Cyclic Behavior of Large Beam-column Assemblies

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A series of experiments were conducted to verify the design criteria for beam-column joints under extreme seismic conditions for a 47-story building in San Francisco. The half-scale cruciform specimens were exceptionally large, requiring 18 in. deep sections. The overall size of the specimens was the largest ever tested in the U.S. for this kind of application. The data on the behavior of such large moment-resisting joints under severe cyclic loading are very limited. The experimental evidence clearly supports the use of stiffeners and doubler plates at the joints for the cross-sectional geometries tested. The results are of direct relevance to seismic design of many steel buildings.

INTRODUCTION

During the design of a perimeter ductile moment-resisting space frame for a 47-story building in San Francisco, a question arose regarding the need for stiffeners and doubler plates at the joints. In recent years, the need to use these costly elements has been questioned, and eliminating them has been justified ostensibly by calculations. In as much as the joint panel zones on this project were very large (approx. 36 x 36 in.), there was some concern regarding the joint behavior if the less conservative approach were to be followed and a substantial experimental and analytical program were developed. A series of specimens were designed and tested to provide the answers sought. For these reasons, the joints of some specimens had no stiffeners (continuity plates), whereas others were provided with them. In one of the experiments, the doubler plate was deliberately left out to study the shear deformations of the panel zone. In some specimens, no vertical panel zone stiffeners were provided in order to examine the buckling characteristics of the column web in these regions. Such conditions may be found in mechanical floors of tall buildings. Several specimens had vertical stiffeners between top and bottom continuity plates. This situation is typical in usual floor framing, as beams commonly occur at right angles to panel zones.

The columns of each specimen tested, in addition to being subjected to in-plane bending and shear by the girders framing into a joint, were also subjected to axial loads. These axial loads simulated the largest axial stress caused in the structure by gravity loads and overturning. It would appear that this frequently encountered condition was not simulated thus far in any other U.S. experiments on specimens of realistic size.

The very limited experimental data[2-7] on the behavior of joints under the conditions enumerated above were the compelling reasons for proceeding with the test program.

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TEST DESCRIPTION

General

As noted, the prototype beam-column assemblies were very large and in most cases involved 36-in. deep members. The loading required on such assemblies to induce significant deformation or failure would have been beyond the capacity of the available equipment. To reduce this loading requirement, the members were made half-size. As rolled sections were not always available to meet the one-half

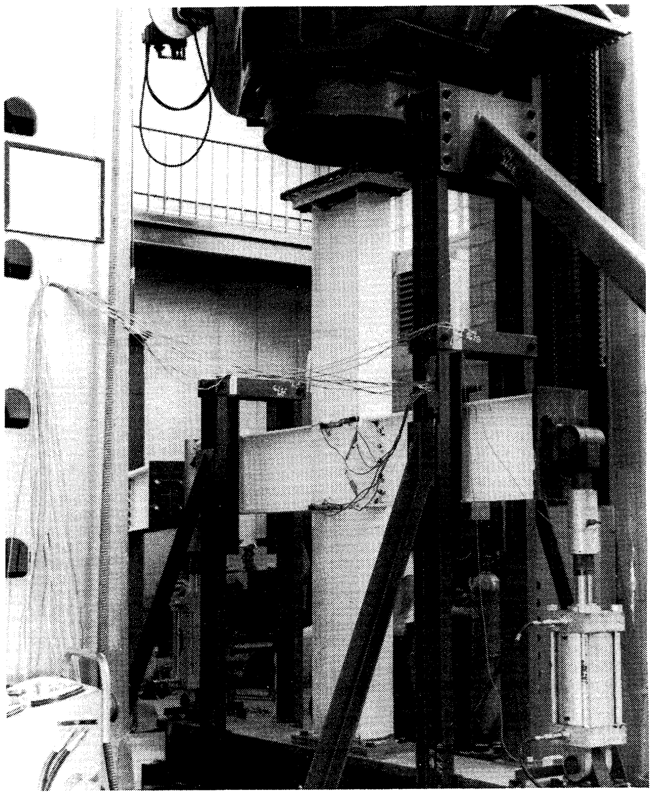


Fig. 1. General view of ~~test~~ arrangement

scale factor chosen, they were fabricated from rolled plate cut to the appropriate dimensions. Each specimen consisted of a single column 9ft-6½ in. long, with cantilever beams on each side of the column at its mid-height. The column length was chosen so the inflection points due to the applied moment at the middle of the column would produce inflection points corresponding to mid-stories in the prototype. For this purpose, both column ends were assumed to be fixed. To cause failure in the specimens, the cantilever beam lengths were set by the capacity of the available actuators.

A special fixture was designed to accommodate the specimen so axial load could be applied to the column and at the same time the beams could be loaded in bending. The axial load was applied through the 4,000-kip Southwark-Emery Universal Testing Machine, and bending loads were applied with 8-in. bore hydraulic actuators. The general arrangement of the experimental setup is shown in Fig. 1, and the design of the special testing fixture is shown in Fig. 2. The experiments were performed at the Richmond Field Station facilities of the University of California, Berkeley.

Design of specimens was based on the following criteria:

1. The beam-column assemblies conformed to the SEAOC recommendation[8] of strong-column weak-beam concept.
2. Doubler plates in the panel zones were sized in accordance with SEAOC recommendations[8] for a ductile moment-resisting frame.

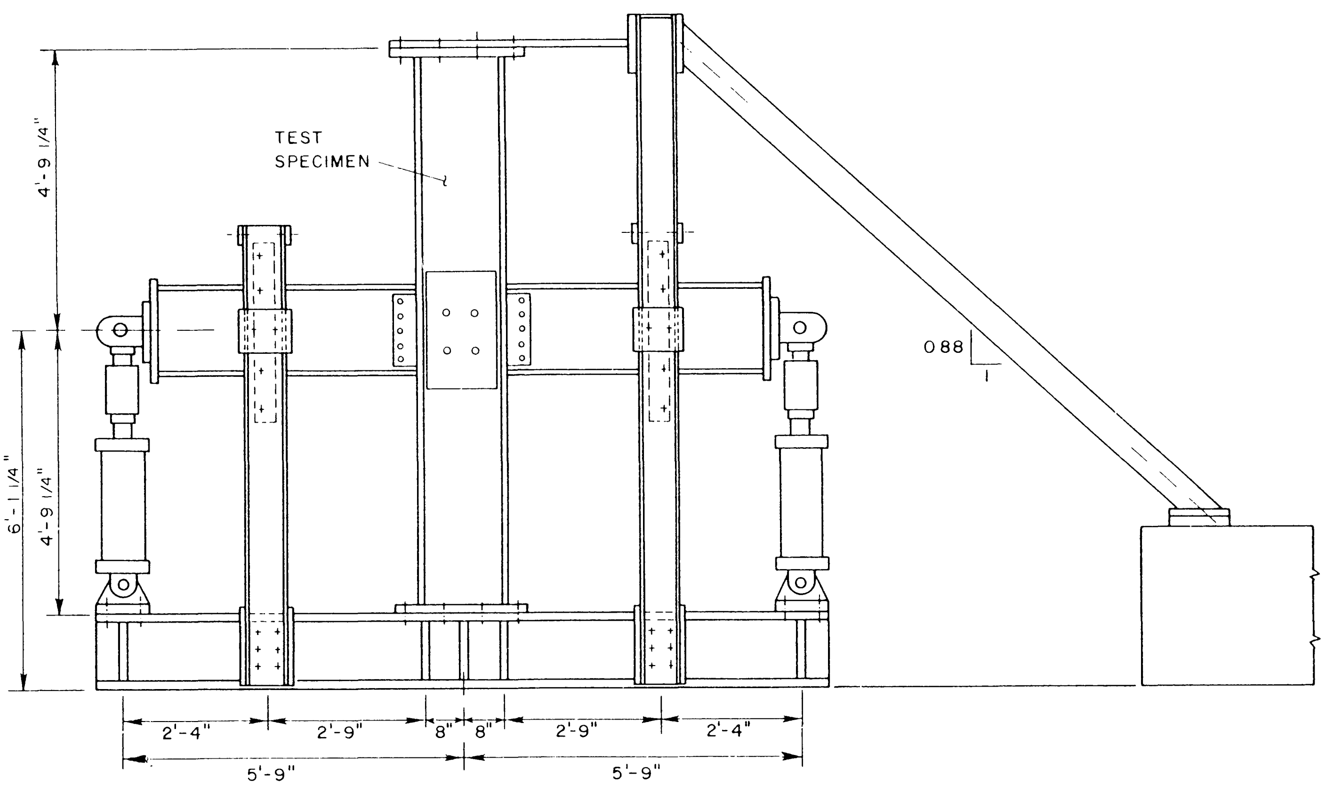


Fig. 2. Test fixture for mounting specimens

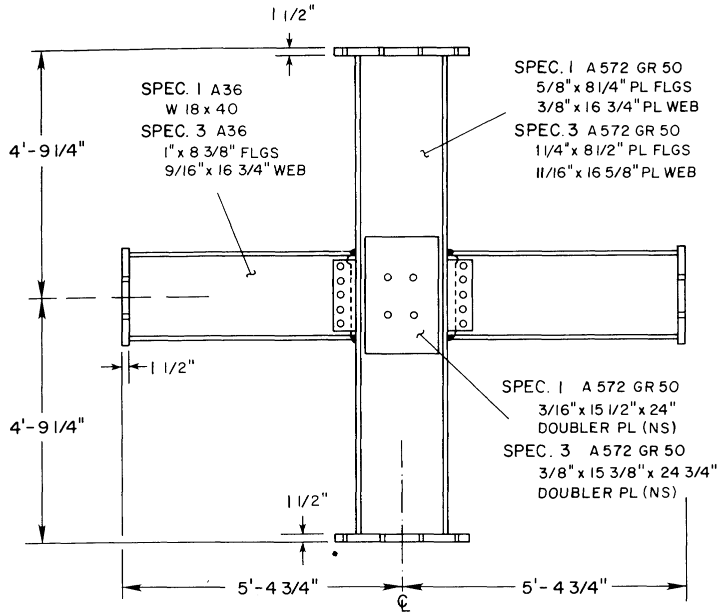


Fig. 3. Specimens 1 & 3—general configuration

3. Stiffeners (continuity plates) were designed according to the AISC Specifications.[1]

The geometry of the specimen was chosen to exhibit the joint behavior during severe cyclic tests of the deep columns and deep beams frequently occurring in frame-tube structures. Behavior of the stiffeners, doubler plates and welding was of particular interest.

DESCRIPTION OF SPECIMENS

Specimen 1—The column section was fabricated using A572 Gr. 50 steel with an overall width of 18 in. The beams were W18x40 rolled sections made from A36 steel. Fabrication details are shown in Figs. 3 and 4a. It should be noted, the backup plates for the welds on the beam flange-to-column flange connections were removed after the full-penetration flange welding was completed and small cosmetic welds appear to have been added and ground off on the underside. This procedure was followed in the fabrication shop prior to their delivery for testing. The same procedure of removing the backup plates was done also on Specimens 2, 3 and 4. A 3⁄16-in. doubler plate was included in the panel zone section on one side of the specimen. A325 high-strength bolts were used to connect beam webs to the shear tabs. The same kind of bolts were also used for Specimens 2, 3, 4, 5 and 6.

Specimen 2—This specimen was fabricated to the same details as Specimen 1, except stiffeners (continuity plates) were added. The size and location of these plates are noted

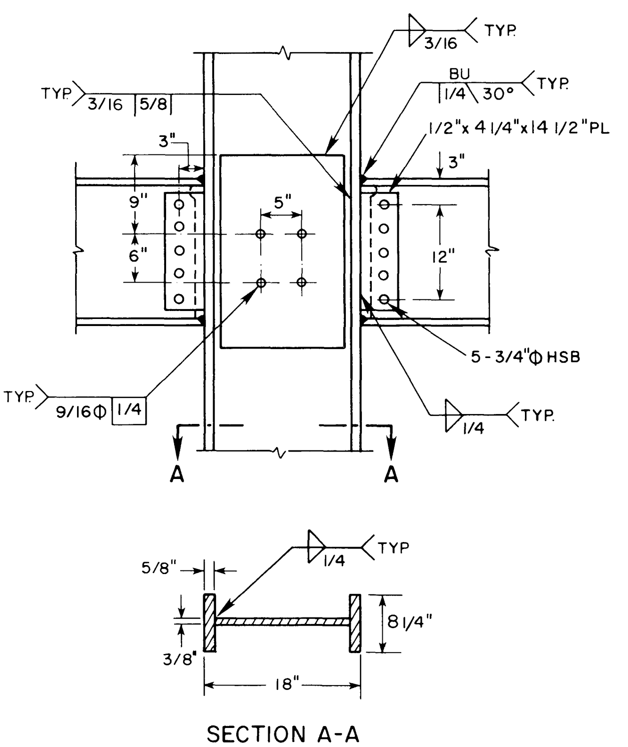
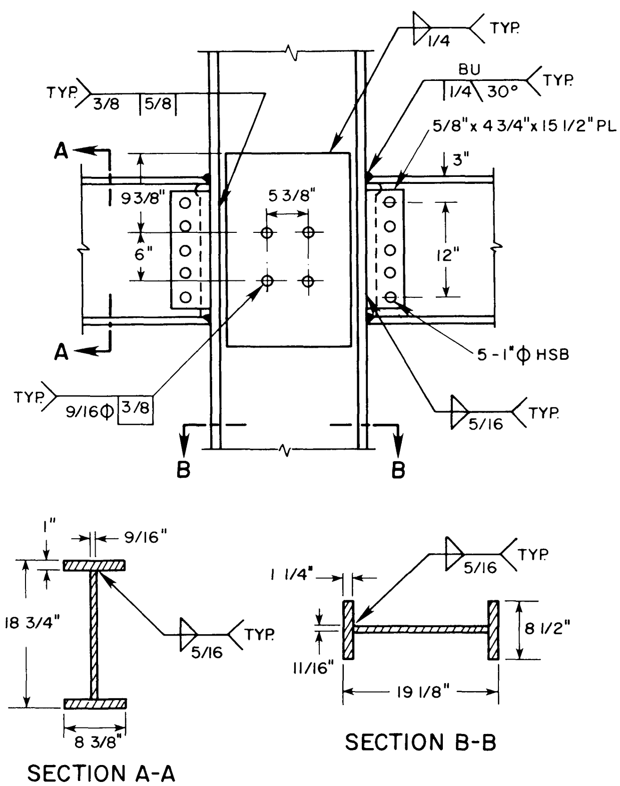


Fig. 4a. Details for Specimen 1



4b for Specimen 3

in Fig. 5. A 3/16-in. doubler plate was again included in the panel zone section.

Specimen 3—The column section was fabricated using A572 Gr. 50 steel with an overall width of 19⅛ in. The beam sections were made of A36 steel having an overall depth of 18¾ in. In this case, a ⅜-in. doubler plate was included in the panel zone section. The fabrication details are shown in Figs. 3 and 4b.

Specimen 4—This specimen was basically the same as Specimen 3, except in this case, stiffeners were added. A ⅜-in. doubler plate was included in the panel zone section (see Figs. 4b and 5).

Specimen 5—Specimen was a reworked version of the failed Specimen 4, which included the stiffeners and a ⅜-in. doubler plate in the panel zone. The beam-to-column flanges were rewelded and the backup plates left in place after fabrication.

Specimen 6—This specimen's fabrication was similar to that of Specimen 4, except the doubler plate was left out. However, a ¼-in. vertical stiffener was provided on one side of the panel zone section between stiffeners. The backup plates on the beam-to-column flange welds were left in place after welding (see Figs. 4b and 5).

Specimens 7 and 8—The columns on these specimens were A572 Gr. 50 steel made from W21x93 rolled section. The beams were A36 steel made from W18x71 rolled sections. A ⅜-in. doubler plate on one side of the panel zone and on the opposite side a ¼-in. vertical stiffener were included. This vertical stiffener was offset ⅛ in. from the center of the column and extended from the top to the bottom stiffener.

The basic difference between these two specimens was in the beam-to-column flange welds. Specimen 7 had a 30° bevel and a ¼ in. root opening, and Specimen 8 used a 30° bevel with a ⅜-in. root opening. In both cases backup plates were left in place after welding was completed. The fabrication details for these two specimens are shown in Figs. 6 and 7. A490 high-strength bolts were used in the two assemblies.

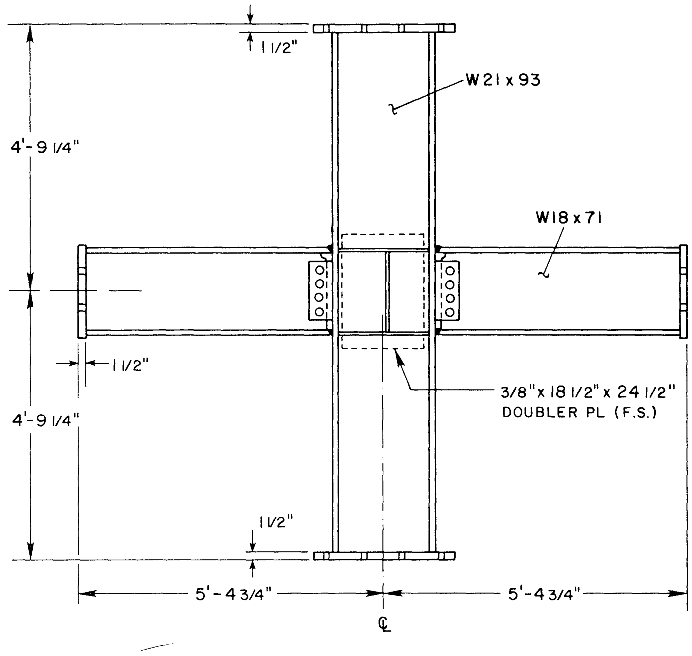
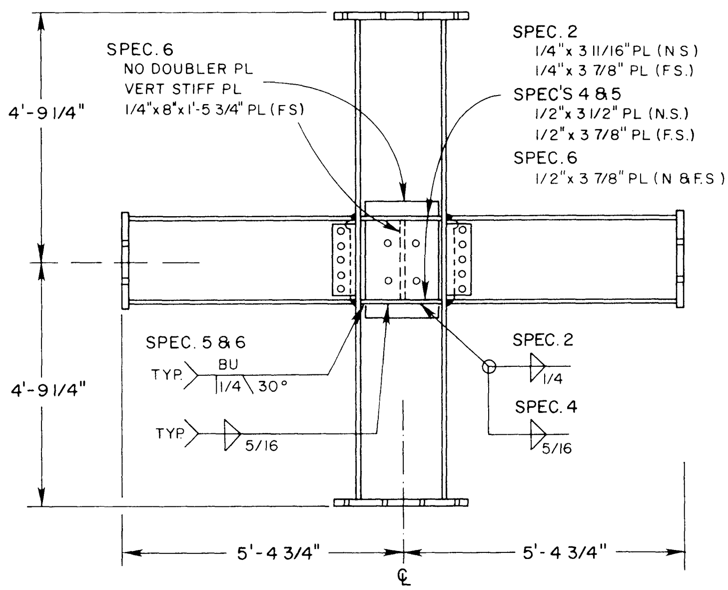


Fig. 6. Specimens 7 & 8—general configuration



SPEC. 2, EXCEPT AS SHOWN, SAME AS SPEC. 1
SPECS. 4, 5 & 6, EXCEPT AS SHOWN, SAME AS SPEC. 3

Fig. 5. Specimens 2,4,6—general configuration

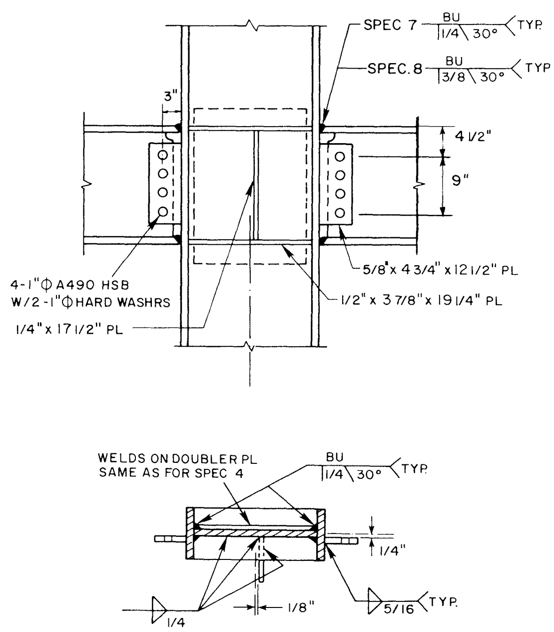


Fig. 7. Details for Specimens 7 & 8

TESTING PROCEDURE

Experimental Setup

Each specimen was installed in the specially designed test fixture (Fig. 2) with the top of the column rigidly bolted to a plate, which in turn was attached to a vertical member and a diagonal brace. The test fixture provided for flexibility in the vertical direction and rigidity in the horizontal direction. The bottom of the column was bolted to the W14x233 spreader beam.

The beams were loaded using hydraulic actuators on each end of the beam. A clevis was bolted to the end of each beam and a 3-in. dia. pin attached the clevis to the eye on the end of the actuator assembly.

Vertical channels were used to provide lateral support for the beams. These channels were bolted to the W14 beam and to braces attached to the testing machine base plate.

Each specimen was instrumented with linear potentiometers to measure tip deflections of the beams, panel zone deformations and longitudinal deformation of the stiffeners. In addition, a linear potentiometer was attached to the column just below the beam to measure the horizontal deformation of the column at this point to provide an overall check on panel zone deformation. The hydraulic actuator loads were monitored by load cells attached to the ends of the rods on the actuators. The testing machine load was monitored through the machine's load cell.

All of the instruments and load cells were attached to a data acquisition system and the data were read and initially recorded in a computer, and later transferred to magnetic tape for subsequent data reduction. Two XYY recorders were used during the testing to monitor the beam tip deflections vs. beam load and the panel deformation vs. beam load. Before each test, the specimen test section was whitewashed to visually observe any yield patterns which might develop under load.

Loading Sequence

The column axial load was initially applied up to a level of a nominal stress of 21 ksi, which corresponded to the largest axial stress in the prototype due to gravity loads and overturning. This axial load was maintained throughout the test program. The beams were then cyclically loaded so an upward load was applied to one beam and a downward load to the other. Initially, the beams were loaded through two cycles to approximately one-half the nominal allowable bending stress (12 ksi) at the column face. The beam loads were then increased to the nominal allowable stress (24 ksi) and again two cycles were applied. These elastic cycles besides providing information on the elastic stiffness of the system, also served to check the instrumentation.

The loads were then increased to reach a nominal yield stress of 36 ksi, assuming elastic behavior. One to two cycles were applied at this load level. The next load step was such as to obtain the 36 ksi stress level based on the plastic modulus of the section. Again, one to two cycles of loading were applied. From this point on, the loads were applied to cause increments in the overall ductility of the assembly of about one-half of that exhibited at first yield. These increases were made until failure in a specimen occurred.

EXPERIMENTAL RESULTS

General

The AISC Manual[1] section properties were used for all rolled sections. In cases where beams were fabricated from plate, elastic moduli were calculated based on the nominal plate dimensions. It was sufficiently accurate to assume plastic moduli for all members to be 12% larger than the calculated elastic moduli because of the insensitivity of the results to this parameter.

A large number of tensile test coupons was made from plates and sections supplied by the fabricator and tested to determine the mechanical properties of the materials used. The plates used to fabricate columns on the average showed yield stress of about 49 ksi, and an ultimate of about 70 ksi. The A36 beam flange plates had a yield of 38 ksi, and an ultimate of 66.4 ksi. The flanges of W18x50 section of A36 material had a yield of 46.4 ksi, whereas the flange yield for the W18x71 section was 43.5 ksi. Both the flanges and the web of Gr.50 W21x93 section had yields very near to 60 ksi. With the exception of one case, the stress-strain curves were typical for that of mild steel.

In the panel zone area, each of the two linear potentiometers provided measurement of the relative movement of two diagonally oriented points. From these measurements, the average shear distortion or strain can be obtained geometrically as:

$$\gamma_{av} = \frac{\Delta - \bar{\Delta}}{2} \frac{d}{bh}$$

where,

Δ and $\bar{\Delta}$ are the measured displacements of the sets of points, d is the length of the diagonals between points, b is the width between points, and h is the vertical distance between the points.

The total moment in the panel zone section was taken as the sum of the applied loads times their respective essentially equal distances from the face of the column to the center of the clevises where the loads were applied.

In each of the eight experiments, the maximum attained loads, as well as the initiation of yielding together with the ultimate inelastic deformations of the beams and panel zones, were measured to provide the basic data. A good deal of such information is exhibited in this paper in the form of hysteretic loops. From this data, the inelastic beam rotations and panel zone distortions can be determined. Observations on failure modes, which are also of great interest, are shown on the accompanying photographs.

Specimen 1—The average cantilever load vs. average beam

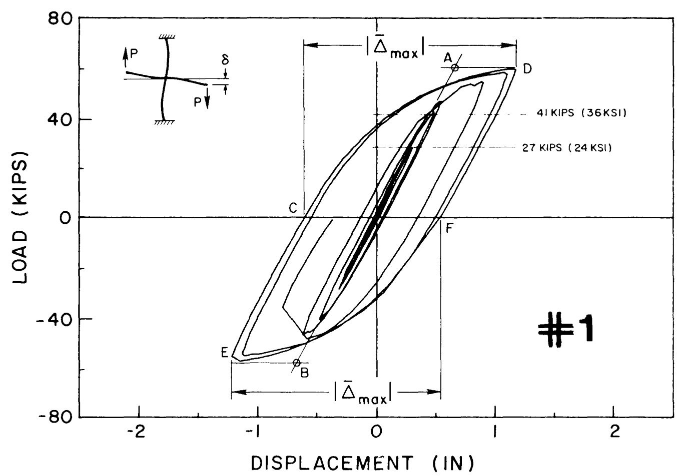


Fig. 8. Average cantilever end load vs. average beam-end deflection—Specimen 1

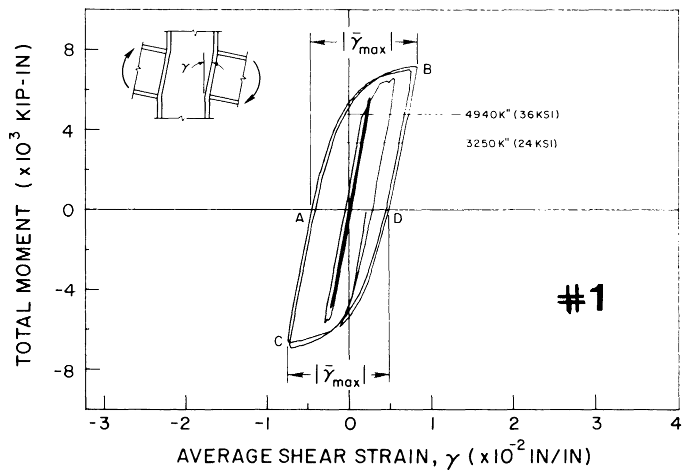


Fig. 9. Total applied moment vs. panel shear strain—Specimen 1

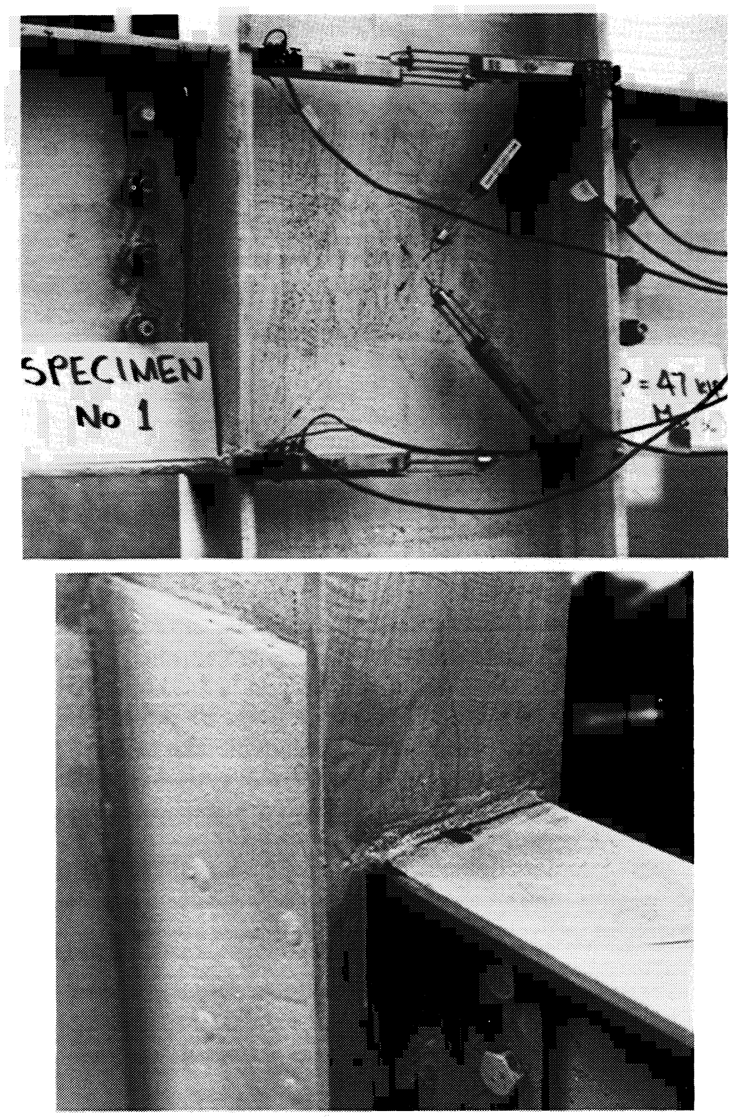


Fig. 10. Specimen 1a instrumentation arrangement and 1b failure location

end deflection for Specimen 1 is shown in Fig. 8. The specimen responded elastically up to and slightly beyond the load equivalent to the nominal maximum beam stress of 36 ksi. The total applied moment vs. panel shear strain is shown in Fig. 9. All of the hysterestic loops exhibited stable characteristics. The maximum average load reached by the actuators was about 60 kips at an average tip displacement of 1.2 in.

In Fig. 10a, the linear potentiometers for measuring horizontal displacements between flanges, as well as a pair of similar instruments placed diagonally for determining the panel zone distortion, are shown. The photograph in Fig. 10a was taken at the nominal beam yield load, and one can note the cracking of the whitewash in the panel zone indicating yielding of the column web. The specimen failed abruptly with noise; complete flange failure occurred in the heat-affected zone of the beam-to-column weld. The fractured specimen is shown in Fig. 10b. Note, no cracking of whitewash can be observed on the doubler plate, showing little participation of this relatively thin plate in resisting panel shear. During the test, the panel zone buckled at center ⅛ in. to $^3\!/_{16}$ in. out of plane.

Specimen 2—The average cantilever load vs. average beam deflection is shown in Fig. 11. Again, the specimen responded elastically up to and beyond the load equivalent to the maximum nominal stress of 36 ksi. The total applied moment versus panel shear strain is shown in Fig. 12. The hysteretic loops exhibited very stable characteristics. The average maximum load applied was approximately 71 kips at an average maximum tip displacement of about 2.4 in. Fig. 13a is a photograph of the joint at the end of test at an overall ductility of about 5. Note the significant cracking of the whitewash in the panel zone, indicating shear yielding. However, shear yielding of the doubler plate (not shown)

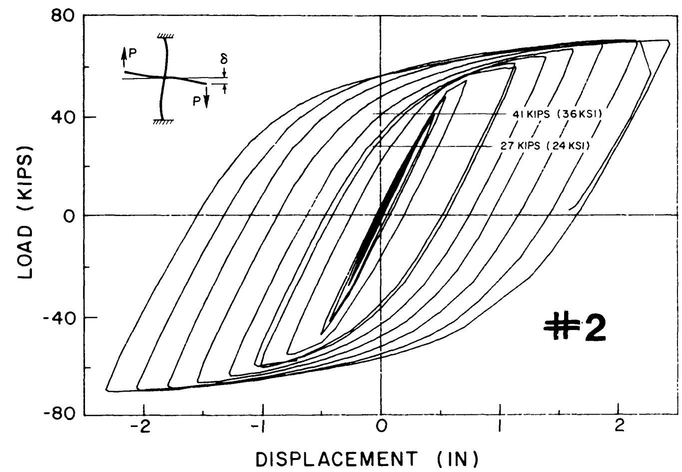
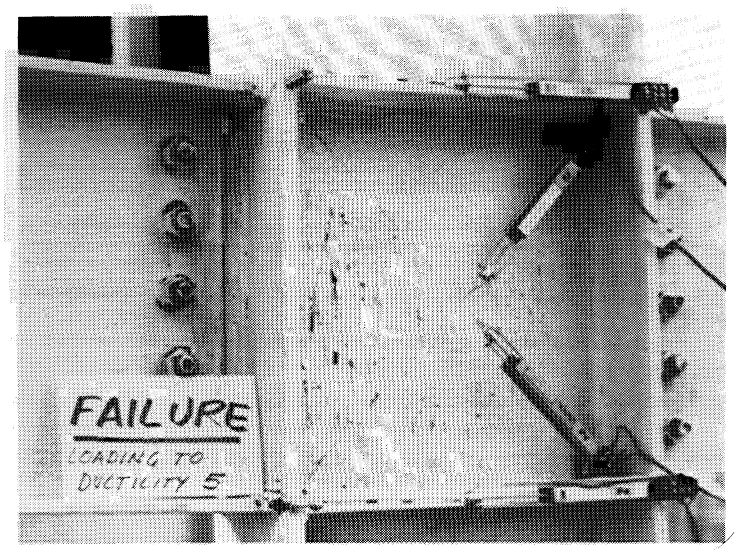
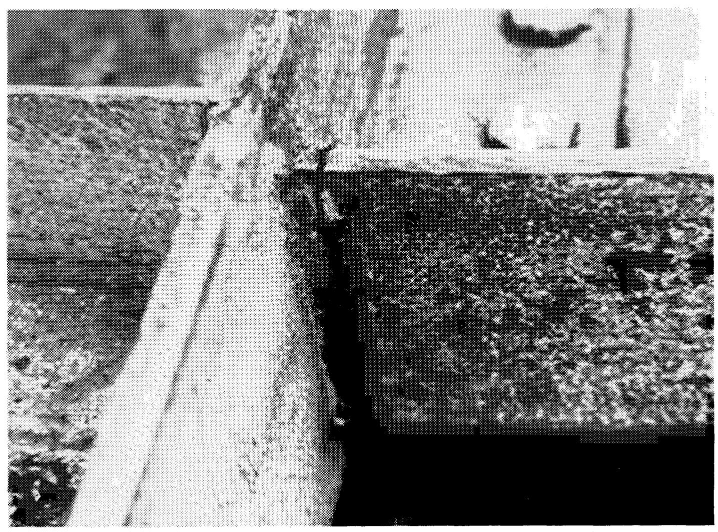


Fig. 11. Average cantilever end load vs. average beam-end deflection—Specimen 2



(a)



(b)

Fig. 13. Specimen 2 after failure

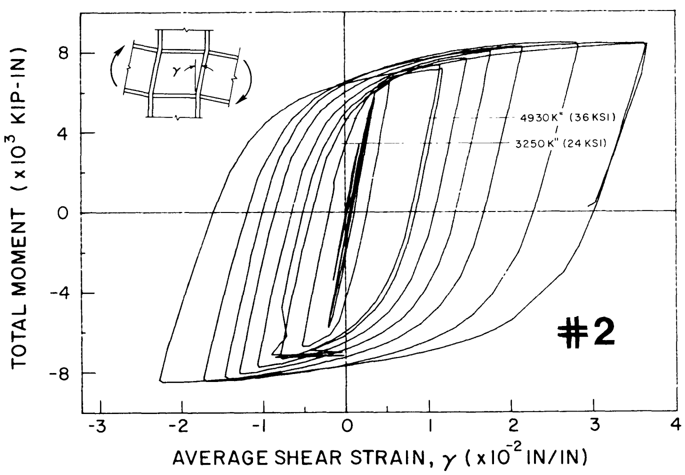


Fig. 12. Total applied moment vs. panel shear strain—Specimen 2

was very limited. Fig. 13b shows the fracture of the bottom beam flange weld which occurred in the heat-affected zone. At the end of the experiment, it was noted that the column web buckled at the center of the panel zone about 0.9 in. out of plane.

Specimen 3—The average cantilever load vs. average beam deflection for Specimen 3 is given in Fig. 14, and the total applied moment vs. panel shear strain is shown in Fig. 15. The average maximum beam tip load was about 125 kips at an average tip displacement of 1.4 in. Figure 16a shows the doubler plate at the end of the test indicating strong participation of the ⅜-in. thick doubler plate in resisting the panel shear. The fracture of the flange weld is shown in Fig. 16b. The failure was very abrupt.

Specimen 4—The average cantilever load vs. average beam end deflection for Specimen 4 is shown in Fig. 17. The total applied moment versus panel shear strain is given in Fig. 18.

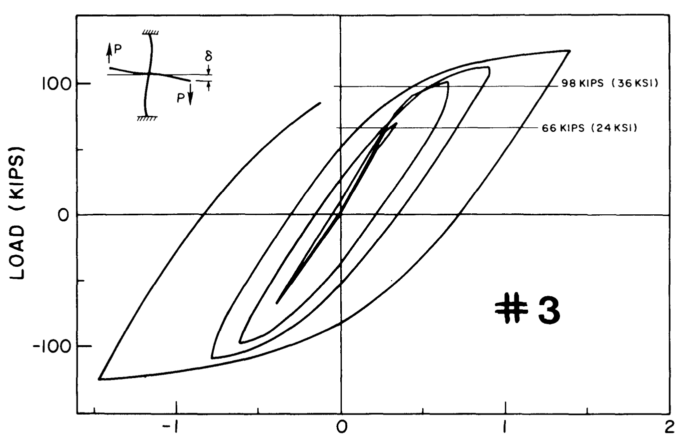


Fig. 14. Average cantilever end load vs. average beam-end deflection—Specimen 3

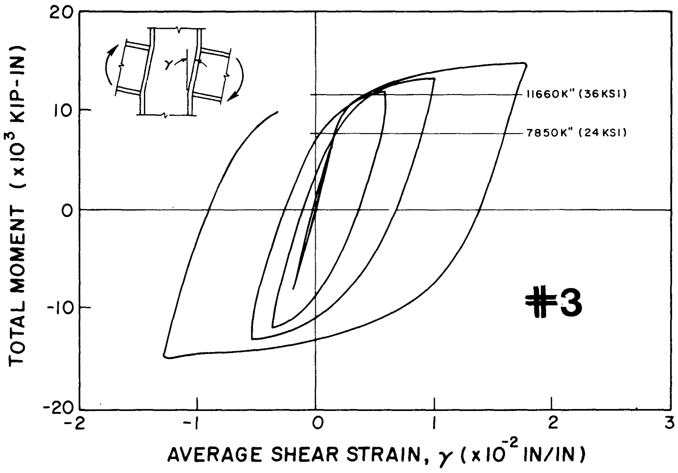


Fig. 15. Total applied moment vs. panel shear strain—Specimen 3

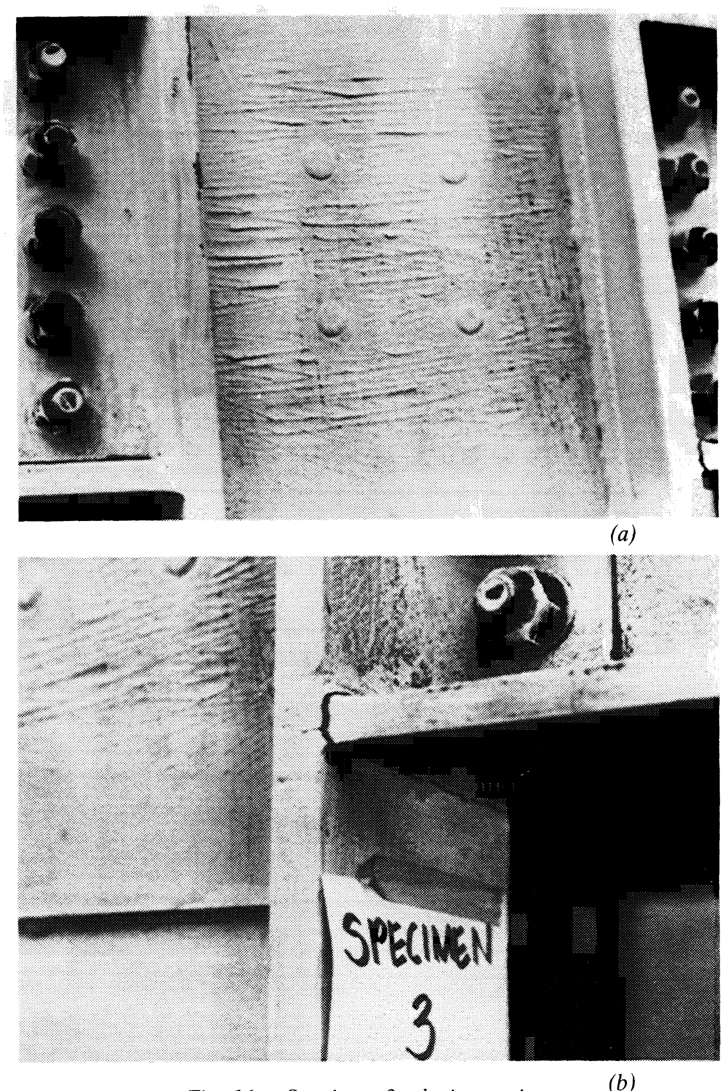


Fig. 16. Specimen 3a during testing and 3b after failure

Cyclic displacement between column flanges at the bottom of the beam is given in Fig. 19, where a sudden offset near the end of the cycling process can be readily recognized. This instant corresponds to the stiffener weld failure which may be seen in Fig. 20a. When this occurred, the joint reverted to the type without a stiffener, and a fracture of the flange, shown in Fig. 20b, took place. Prior to this premature failure, good participation of the doubler plate in resisting shear was evident. The average maximum beam tip load was 140 kips at an average tip displacement of 1.6 in.

Specimen 5—Because Specimen 4 failed prematurely, it was repaired and designated as Specimen 5. The stiffener welds to the column flanges were gauged out and rewelded with full-penetration welds. All beam flange welds were also gauged out and rewelded. The backup plates were left in place.

The average cantilever load vs. average beam deflection for Specimen 5 is shown in Fig. 21, and that total applied moment versus panel shear strain is given in Fig. 22. The average maximum load reached was about 112 kips, which occurred at an average beam displacement of only 0.65 in. The appearance of the fractured flange is shown in Fig. 23. The substandard performance of this specimen can be attributed either to a poor repair job or more likely to poor material used for the column flange plate, since there was some evidence of lamination in the fracture.

Specimen 6—This specimen was similar to Specimen 4, except no doubler plate was applied, making the panel zone weaker in shear. To prevent the panel zone from buckling, a vertical stiffener was attached approximately at mid-width of the panel zone. Such plates occur in most framing. The average cantilever beam load vs. average beam deflection is shown in Fig. 24, and the total applied moment vs. panel shear strain is given in Fig. 25. The average maximum load applied was 106 kips at an average beam tip displace-

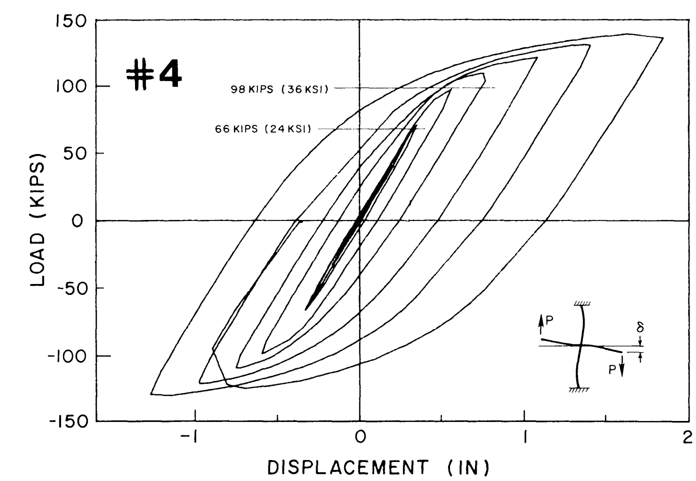


Fig. 17. Average cantilever end load vs. average beam-end deflection—Specimen 4

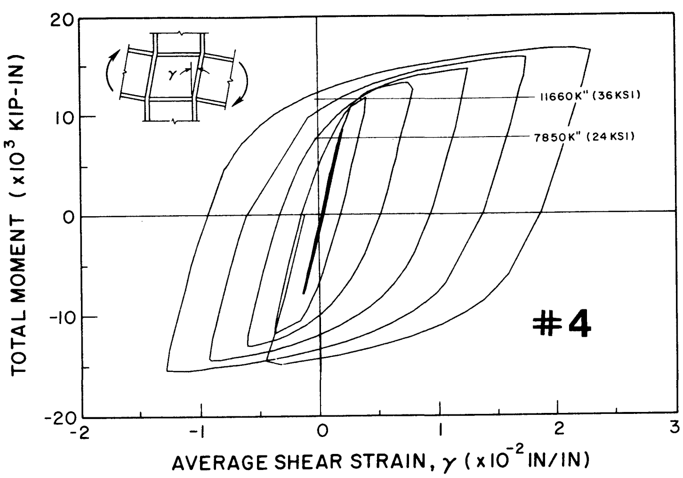


Fig. 18. Total applied moment vs. panel shear strain—Specimen 4

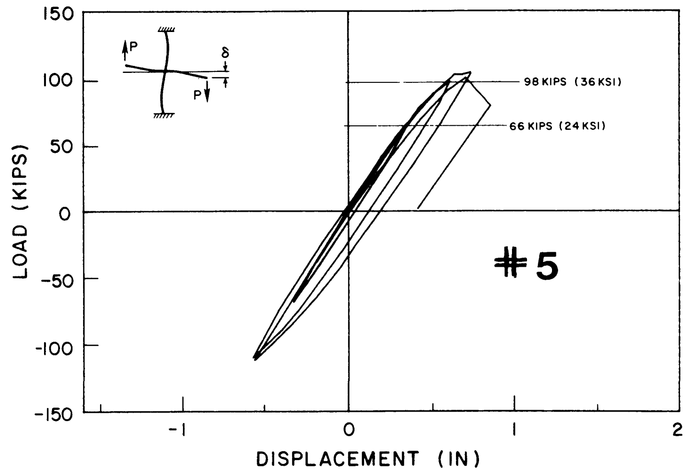


Fig. 21. Average cantilever end load vs. average beam-end deflection—Specimen 5

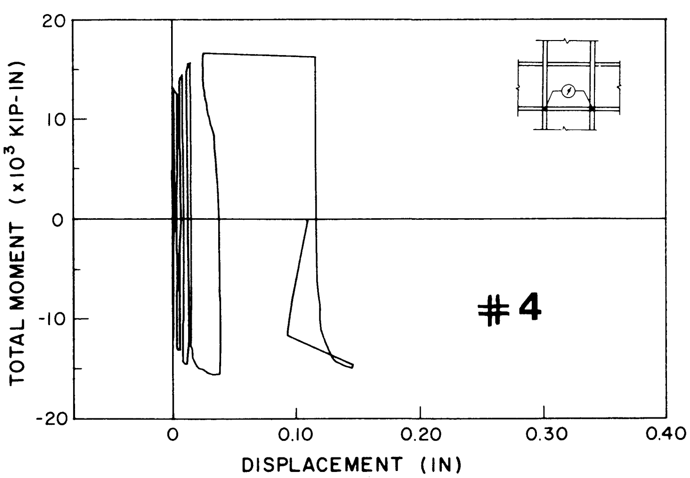


Fig. 19. Total applied moment vs. maximum displacement between column flanges at bottom of beam—Specimen 4

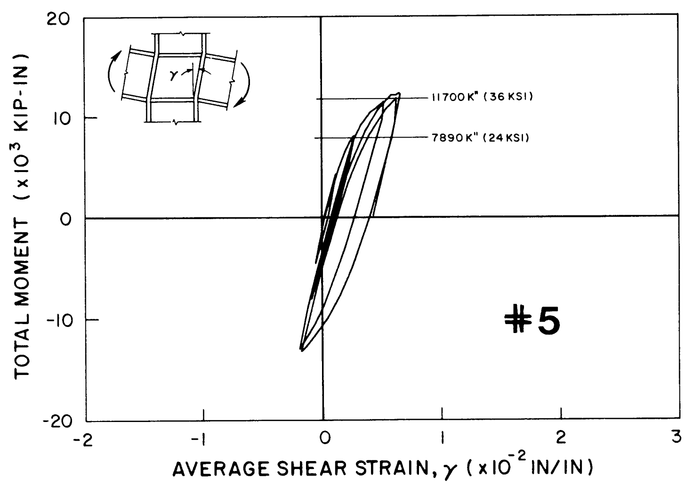
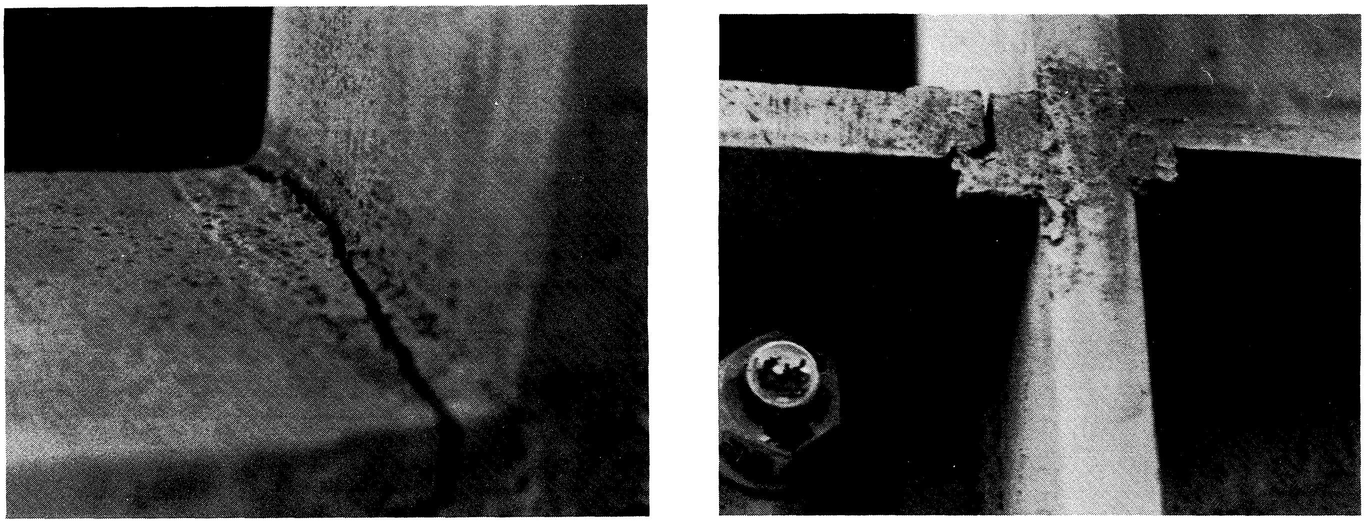


Fig. 22. Total applied moment vs. panel shear strain—Specimen 5

Fig. 20. Specimen 4 after failure



(a) Fig. 23. Specimen 5 after failure (b)

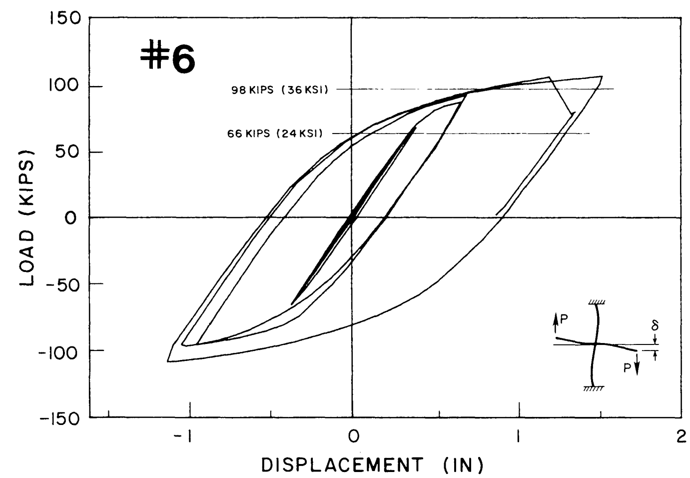


Fig. 24. Average cantilever end load vs. average beam-end deflection—Specimen 6

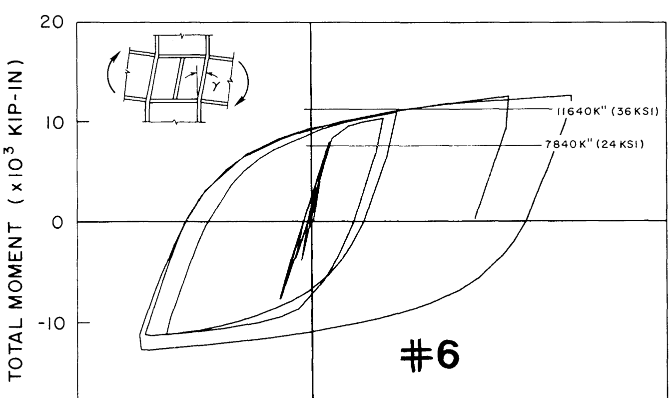


Fig. 25. Total applied moment vs. panel shear strain—Specimen 6

ment of about 1.3 in. Photographs of the fractured flange are in Fig. 26.

Specimen 7—Figure 27 shows the average cantilever load versus average beam end deflection for Specimen 7. The total applied moment versus panel shear strain is shown in Fig. 28. Prior to failure, all of the hysteretic loops exhibited stable characteristics. The maximum average applied load was 128 kips at an average beam deflection of 2.25 in. The displacements between the flanges at the stiffeners were determined using two potentiometers both at the bottom and top flange of the beam (Fig. 32a). Each potentiometer measured the displacement from the flange to the center of the column. Figure 29 shows the fractures in the top and bottom flanges of the beam. Note, the failures took place outside the weld and the fractures were rather ductile.

Specimen 8—The average cantilever load vs. average beam end deflection for Specimen 8, which was very similar to Specimen 7, is shown in Fig. 30; the total applied moment vs. panel shear strain is given in Fig. 31. Again, the hysteretic loops exhibited stable characteristics. The average maximum applied load was 132 kips at an average beam deflection 2.5 in. Figure 32a shows the joint during the test and Fig. 32b shows the fracture of the bottom flange. This specimen had two flanges fail at the same time: the bottom beam flange on one side and the top flange on the other side where the fracture occurred in the flange-to-column weld (not shown).

Note, in all cases the plug welds on doubler plates were found to be effective.

DISCUSSION OF RESULTS

General Observations

All specimens carried loads well above the 36-ksi nominal yield-strength of the cantilever beams. In general, specimens with stiffener plates (Specimens 2, 4, 6, 7 and 8) had better ductility than the others. Also, doubler plates con-

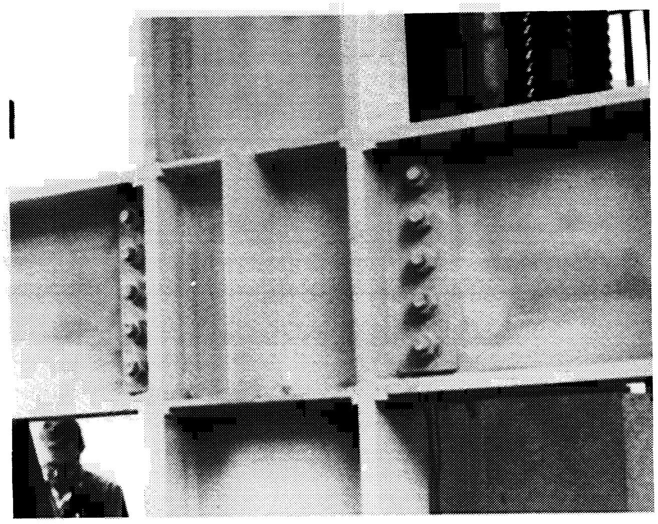
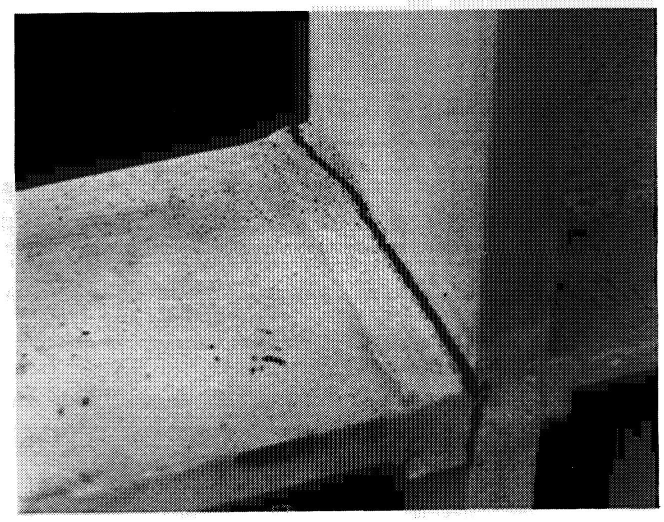


Fig. 26. Specimen 6a during test;



6b after failure

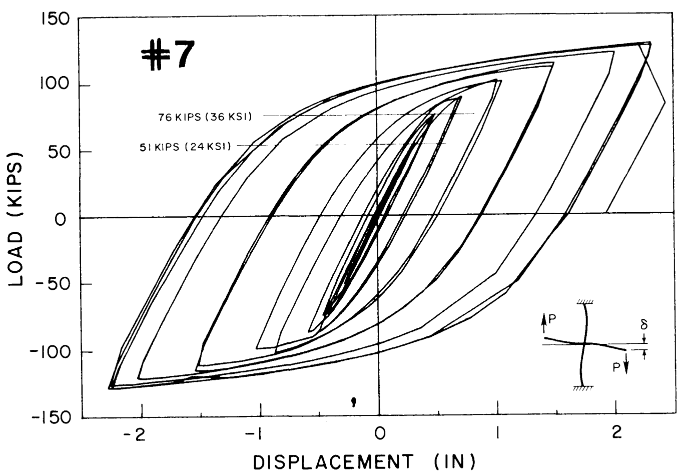


Fig. 27. Average cantilever end load vs. average beam-end deflection—Specimen 7

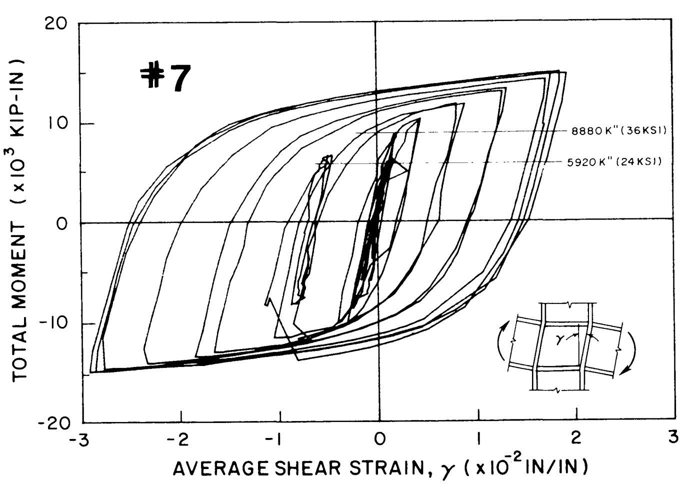


Fig. 28. Total applied moment vs. panel shear strain—Specimen 7

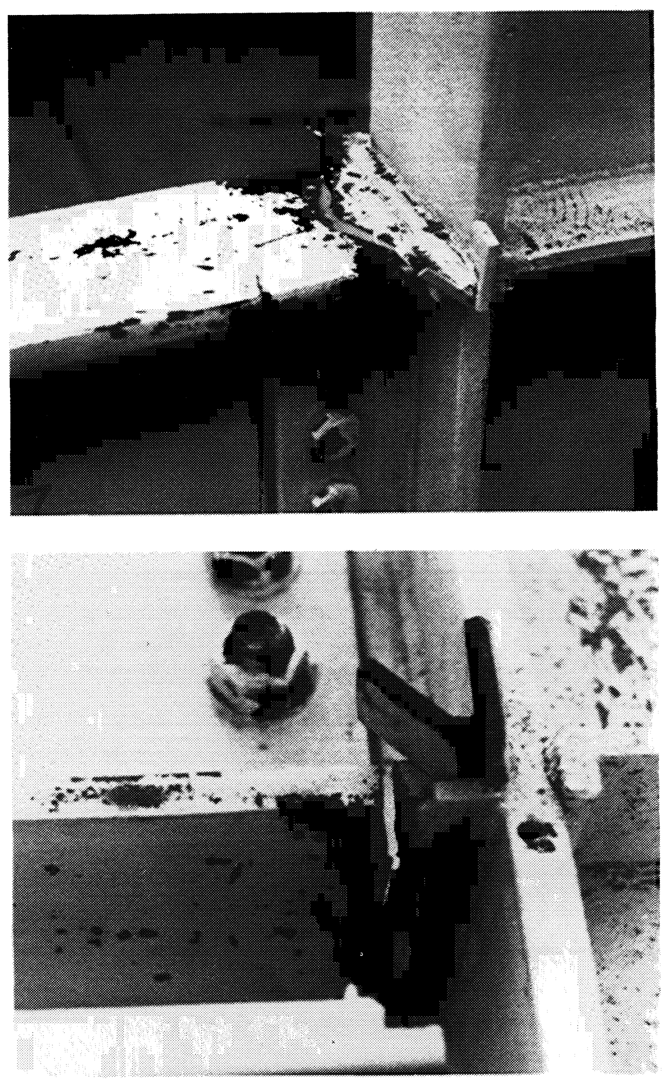


Fig. 29. Specimen 7 after failure

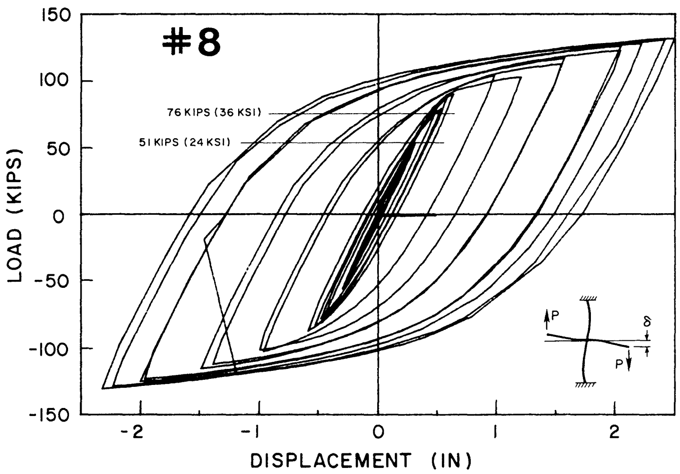


Fig. 30. Average cantilever end load vs. average beam end deflection—Specimen 8

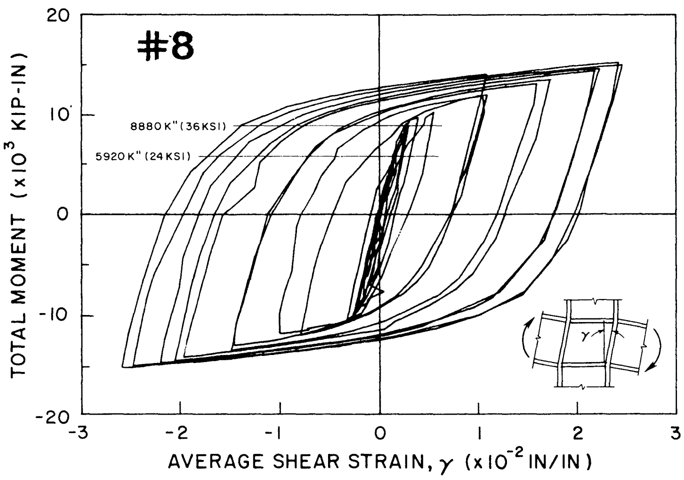


Fig. 31. Total applied moment vs. panel shear strain—Specimen 8

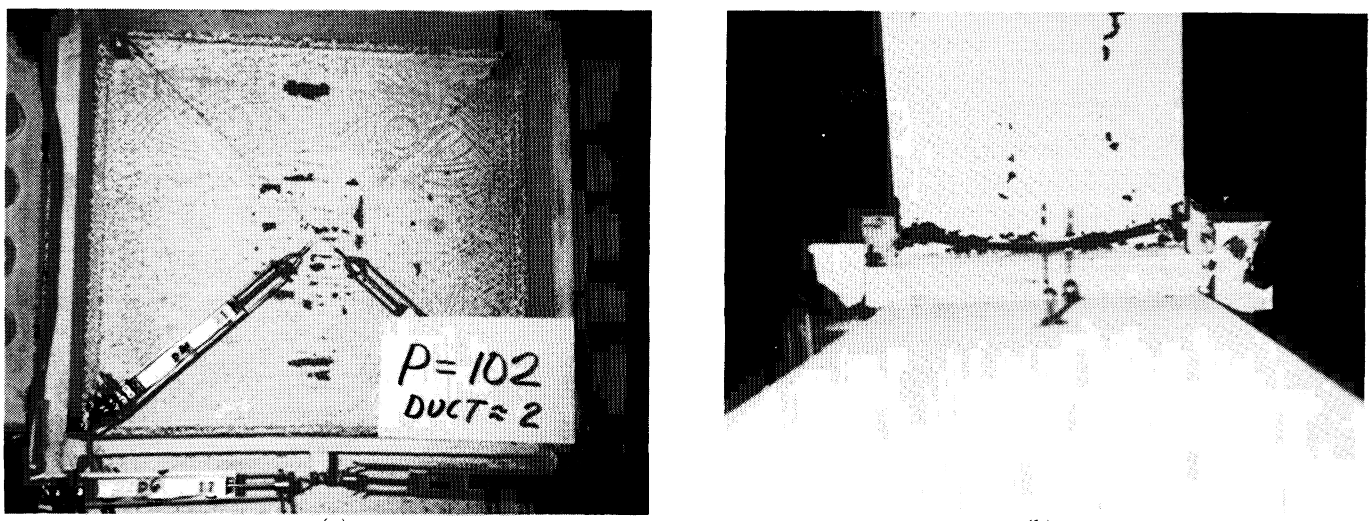
tribute to the capacity increase of joints, as noted by comparing the data for Specimens 4 and 6. The first six specimens failed very abruptly, and in general, the fracture was located in the weld or in the immediate heat-affected zone of the weld. Specimens 7 and 8 showed much better yielding of the flange section at failure. During cyclic testing, some slipping of the bolts was evident from the emitted noise on the first six specimens where A325 high-strength bolts were used. The slippage could not be detected in this manner on Specimens 7 and 8, where A490 bolts located closer to the beam center were used; and most likely the faying surfaces were better.

In general, the hysteretic loops exhibit a considerable amount of strain hardening of the material, and usually are very stable. Some of the above observations are summarized in Table 1.

Very little inelastic behavior was observed in the beams in the first six experiments. Yielding was largely confined to the panel zones. Some beam as well as panel zone yielding occurred in the last two specimens. Generally, stiffeners showed yielding near the column flanges (an example in Fig. 33). Moreover, during some tests it could be observed the stiffeners buckled in the regions adjoining column flanges. Measurements of the longitudinal stiffener elongations essentially confirm this observation (see Fig. 19 and another example in Fig. 34). From these two illustrations it is evident stiffeners yield when beam moments are in the strain-hardening range.

Analytical Comparisons

By examining Columns 5 and 6 in Table 2, it can be concluded that the panel zones are very well designed for Specimens 3, 4 and 5 according to the current SEAOC Recommendations.[8] The shear capacity of the panel zone for Specimen 6 is less than permitted by the code, and for



(a) *(b)*

Fig. 32. Specimen 8a during test; 8b after failure

Table 1. Specimen Characteristics at Failure

Specimen No.	Column Web Thickness in.	Doubler Plate Thickness in.	Stiffener Plate Thickness in.	Maximum Attained Tip Load P_u kip	Type of Failure
1	3/8	3/16	None	60	Abrupt failure through weld
2	3/8	3/16	1/4	71	Failure through HAZ*
3	11/16	3/8	None	125	Abrupt failure through weld
4	11/16	3/8	1/2	140	Stiffener weld failed first**
5	11/16	3/8	1/2	112	Repaired specimen; abrupt failure through weld
6	11/16	None	1/2	106	Failure through weld; large panel distortion
7	9/16	3/8	1/2	128	Ductile failure outside HAZ*
8	9/16	3/8	1/2	132	Ductile failure outside HAZ*

*HAZ = Heat-affected Zone
**See text on Specimen 5

Table 2. Comparison of Joint Capacities

Specimen No. (1)	P_{36} kips (2)	P_y kips (3)	P_u kips (4)	$V_{20.0}$ kips (5)	V_{p24} kips (6)	$V_{27.5}$ kips (7)		V_{p36} kips (8)	Θ_p* % (9)	Θ_p** % (10)
1	41	53	60	202	152	278	>	227	0.6	1.8
2	41	53	71	202	152	278	>	227	0.2	5.3
3	98	103	125	406	402	559	<	603	0	2.2
4	98	103	140	406	402	559	<	603	0	2.7
5	98	103	112	406	402	559	<	603	0	0.2
6	98	103	106	263	402	362	<	603	0	2.5
7	76	92	128	413	297	568	>	445	0.7	5.2
8	76	92	132	413	297	568	>	445	1.2	5.8

P_{36} = calculated cantilever tip load at 36 ksi max. beam stress
P_y = as above at measured yield
P_u = Measured ultimate tip load
$V_{20.0}$ = calculated panel shear at stress of 0.4 F_y = 20 ksi; similarly, $V_{27.5}$ is for 0.55 F_y = 27.5 ksi
V_{p24} = panel shear in kips for two flanges at 24 ksi; similarly, V_{p36} is at 36 ksi
Θ_p* = inelastic (plastic) beam rotation at P_u
Θ_p** = inelastic beam rotation + rotation due to panel distortion at P_u
The slope of angle Θ_p is measured in percent.

Fig. 33. Evidence of stiffener yielding next to column flanges shown by cracked whitewash—Specimen 8

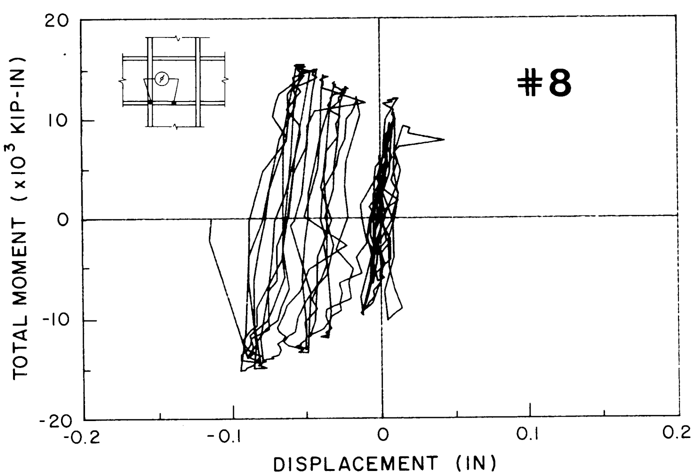


Fig. 34. Total applied moment vs. displacement between a column flange and column centerline at bottom of beam—Specimen 8

Specimens 1, 2, 7 and 8 it is more than required. These were deliberate choices in this experimental program.

Comparing the specimen panel shear capacities at yield with the shear, which could be applied by two yielding beam flanges, one finds a more accurate estimation during yielding. This comparison can be made by examining Columns 7 and 8 in Table 2, which shows that the shear capacities for the panel zones of Specimens 1, 2, 7 and 8 are adequate whereas for the others they are deficient. To this, one must add that a decrease in the panel zone shear capacity on the order of 9% would be reasonable due to the presence of the axial force. Therefore, it is evident the panel zones would yield in Specimens 3, 4, 5 and 6 before yield occurs in the beam flanges. The use of actual, rather than code stress, except for added numerical complexities, leads to the same conclusions.

To gain an insight into the consequences of yielding first in the panel zones, Columns 9 and 10 in Table 2 were developed. The inelastic beam rotation together with the inelastic rotation caused by the panel zone deformation designated by $\Theta_p{}^{**}$ is given in Column 10. For each specimen, the total maximum displacements from the zero intercept, such as point C in Fig. 8, to the maximum, point D, was averaged with the corresponding data points F and E. The elastic displacements corresponding to points A and B were then deducted from these total displacements. By dividing these net quantities by the cantilever lengths, one obtains the maximum $\Theta_p{}^{**}$.

By subtracting from $\Theta_p{}^{**}$, the panel zone rotation γ_{max}, corresponding to points such as A and B and/or C and D in Fig. 9, one obtains $\Theta_p{}^*$—the inelastic beam rotation. The values of $\Theta_p{}^*$ so found are approximate to the extent that it is assumed that the top and bottom edges of the panel zone are horizontal. In reality, they rotate slightly, thereby increasing $\Theta_p{}^*$. However, this being essentially an elastic rotation, the admitted inaccuracy is believed not significant.

As can be seen from Column 9 in Table 2, the inelastic beam rotations are small. In fact, some of the zero entries were small negative values caused by the approximation noted in the above paragraph and by an accumulation of measurement inaccuracies.

Note, when the shear capacity of the panel zones were overdesigned according to the current SEAOC Recommendations, as for Specimens 1, 2, 7 and 8, the inelastic beam rotations increased.

Since in actual design of moment resisting frames it is essential to achieve a $\Theta_p{}^{**}$ of at least 2% times a reasonable factor of safety, the behavior of Specimens 2, 4, 7 and 8 may be considered satisfactory. Specimen 4 is included in this group assuming that good quality stiffener welds can be achieved on a job.

Conclusions and Code Implications

The objectives of verifying the design criteria for the prototype were achieved by this experimental and analytical program. These criteria require the use of stiffeners and doubler plates as well as a careful control of the welding process. It should be emphasized, all beam connections had bolted webs and full-penetration flanges welds. Since the obtained results have general implications for steel design, these are summarized here in an effort to help improve upon current practice:

Stiffeners—Even with the column flanges 1¼-in. thick, it was found stiffeners were essential. Moreover, since beam flanges at the connections strain-harden during severe cyclic loading, and the stiffeners have a propensity to buckle and/or yield, selection of the stiffeners on the basis of nominal yielding in beam flanges is unconservative.

Moreover, an indication that no stiffeners are required by the current code may be misleading for the type of beam-column joints reported herein. Full-penetration welding of the stiffeners to column flanges, rather than fillet welds, is desirable.

Panel Zones—The current code provisions for sizing the thickness of the panel zones appear to require further review. There is a very delicate balance as to how much inelastic rotation must be taken by the beams, and how much may be permitted to be taken by the panel zones. This is governed by the shear strain in the panel zone measured by the angle γ which creates severe kinks right at the beam flange welds (see inserts in Figs. 9, 12, 18, etc.). The induced severe local curvature in a column flange together with the longitudinal beam flange stress tend to cause cracking of the flange welds. For well behaving panel zones the angle γ can reach a magnitude on the order of 2°. A modest increase in panel zone thickness over the currently stipulated code provisions improves the ductility of joints by forcing the development of inelastic rotation in the beams. This is evident from tests on Specimens 2, 7 and 8. In that regard, extending the doubler plates above and below a beam as shown in Fig. 4 was effective. The large plug welds used to attach doubler plates to column webs were very satisfactory.

Lastly, it should be noted, that for geometries of the tested column cross sections the flanges were relatively thin. As the thicknesses of the column flanges increase, requirements for the stiffener sizes, as well as the thicknesses of the panel zone doubler plates, would likely decrease.

Future Research Needs

The experimental work described in this paper was directed toward a study of particular geometric parameters of the joints. Moreover, the study was limited to connections to column flanges. It seems most desirable to continue this type of an investigation to include other geometries of members and types of connections. Extending the studies to columns with thicker flanges and narrower panel zones, as well as to tubular columns, may lead to different conclusions.

The information presented in this paper should not be relied upon without careful verification and scrutiny by the user.

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