322	286	271	0.361	1.13	1.27	1.34	4.4
C11	392.5	2.05	315	266	199	0.341	1.25	1.47	1.97	6.0
C12	838	3.63	748	590	522	0.276	1.12	1.42	1.61	13.2

a. All loads (P) listed are column loads in kips; all deflections (Δ) are in inches.
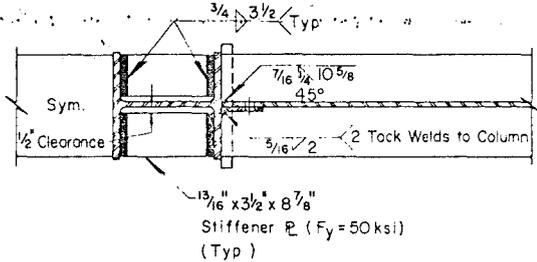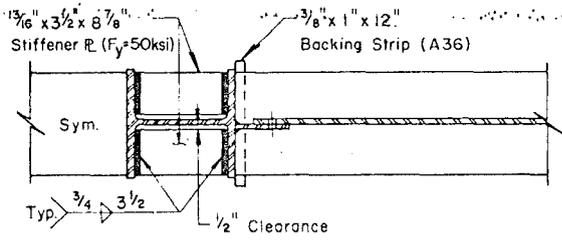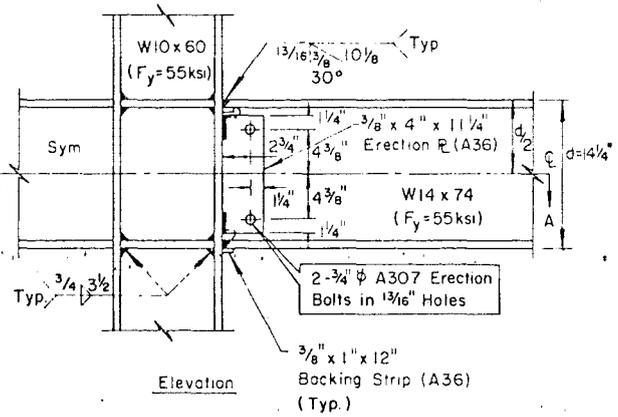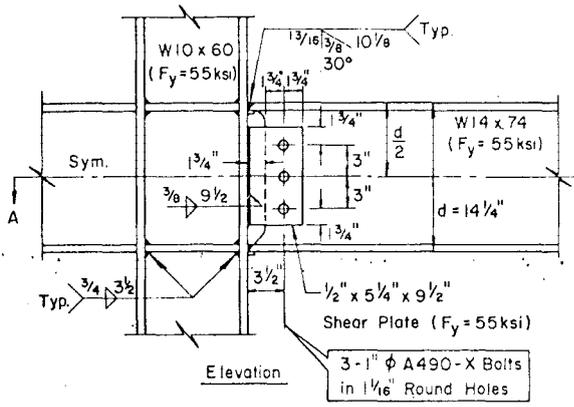
Fig. 1   Test C1

Fig. 2   Test C10
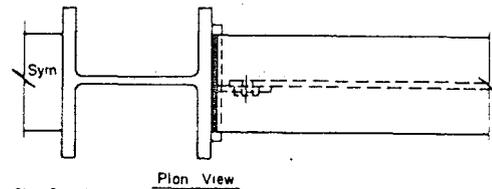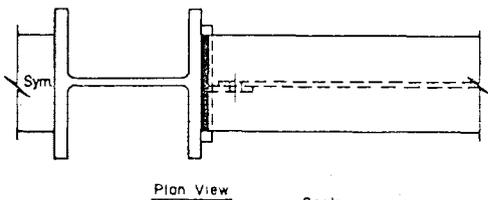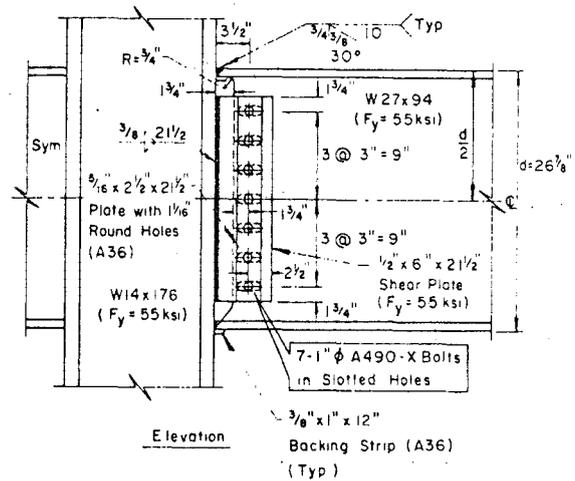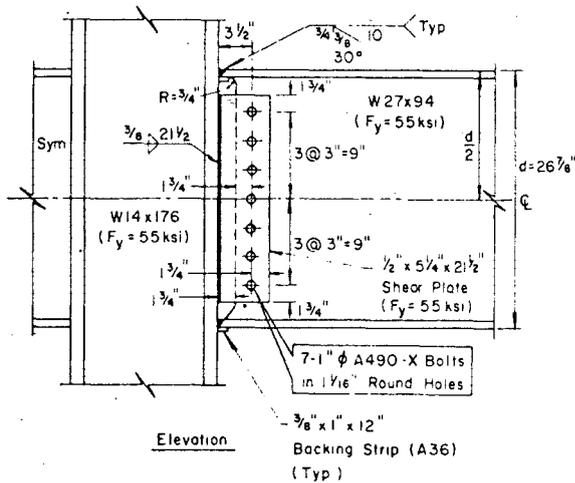
Fig. 3 . Test C2

Fig. 4   Test C3
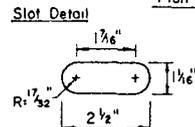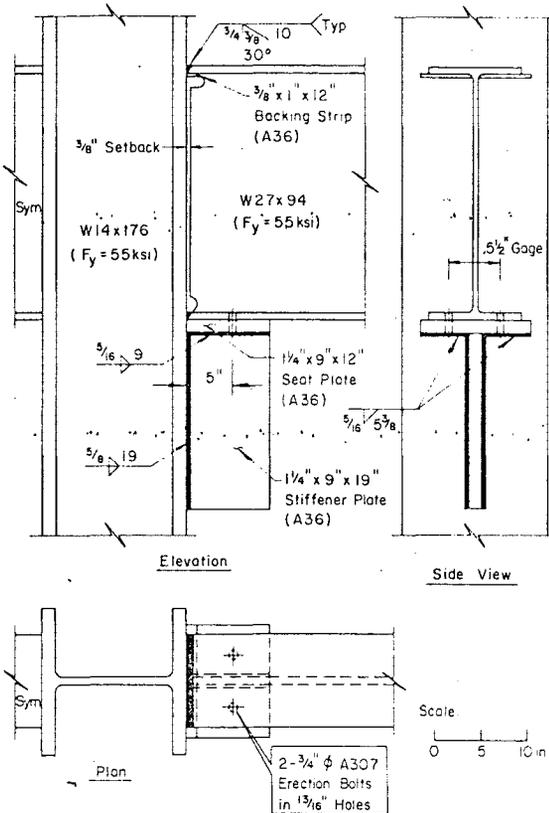
Fig. 5   Test C4
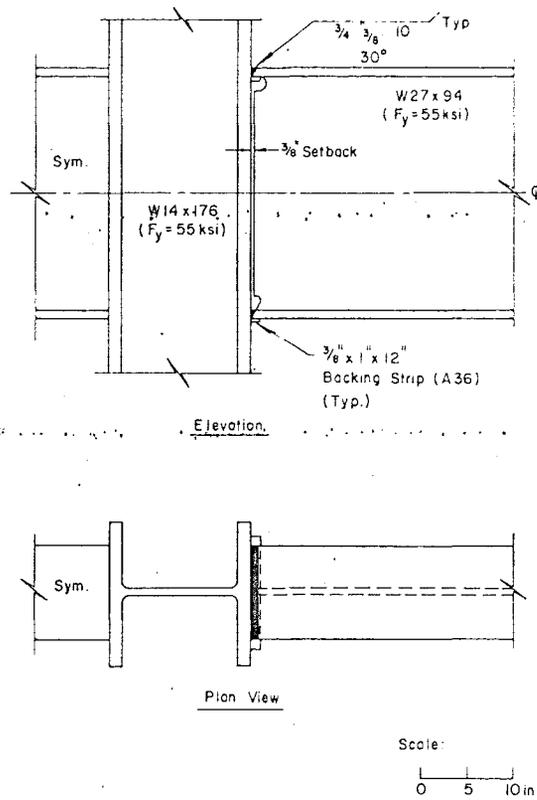
Fig. 6   Test C5
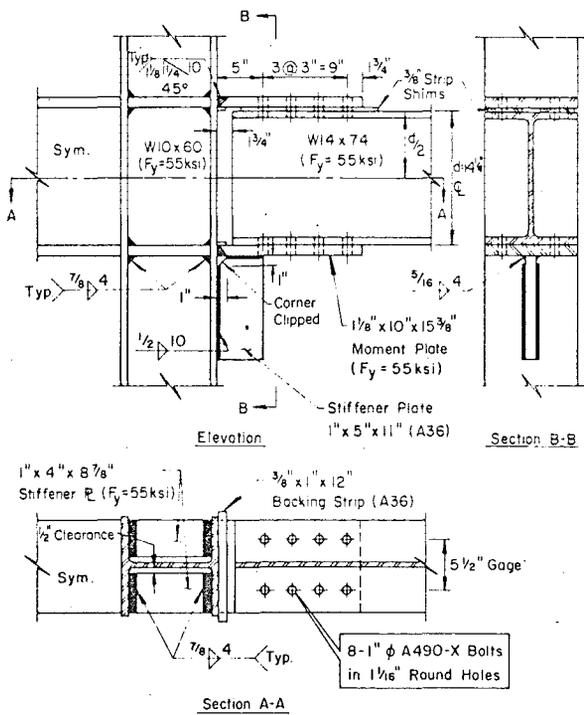
Fig. 7   Test C6
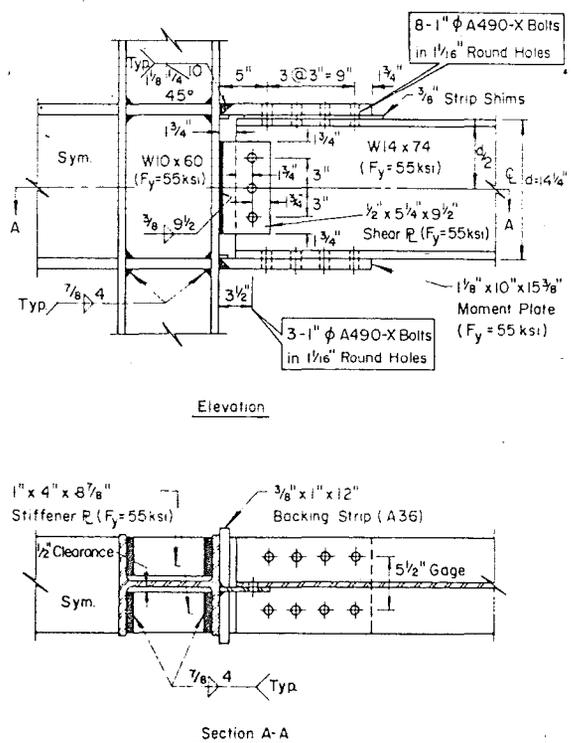
Fig. 8   Test C7
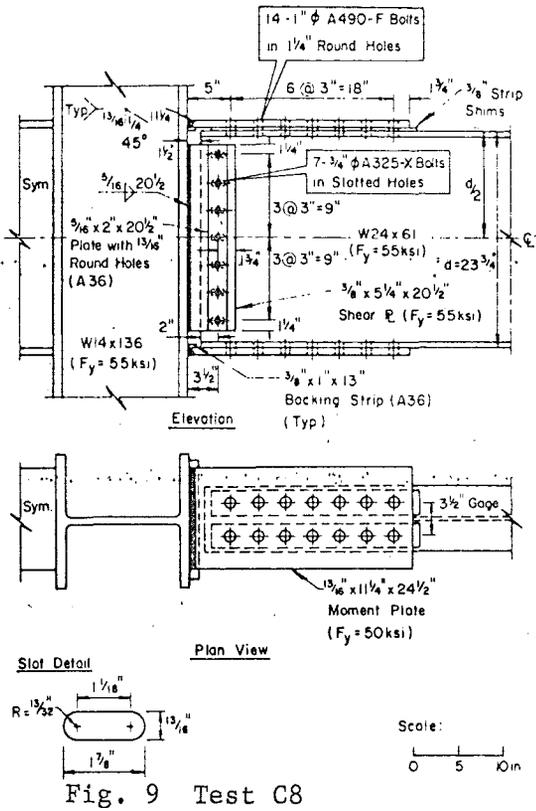
Fig. 9   Test C8
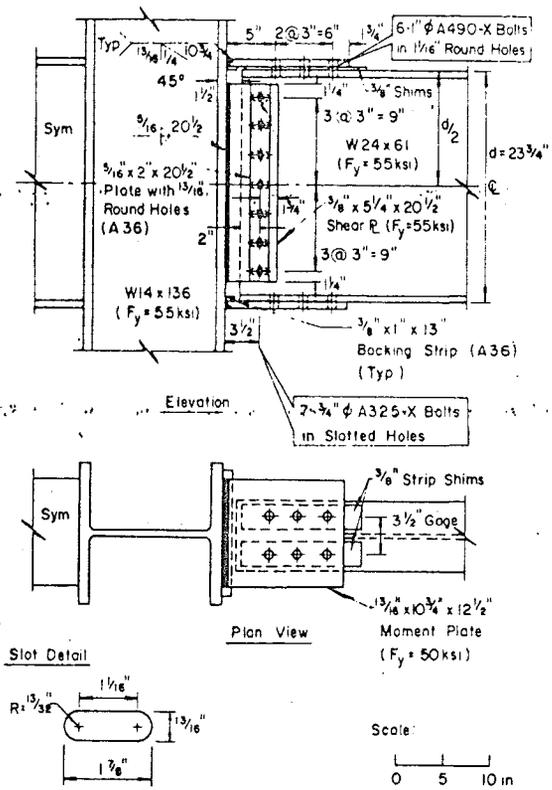
Fig. 10   Test C9
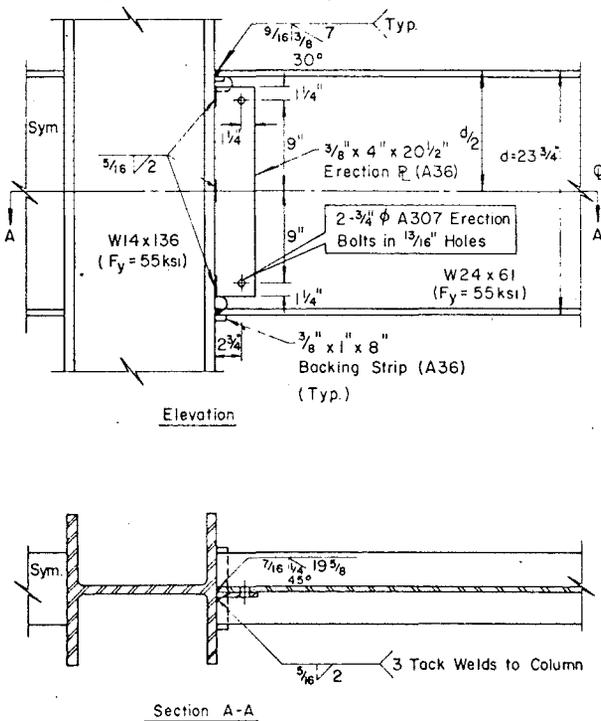
Fig. 11   Test C11

Fig. 12   Test C12

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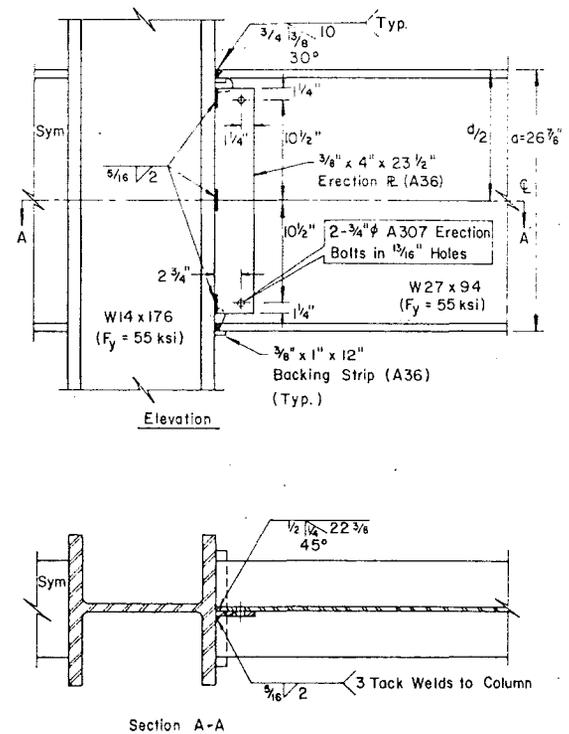
TABLE 3   DESCRIPTION OF FAILURE

Test | Description of Failure

C1   Crack at the fillet weld of the tension stiffener.

C2   Tearing of column web along web-to-flange juncture.

C3   Fracture occurring at the heat-affected zone of the tension

      flange.groove weld.

C4   Buckling of beam web and fracture of beam web initiated at the cope

      hole.

C5   Fracture of beam web initiated at the cope hole.

C10   Crack at the fillet weld of the tension stiffener.

C11   Crack at the heat-affected zone of the tension flange groove weld.

C12   Fracture at tension flange groove weld; excessive column web.

      deformation in the compression region.
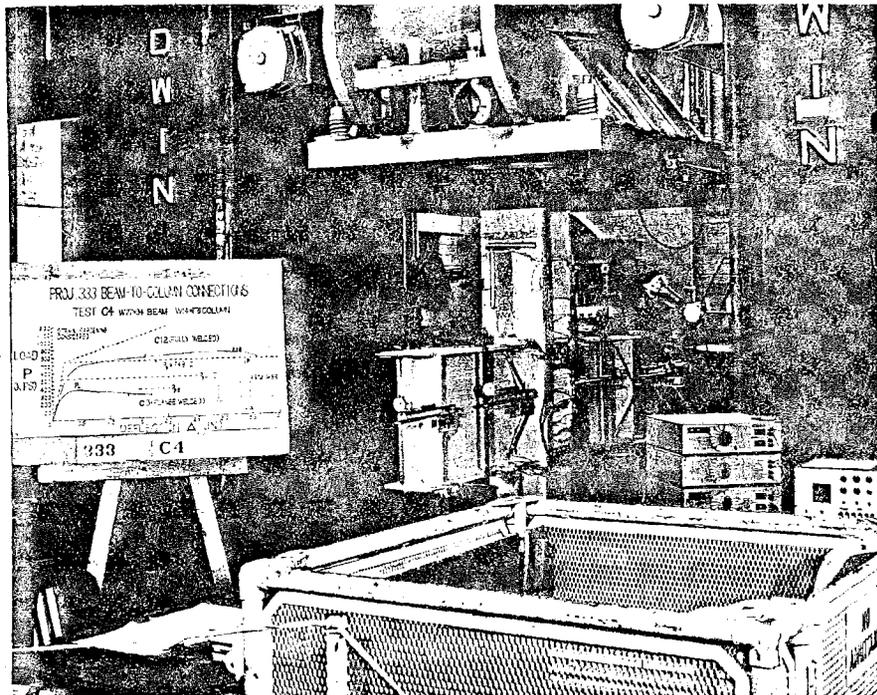
Fig. 13   Test Setup
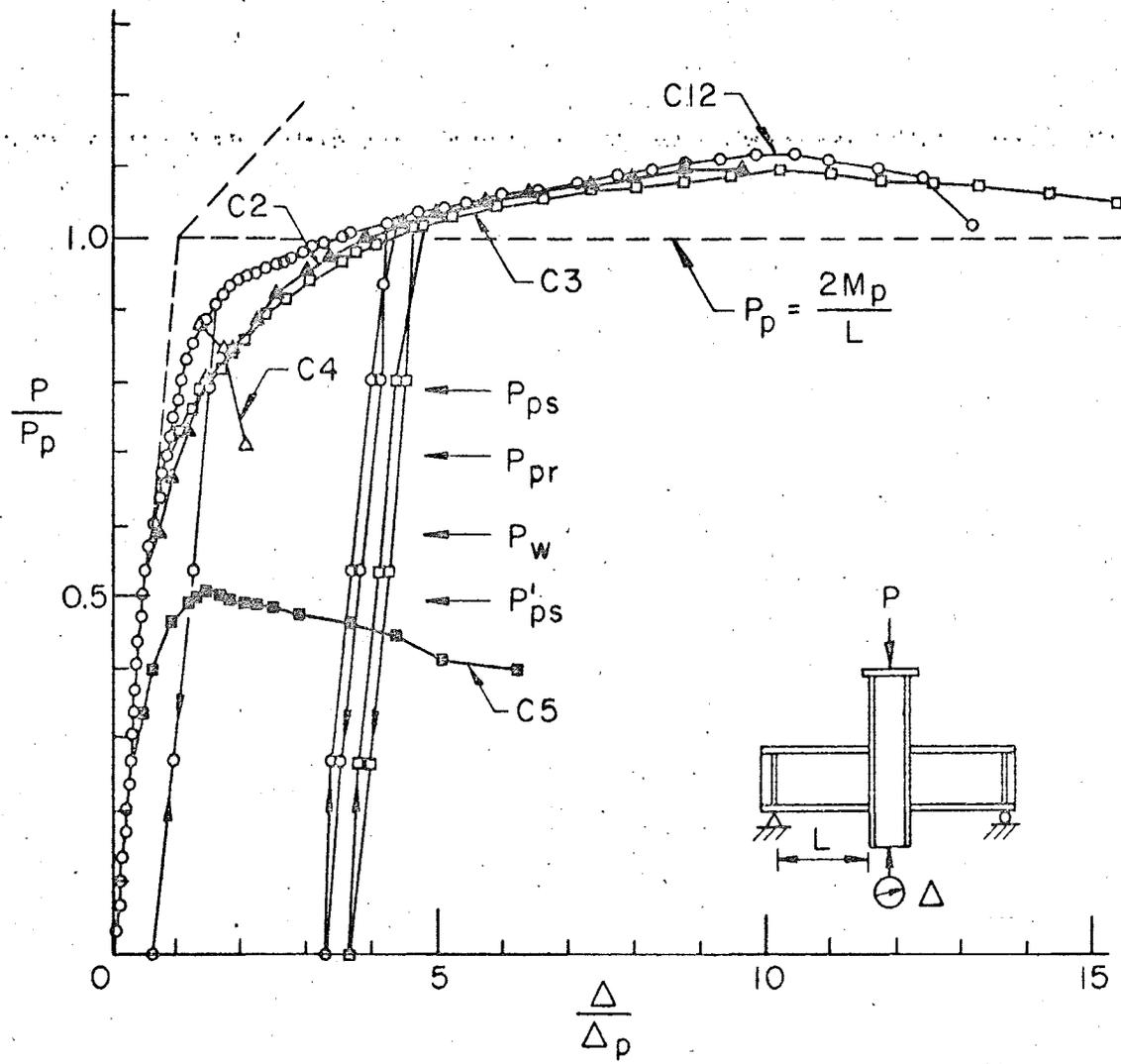
Fig. 14   Load-Deflection Curves of Connections C2, C3, C4, C5 and C12
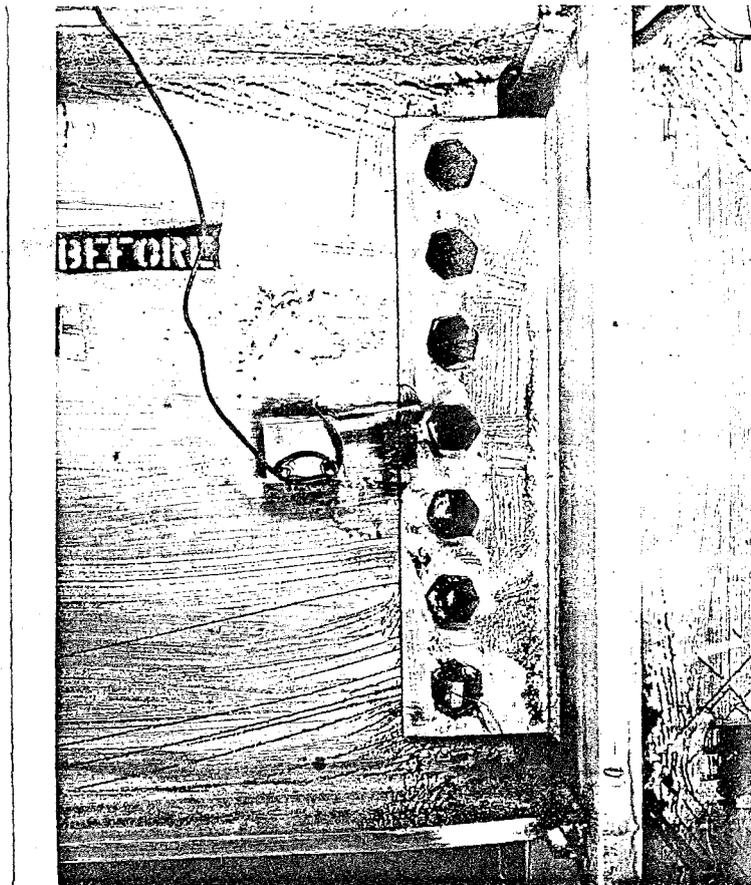
Fig. 15   Joint of C2 at Plastic Limit Load
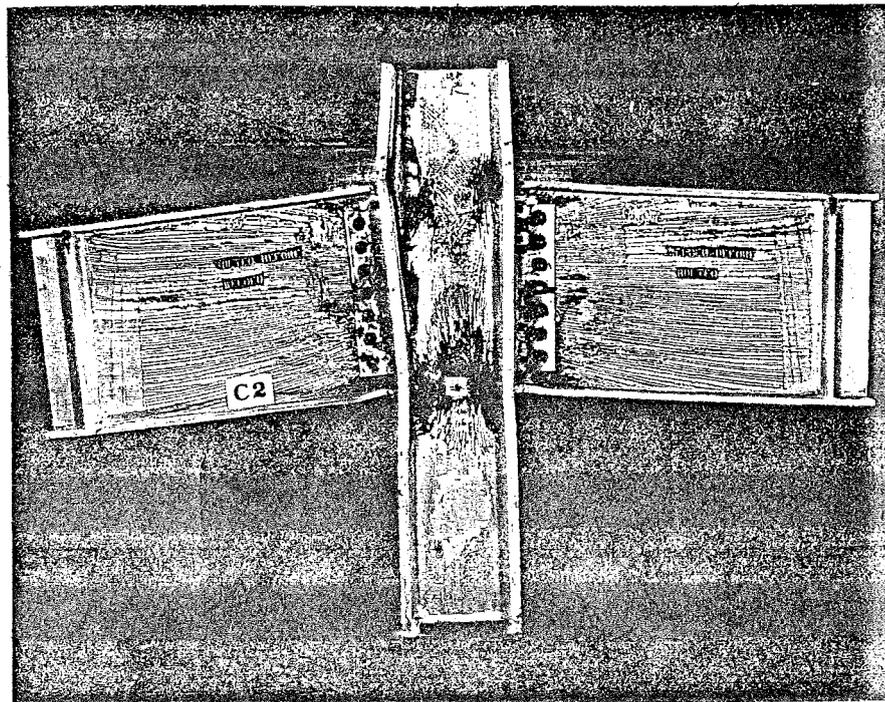
Fig. 16   Over-All View of C2 After Testing (Left-hand Side was Bolted Before
Welded; Right-hand Side was Welded Before Bolted)
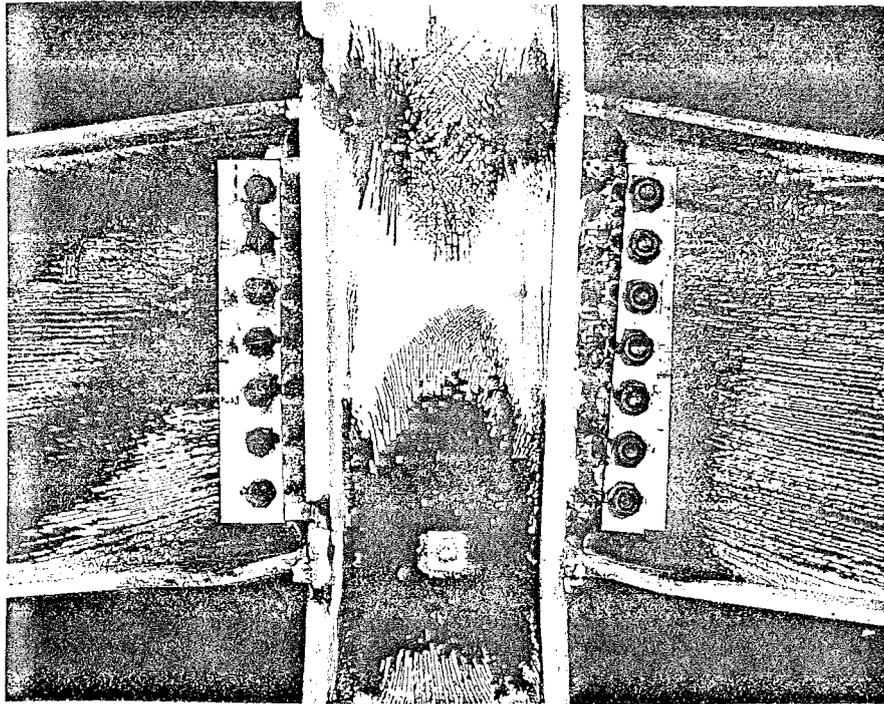
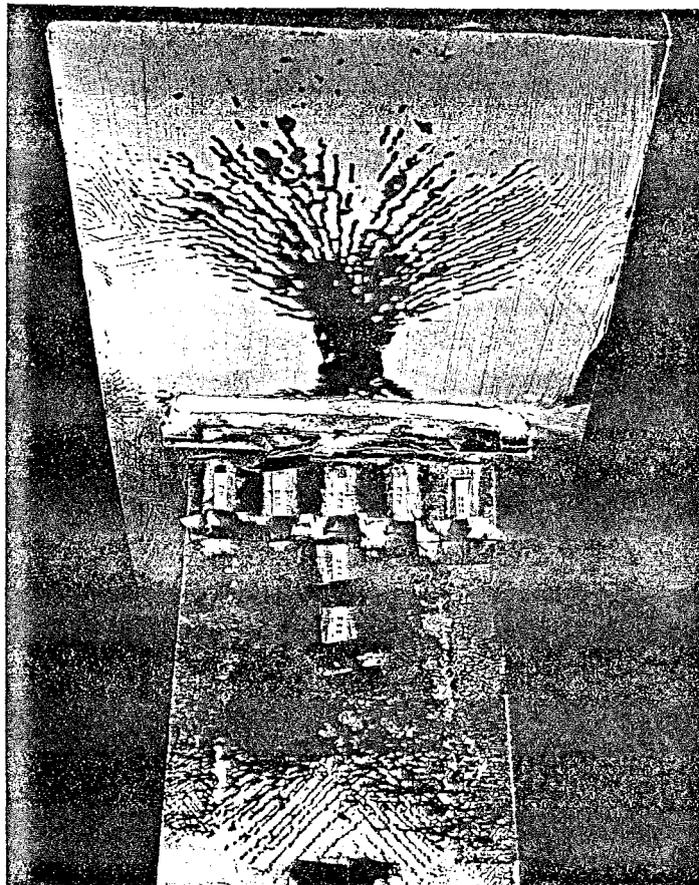Fig. 17   Panel Zone and Joints of C3 After Testing

Fig. 18   Fracture at the Heat-Affected Zone of the Groove Weld at the Tension
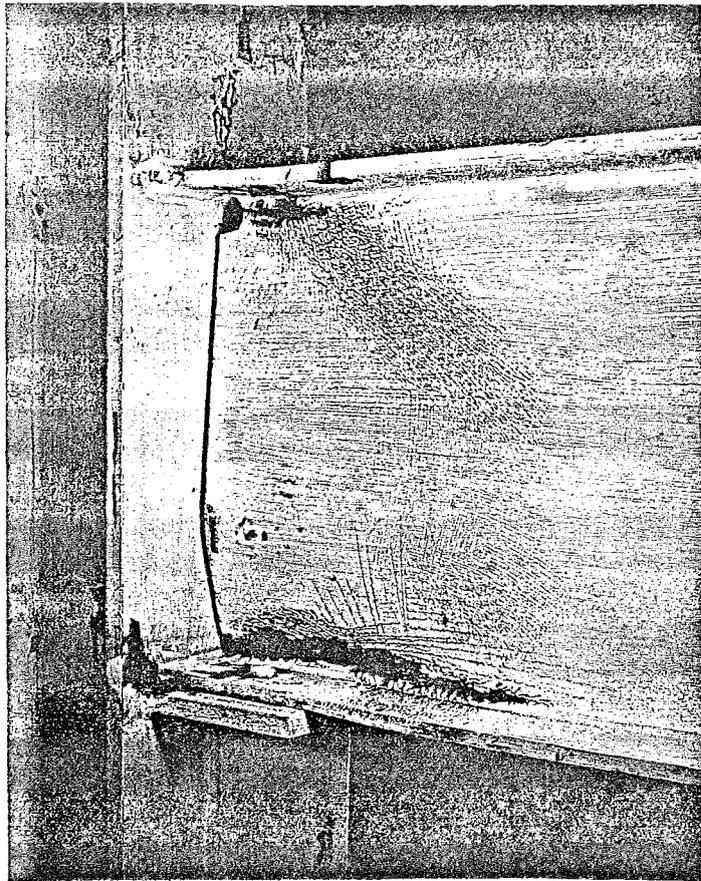Flange of C3

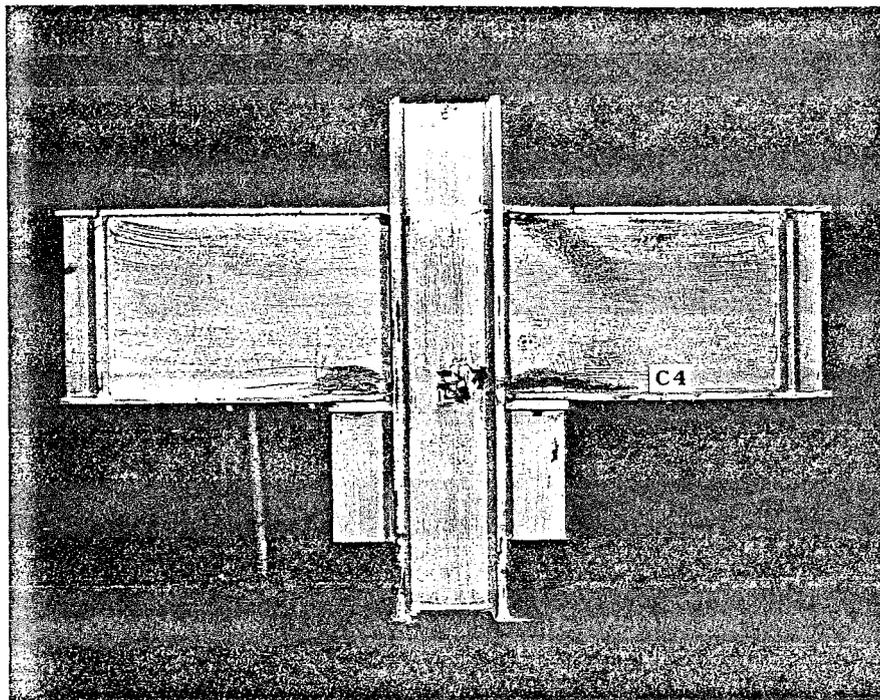Fig. 19 Buckling of Beam Web and Tearing of Beam Web at Cope Hole of C4

Fig. 20 Over-All View of C4 After Testing
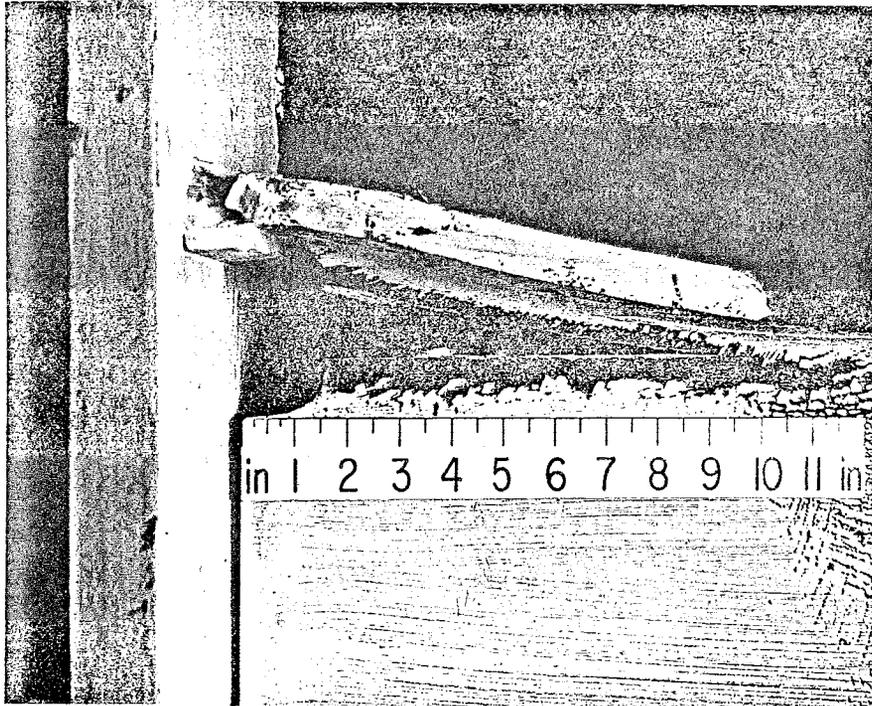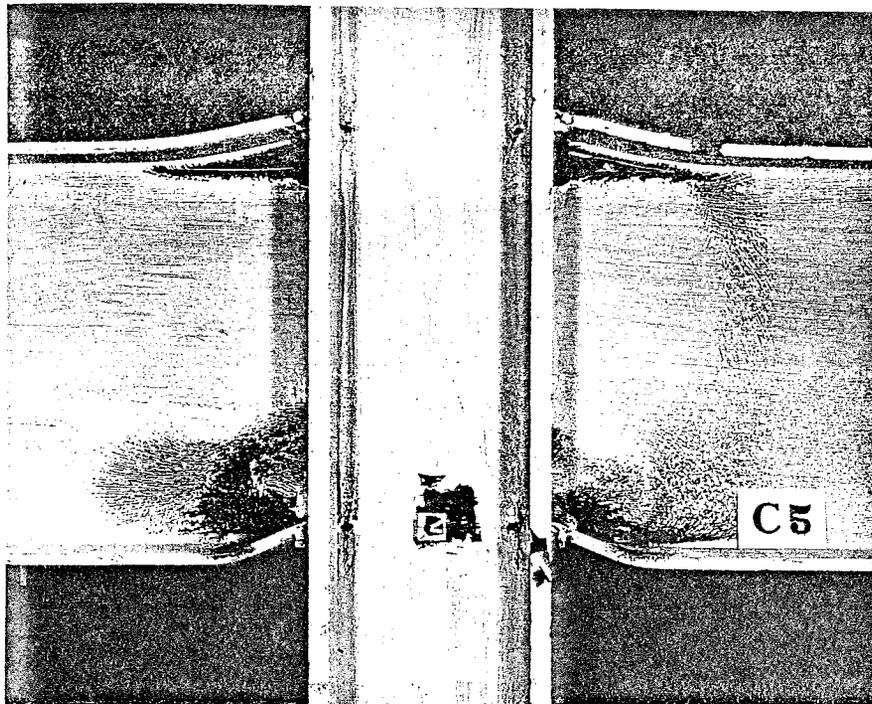
Fig. 21 Tearing of Beam Web at Cope Hole of C5

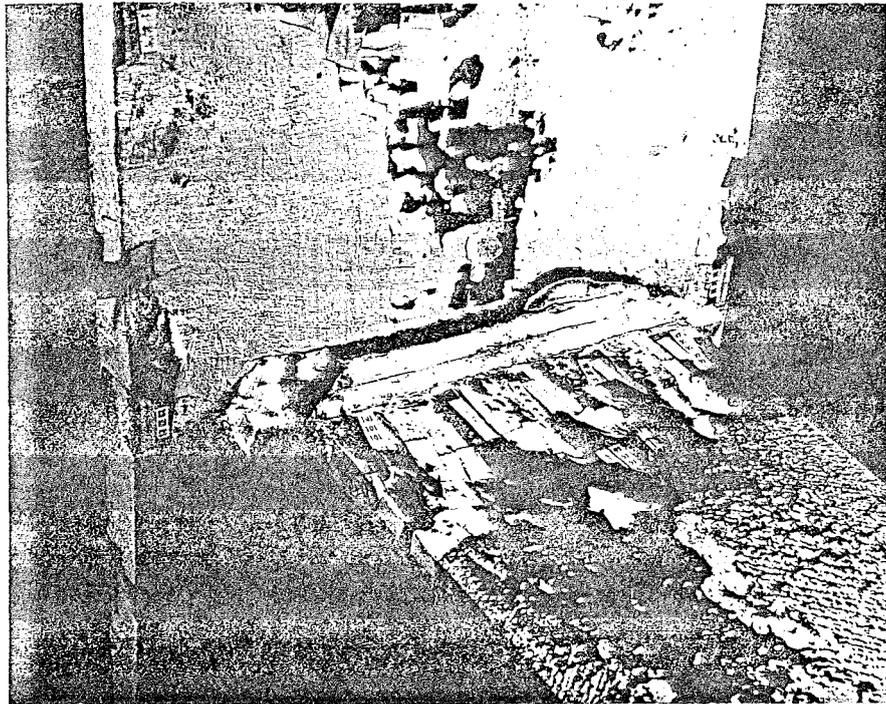Fig. 22 C5 After Testing

Fig. 23 Fracture at Tension Flange Groove Weld of C12
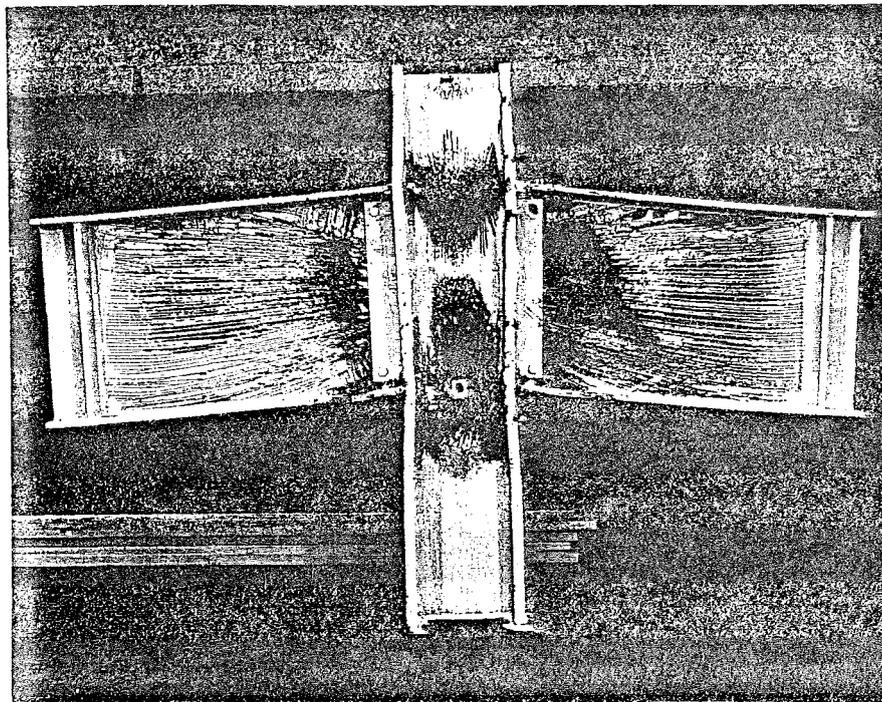

Fig. 24 Over-All View of C12 After Testing
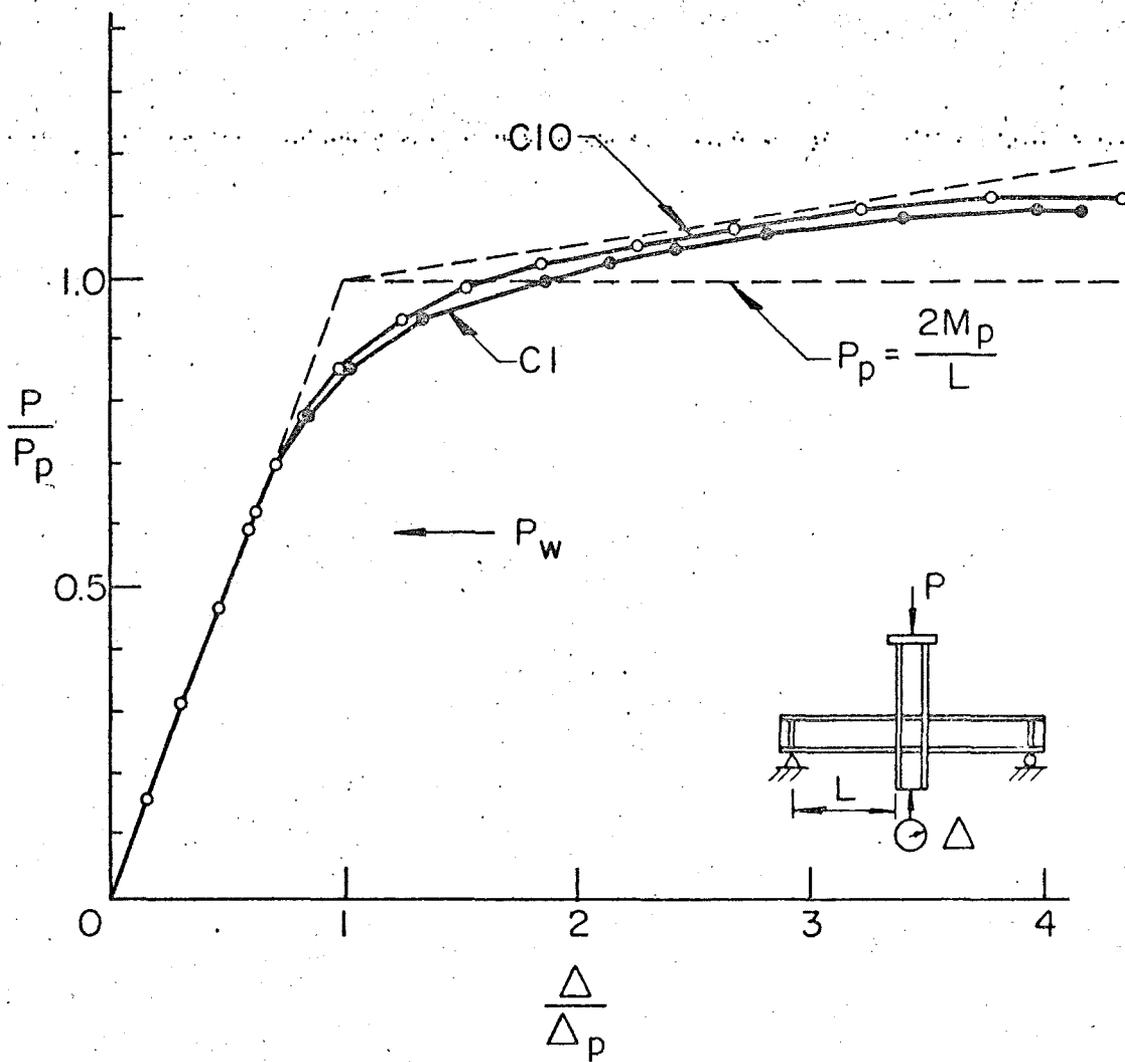
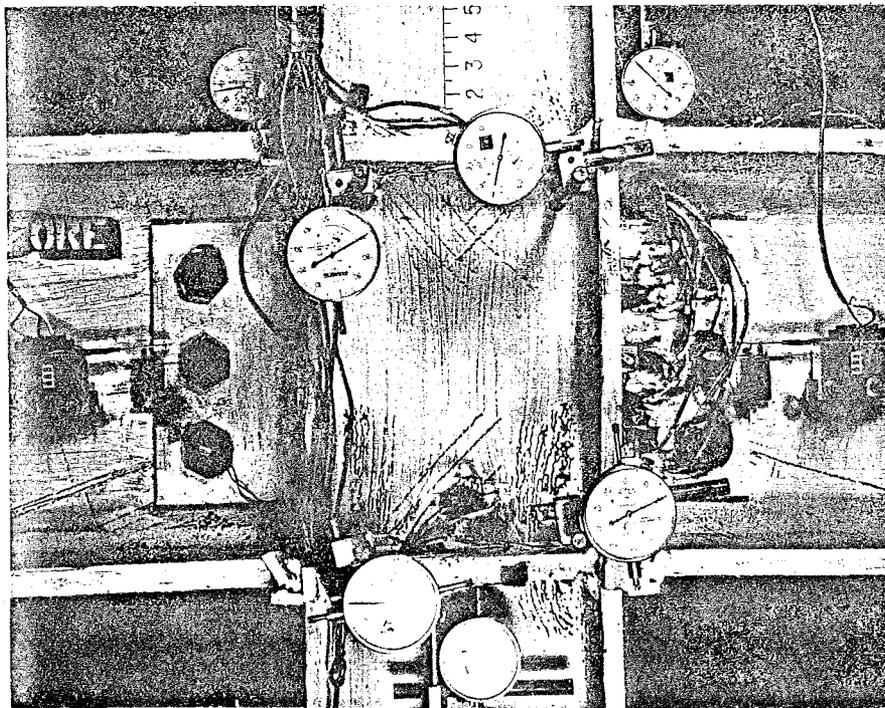Fig. 25 Load-Deflection Curves of C1 and C10

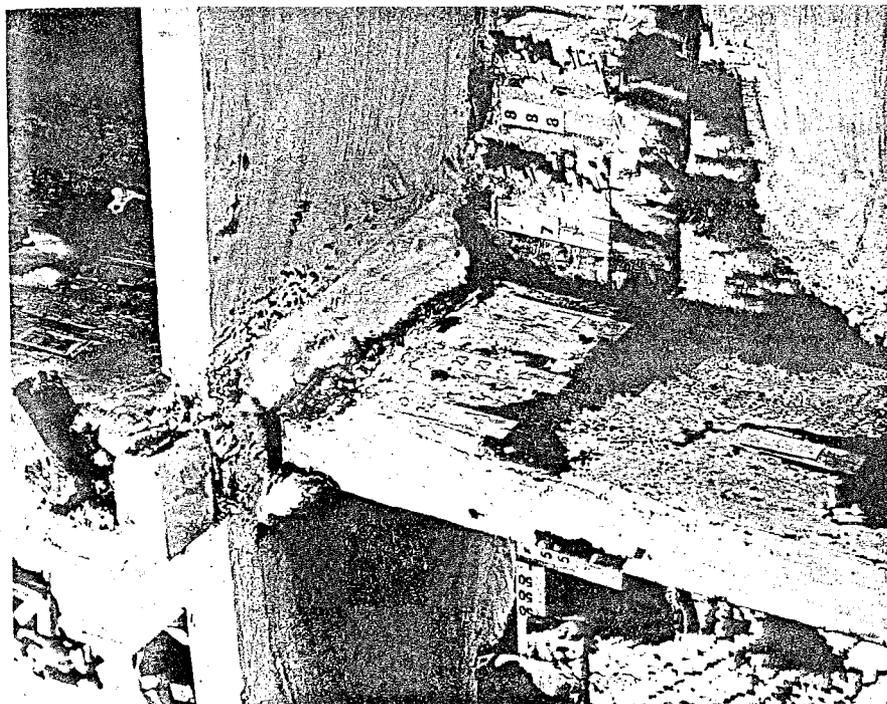Fig. 26   Panel Zone of C1 at Plastic Limit Load

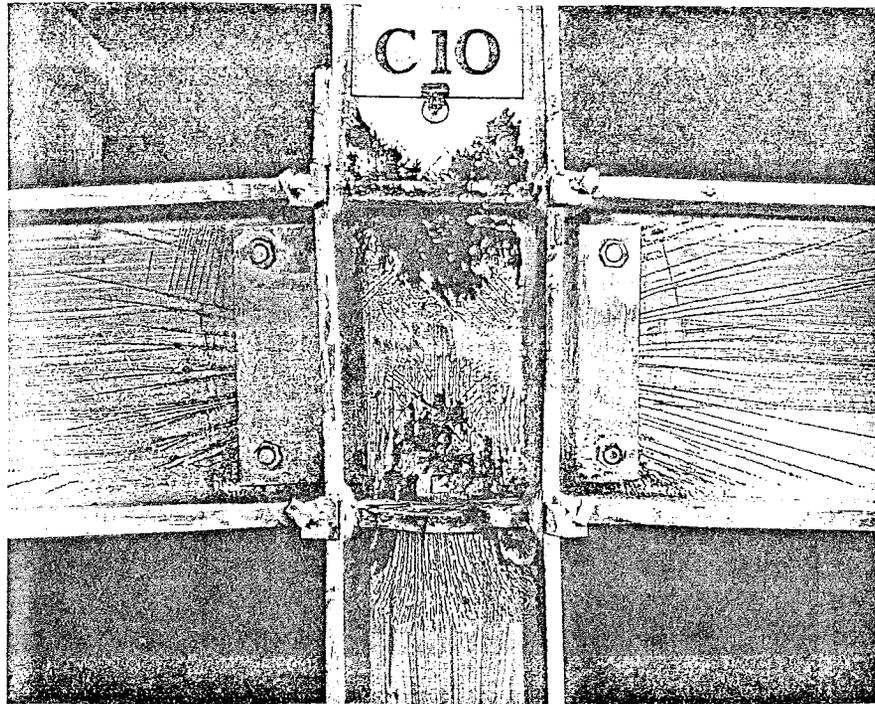Fig. 27   Crack at the Fillet Weld of the Tension Stiffener of C1

Fig. 28   C10 After Testing.   The Compression Horizontal Stiffeners Buckled After
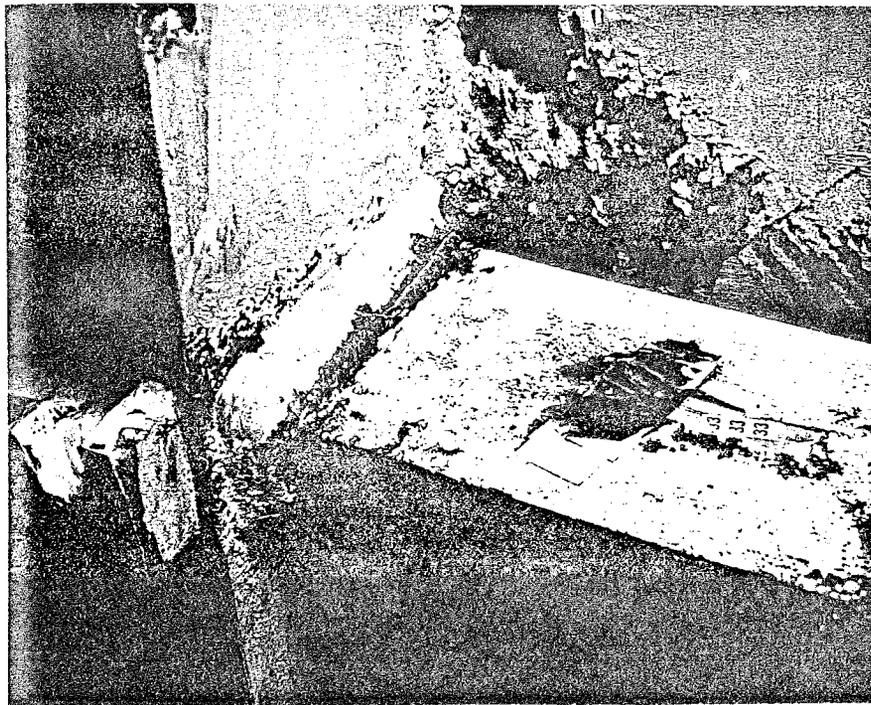the Attainment of Plastic Limit Load.

Fig. 29   Crack at the Fillet Weld of the Tension Stiffener of C10
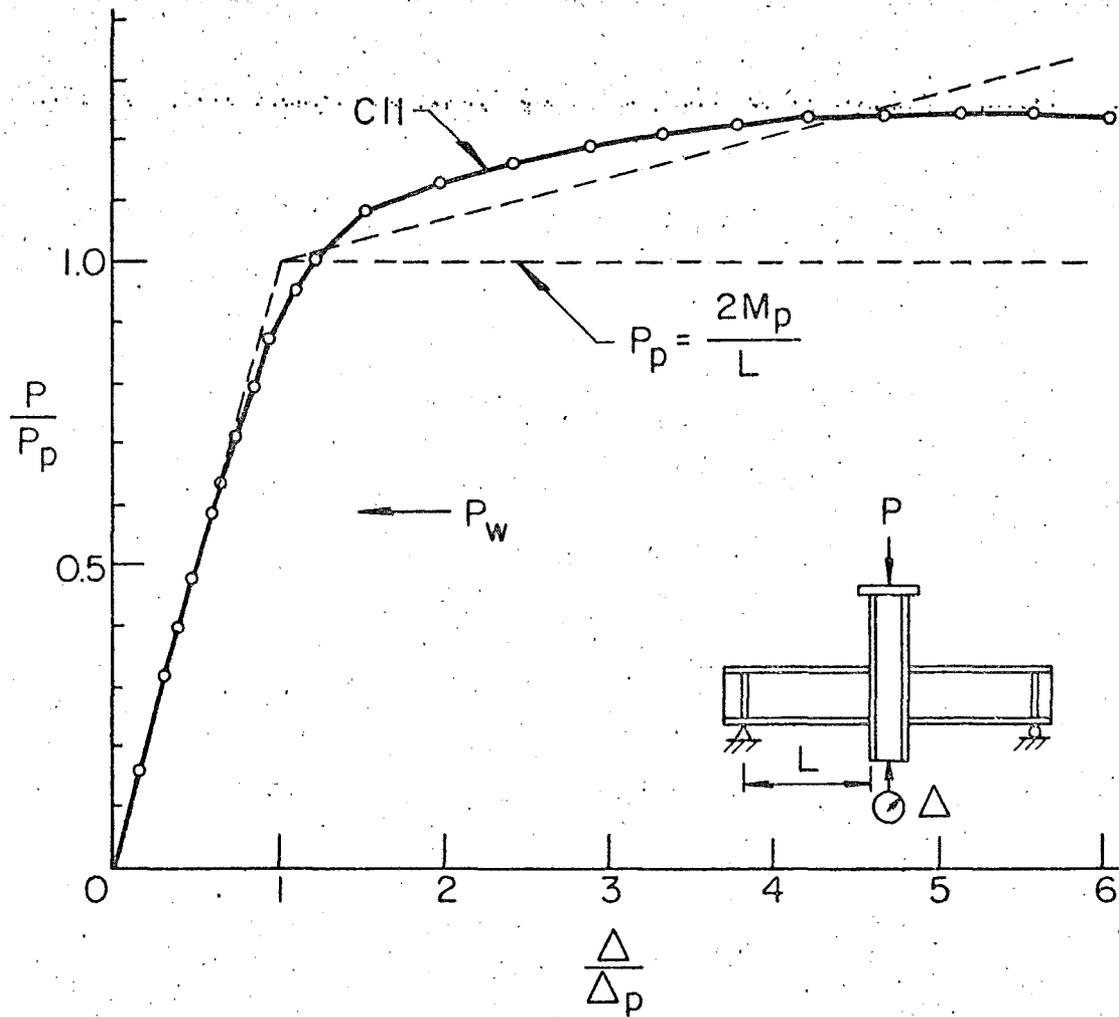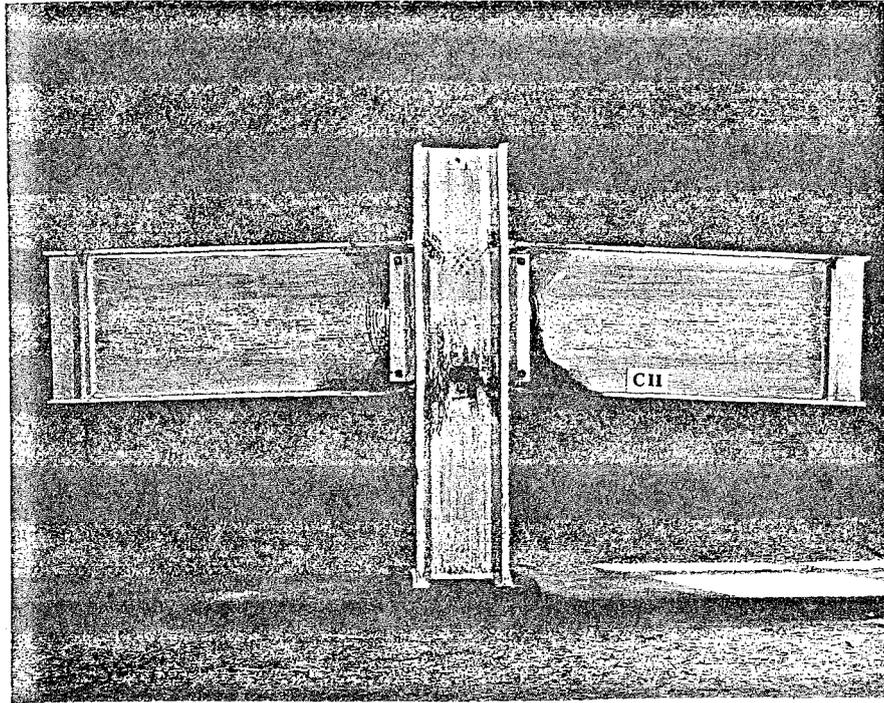
Fig. 30 Load-Deflection Curve of C11

Fig. 31 Over-All View of C11 After Testing. Plastic Hinges Formed at the Joints.
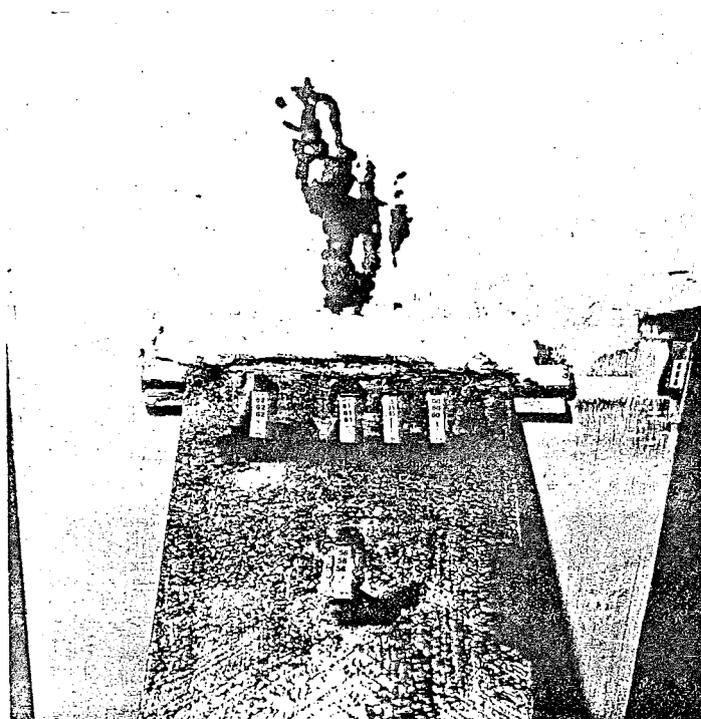

Fig. 32 Crack at the Heat-Affected Zone of the Groove Weld at the Tension Flange
of C11

APPENDIX I - REFERENCES

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APPENDIX II - <u>NOTATION</u>

The following symbols are used in this paper:

L = beam span;

M_p = plastic moment;

P = applied column load;

P_m = maximum load of a connection under test;

P_p = plastic limit load;

P_{pr} = plastic limit load assuming the area of beam web is zero;

P_{ps} = plastic limit load modified to include the effect of shear force;

P'_{ps} = plastic limit load modified to include the effect of shear force, assuming the area of beam web is zero;

P_w = working load, $P_w = P_p/1.7$;

V = shear force;

V_p = shear force that produces full yielding of web;

Δ = deflection;

Δ_m = maximum deflection;

Δ_p = deflection at plastic limit load; and

μ = ductility factor, $\mu = \Delta_m/\Delta_p$.