Capacity of Columns with Splice Imperfections

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During the erection of a structure, the milled or cut surfaces at column splices may not make perfect full and even contact. AISC has pointed out some of the reasons for these column splice imperfections:[1]

"1. The most advanced milling techniques utilized with appropriate shop layout to theoretical center line will realistically result in some measurable deviation from perfection.
2. The accuracy of substructure levels and elevations is limited and is beyond the control of the fabricator.
3. Erection techniques involve measurable movement from theoretical dimensions to compensate for permissible individual member tolerances in achieving overall compliance to plumb, level, and line.
4. Erection techniques in field welded structures may deliberately cause the columns to be tipped slightly out of plumb just prior to welding beams to columns in order to compensate for weld shrinkage."

Recognizing these possibilities, AISC[1] accepts a maximum gap of $\frac{1}{16}$-in. without shims. For larger gaps, the use of non-tapered mild steel shims to pack out the gap is required. In either case partial penetration welds or bolted connections across both flanges are used to join the members together. The finished spliced members must conform to specified tolerances.[2]

Except for work by Hayes, who investigated bolted splice connections using small column sections,[3] data on the behavior of full-size columns having imperfect contact at the splices are not available. Therefore, it was decided to conduct a modest investigation to provide some needed information on welded splices.[4] Since it is known that the initial imperfections and residual stresses are important factors influencing the behavior of columns,[5,6] in addition to the column compression tests, careful initial measurements of the specimens were made and the residual stresses determined.

Because very little data are available on the behavior of spliced columns subjected to tensile forces simulating situations which may develop in an earthquake, all of the spliced specimens were tested to failure in tension after first having been subjected to large compressive loads.

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GENERAL DESCRIPTION

The tests were conducted using W14×176 shapes of A36 material, due to their common use in buildings. Five square-ended specimens were fabricated and tested; four of these were approximately 14 ft-8 in. long and one was 10 ft long. Four of the five specimens were spliced at mid-height to obtain the most critical condition for buckling, with various sizes of gap introduced about the two major axes. It is important to note that these wedge shaped gaps were achieved by first cutting the unspliced specimen in half and then cutting one of the faces to be rejoined at a slightly oblique angle with the longitudinal axis of the member before welding the two members together. This eliminated any large discontinuity at the splice and maintained the overall initial camber and sweep requirements.

The wedge shaped gaps ranged from a full contact condition on one side of the splice to the desired gap width on the other. Two of these intentionally introduced gaps were made to occur about the weak axis and two about the strong axis. Specimens No. 1 (weak axis splice; see Fig. 1) and No. 3 (strong axis splice; see Fig. 2) had maximum gap widths of $\frac{1}{16}$-in. and no shim was used to pack out the splice. A short length of $\frac{1}{16}$-in. diameter wire was used to maintain the desired $\frac{1}{16}$-in. gap during the welding process. Specimens No. 2 (weak axis splice; see Fig. 1) and No. 4 (strong axis splice; see Fig. 2) had maximum gap widths of $\frac{1}{4}$-in., and $\frac{1}{16}$-in. flat shims of A36 steel were used to pack out the splice. The procedure used for installing shims differed from standard field practice, as it was not feasible to drive in the shims between the unrestrained halves of the specimen in the fabricating shop. Shim material was placed

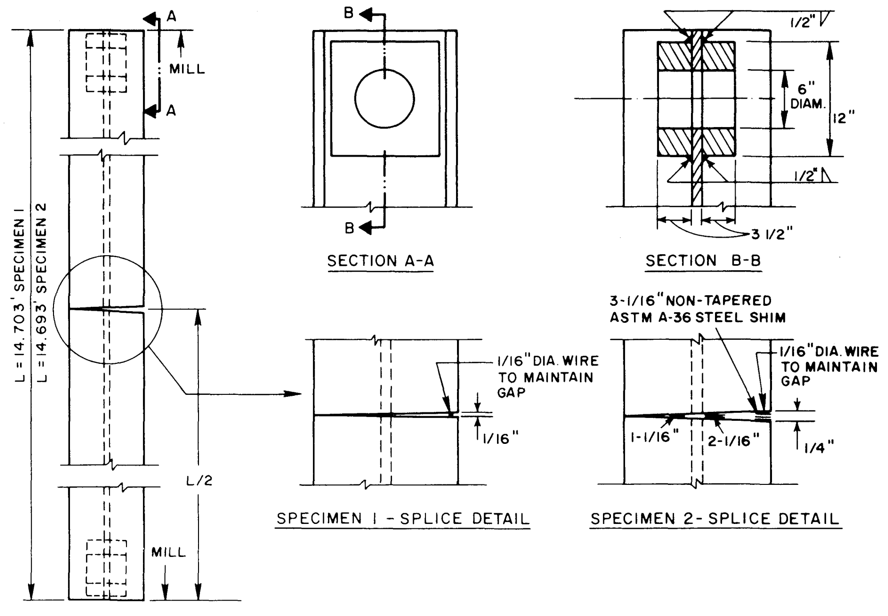

Fig. 1. Details of Specimens No. 1 and No. 2

in stacks at discrete locations in the joint to maintain the required gap. Again, a short length of $\frac{1}{16}$-in. diameter wire was used to maintain this desired $\frac{1}{4}$-in. gap during the welding process. Specimen No. 5 was left unspliced and treated as the control member. In the process of fabricating Specimen No. 3, two 2 ft-4 in. pieces were removed from the center of the member, to be tested as a stub column and for residual stress measurements. The remaining two 5 ft-0$\frac{1}{4}$ in. long segments were then spliced together as described above, producing a column approximately 10 ft long.

The splices in each specimen were maintained by having $\frac{7}{16}$-in. partial penetration groove welds on the outer face of both flanges. The weld size was the minimum permitted for the thickness of material, according to the AWS Code in effect at the time the specimens were fabricated. No welding was done on the inside face of the flanges or on the web at the splice. The average measured weld sizes, de-

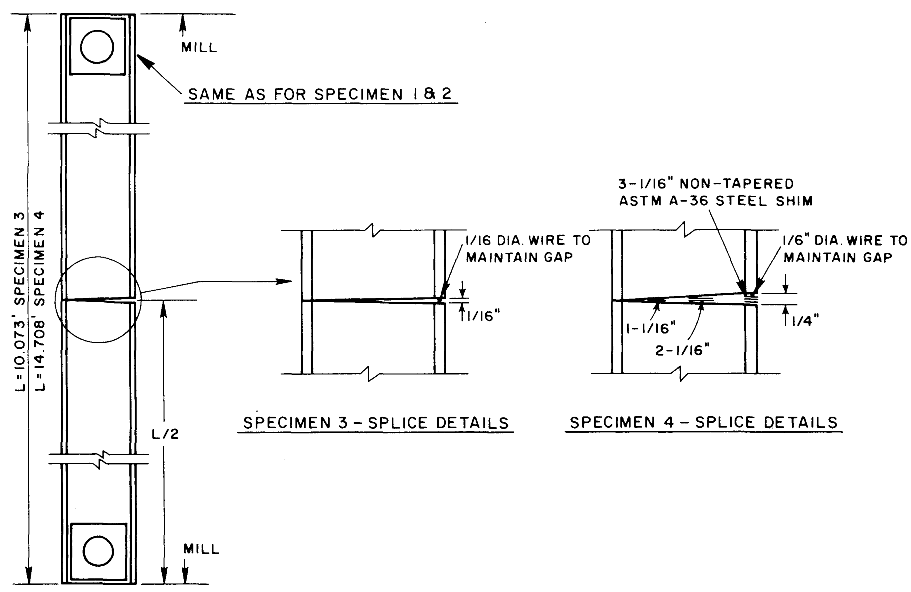

Fig. 2. Details of Specimens No. 3 and No. 4

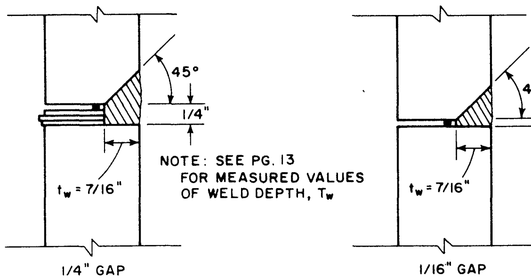

Fig. 3. Flange weld detail of splice

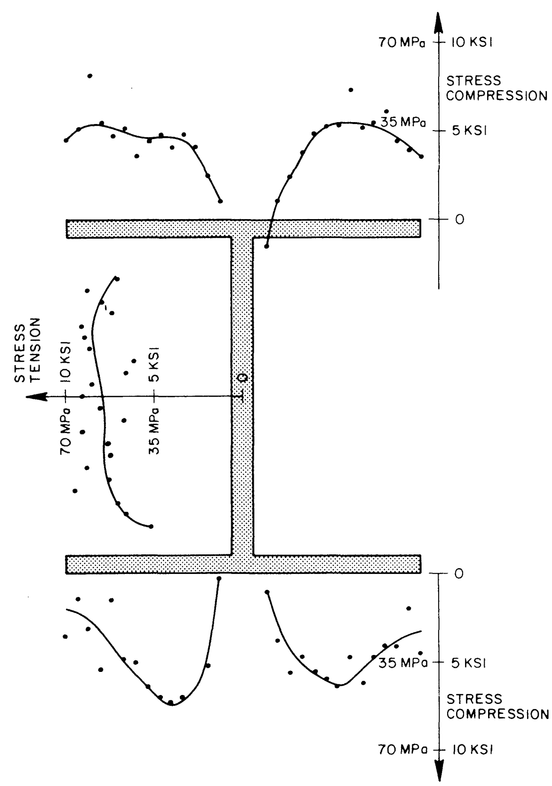

Fig. 4. Results of residual stress measurements

termined after the column tension tests were completed, are noted in Table 1. Gap weld details are shown in Fig. 3.

Once the specimens were spliced, the ends of each were milled at 90° to the longitudinal axis. In addition, Specimens No. 1 through No. 4 were fitted with end fixtures to accommodate pins for carrying out the tension tests. After the compression test, similar plates were added to Specimen No. 5.

Careful measurements were taken of each specimen to determine the section dimensions. It was found that measurements compared very closely with the dimensions given by AISC.[2] Measurements also were taken to determine the initial camber and sweep of the five specimens. These results indicate that the maximum camber and sweep did not exceed $\pm 1/8$ in.

TESTING PROCEDURE

Residual Stress Measurements—The residual stress distribution was determined by the sectioning method from the 2 ft-4 in. length of column cut from Specimen No. 3.[7] The residual stress pattern is shown in Fig. 4.

The general residual stress distributed does not follow the typical, approximately parabolic shape given by Tall.[8] This is partly because the W14X176 is a heavy shape, much heavier than any considered by Tall. However, the most important information to be gained from Fig. 4 is that the shape was apparently straightened. The dip in compressive residual stresses towards the flange tip is typical for straightened shapes. The values of E used to convert all strain readings to stress was taken as 30 x 10^3 ksi.

Coupon Tests—Following the residual stress determination, nine ½-in. thick slices of column material were chosen to determine the average material properties over the cross section. All specimens were approximately 12 in. in length. Six of these were from various flange locations, while the remaining three were taken from the web. The specimens were originally 12 in. x 1.2 in. x 0.5 in. strips and were machined into tension test coupons in accordance with ASTM A370. The gage length varied from 2 in. to 4 in.

Table 1. Summary of Test Results

Specimen No.	Splice Type	Length (ft)	Maximum Compressive Force (kips)	Maximum Center Deflection (in.)		Ultimate Tensile Force (kips)	Average Weld Size (in.)	
				Weak Axis	Strong Axis		North Side	South Side
1	Weak Axis Min 1/16 in. Gap	14.703	1,650	3.25	0.12	532	0.50	0.48
2	Weak Axis Max. 1/4 in. Gap	14.693	1,655	3.00	0.07	800	0.54	0.63
3	Strong Axis Min. 1/16 in. Gap	10.047	1,960	0.47	0.05	1,070	0.46	0.43
4	Strong Axis Max. 1/4 in. Gap	14.708	1,760	3.50	0.86	200	0.61	0.29
5	Unspliced	14.646	1,675	4.70	0.50	1,800	—	—

The yield point varied over the section as expected, and is consistent with results obtained by Tall.[8] The web exhibited the highest average yield stress of 36.5 ksi, while the flange material had an average of 33.9 ksi. The extreme edge of the flange had a yield stress of 38.0 ksi. The ultimate tensile stress values averaged 65.5 ksi with extremes of 63.5 ksi and 67.0 ksi. The mill test report indicated a yield point of 39.7 ksi and a tensile strength of 68.4 ksi.

Compression Tests of Columns—The column compression tests as well as the tension tests were performed in a 4,000,000-lb Southwark-Emery Universal testing machine. In the column compression tests the longitudinal axis of each specimen was aligned with the axis of the testing machine. Aluminum plates approximately ⅛-in. thick, along with steel shims, were used at the top and bottom bearing surfaces, where it was determined that insufficient contact between column and testing machine head existed. The combination of bearing plates and steel shims resulted in uniform strain readings (within ±5%) over the mid-height cross section under loads of 400 to 800 kips. The swivel head of the testing machine was locked in a horizontal alignment to prevent its rotation and this, in addition to the square ends of the columns, produced essentially fixed end conditions for the specimens.

The instrumentation for each specimen consisted of the following: (1) Four SR-4 type strain gages, two on the outside of each flange, each set 4 in. from the edge and 15 in. away from the splice at mid-height. In the unspliced column these were placed at mid-height. (2) Four linear potentiometers, two on each flange directly over the splice, set 1 in. from the edge with 10-in. gage lengths. (3) Seven linear potentiometers; three were placed at the quarter-points of the column specimens to measure lateral motion in the weak axis direction. Three others were placed at quarter points to measure lateral motion in the strong axis direction. The seventh potentiometer was used to monitor the axial shortening. The test arrangement is shown in Fig. 5. In both the elastic and inelastic range the loading was applied in increments. In the elastic range the loads were applied in 200 kip increments up to 1000 kip and thereafter this loading was reduced to a minimum of 25 kip increments. When the loading was in the inelastic range, the load was allowed to stabilize before any strain or deformation readings were taken. Testing was continued until either deformations were sufficient to completely define buckling or until the load reached a maximum point and began to decrease.

Tension Tests of Columns—After each specimen was tested in compression, and thus had sustained considerable lateral deformation, tension was applied until failure took place. The pin ended fittings allowed the rotations of the longitudinal axis as the axial loads were applied. Mid-height lateral displacements and axial deformations were the only instrumentation used during the tension tests.

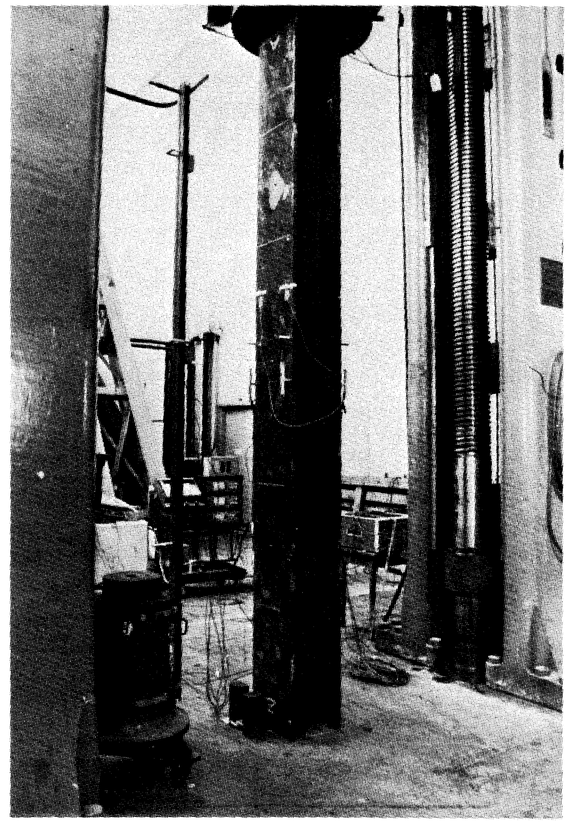

Fig. 5. Typical compression test arrangement

TEST RESULTS

Principal Results—The load deflection characteristics in compression for Specimens 1, 2, 3, 4, and 5 are shown in Figs. 6 through 10, respectively, and a summary of the principal test results is given in Table 1. A typical buckled configuration of a column in an advanced stage of testing is shown in Fig. 11. Local buckling of the type shown in Fig. 12 occurred in all spliced specimens. All local buckles were confined to 14 in. on either side of the splice.

The results of the tension tests are shown in Figs. 13 through 17 as hysteresis loops. The initial compressive loading curves have been replotted, and the tensile load-deflection characteristics added to the diagram. Since the boundary conditions for the two types of tests are different, these results must be carefully interpreted.

It should be noted that besides the overall buckled shape, the specimens also had significant local buckles in the flanges at the splices. Measurements were taken after the welds had failed, to determine their actual widths. Pertinent data for these tension tests, including fracture load and average weld thickness, are included in Table 1.

The unspliced specimen, No. 5, also was tested in tension, and reached an axial force of 1800 kips. An abrupt failure took place at this load, causing fracture at the pin-plates which had been added after the compression test in order to salvage an otherwise excellent specimen.

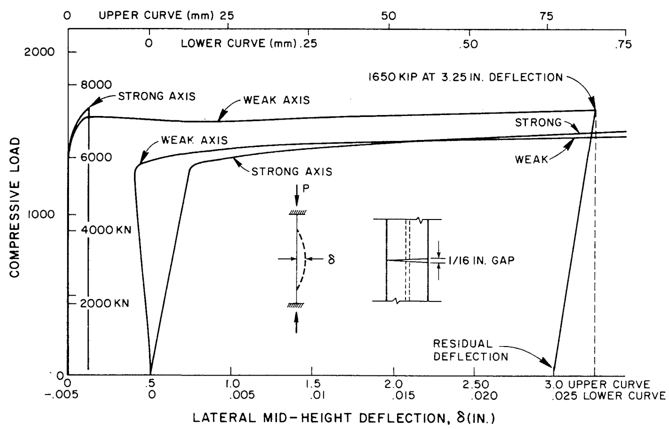

Fig. 6. Load-deflection for Specimen No. 1

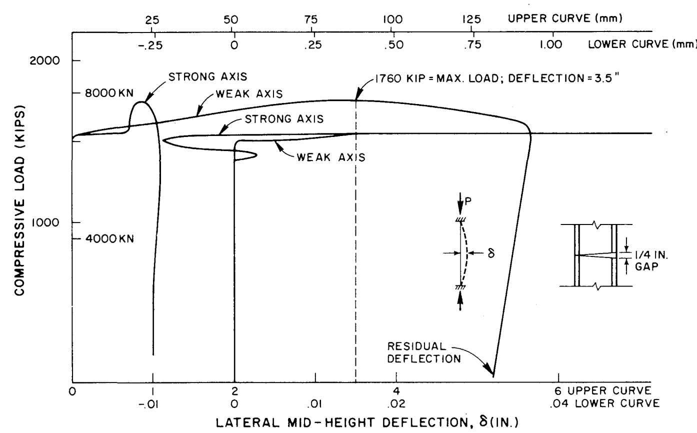

Fig. 9. Load-deflection for Specimen No. 4

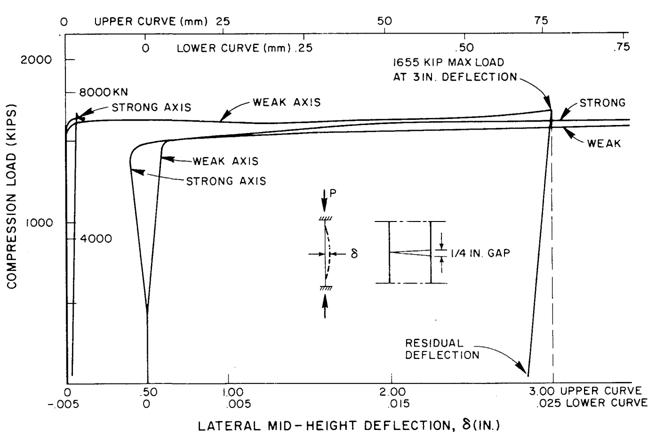

Fig. 7. Load-deflection for Specimen No. 2

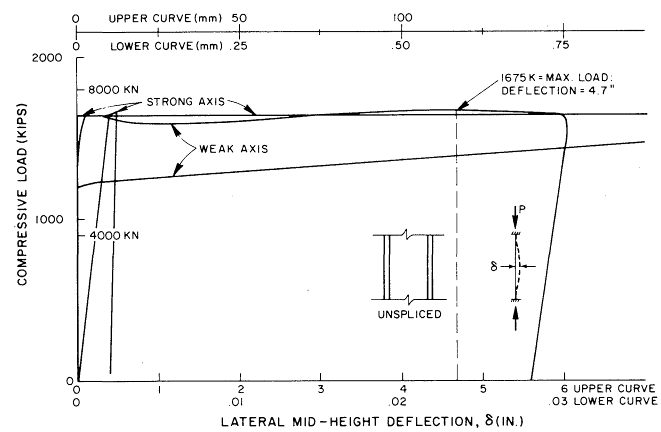

Fig. 10. Load-deflection for Specimen No. 5

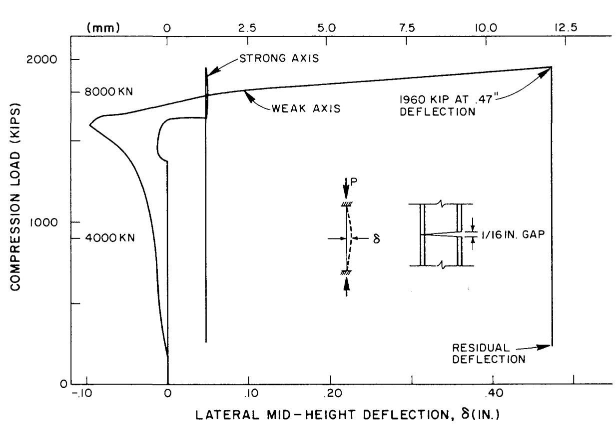

Fig. 8. Load-deflection for Specimen No. 3

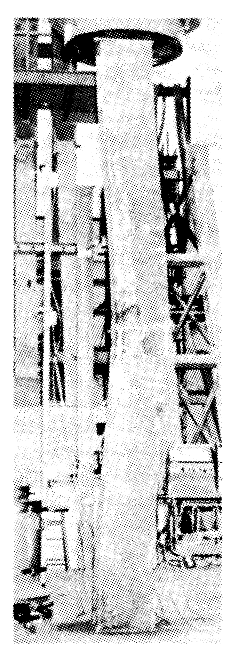

Fig. 11. Typical column buckle configuration

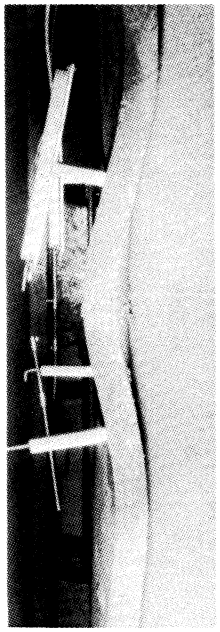

Fig. 12. Typical local buckle

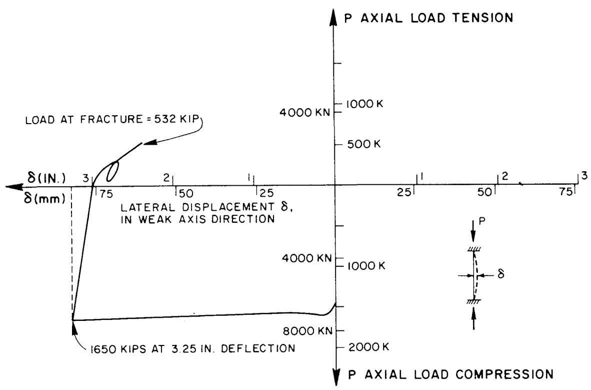

Fig. 13. Load-deflection hysteresis for Specimen No. 1

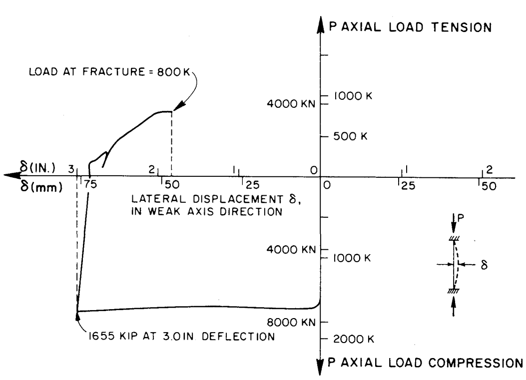

Fig. 14. Load-deflection hysteresis for Specimen No. 2

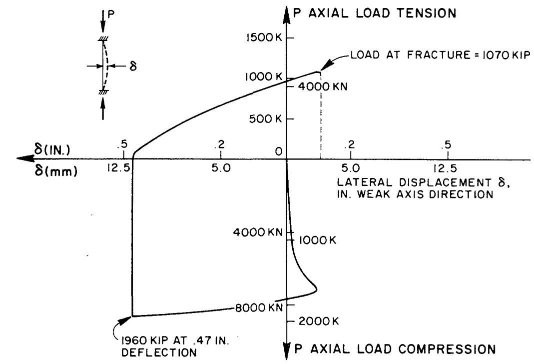

Fig. 15. Load-deflection hysteresis for Specimen No. 3

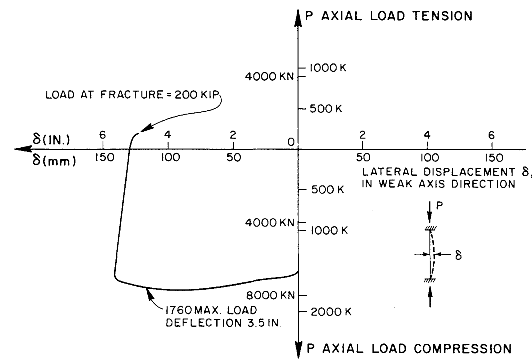

Fig. 16. Load-deflection hysteresis for Specimen No. 4

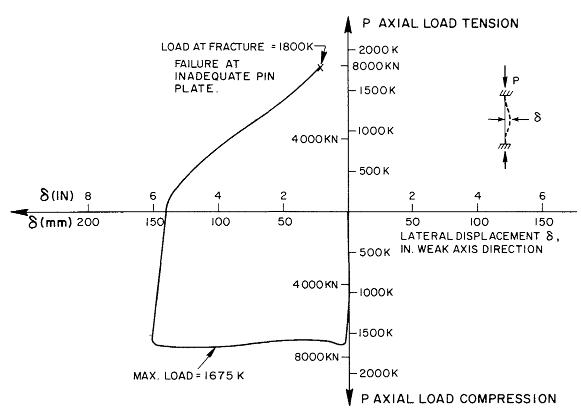

Fig. 17. Load-deflection hysteresis for Specimen No. 5

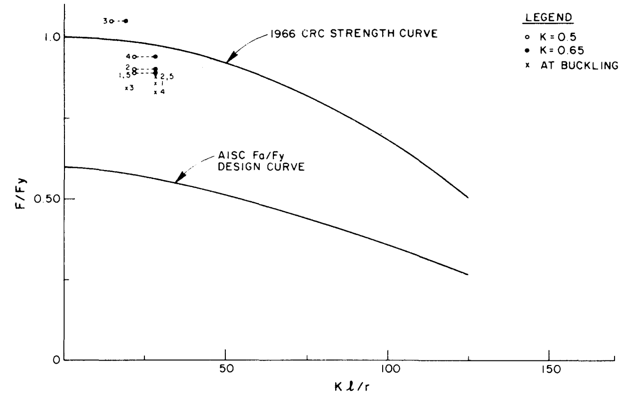

Fig. 18. Theoretical and experimental column strengths

Discussion of Results—Figure 18 compares the performance of the five test specimens with theoretical column strengths as predicted by the CRC Curve.[5] It is important to note that the maximum compressive force does not necessarily occur at the maximum deflection.

In reviewing the data in Table 1 for the column compression tests, it is seen that the maximum compressive forces, for Specimens 1, 2, 4, and 5, which have the same slenderness ratios, are essentially the same. This is clearly brought out by the data points in Fig. 18.

The columns were tested in a "flat-ended" condition, and during the application of the compressive load the head of the testing machine was locked in position. This testing procedure tends to make the column behave as something close to a fixed-end member, which may be noted from Fig. 11. However, complete fixity of large members is very difficult to achieve. The columns had square milled ends, but required shimming to achieve uniform bearing against the testing machine platens. For this reason, two sets of points corresponding to the experimentally determined maximum load points are plotted. For the one set K is assumed to be 0.5, and for the other it is taken as 0.65. It is believed the latter value is more representative of the conditions of these experiments.

It can be seen from Fig. 18 that there is only approximately 7 percent difference between the lowest failure load and the highest. The higher strength of Specimen No. 3 was due to its lower Kl/r ratio, yet on the dimensionless plot of Fig. 18 the results are in reasonable conformity with those for the other four specimens. Because of the closeness of the test points, it is difficult to discern any pattern or correlation between the direction of the splice (weak or strong axis), the size of the gap, and the column strength.

The loads at which each specimen had sustained a 0.10-in. weak axis lateral deflection was defined as the column "buckling" load, and the corresponding values are shown in Fig. 18 by the symbol "x." This arbitrary buckling criterion was chosen on the basis of the behavior of Specimens Nos. 1, 2, 4, and 5, as seen in Figs. 6, 7, 9, and 10. This "buckling" load represents a point beyond which the load may increase, but with corresponding large lateral deformations. It is difficult to define a similar point for Specimen No. 3.

Several factors contributed to the high values of column buckling load in these tests, especially as this pertains to the spliced columns. Of most importance, the Kl/r ratios were small, which tended to reduce the effect of splicing. Moreover, the specimens tested were quite straight. The maximum initial sweep was 0.094 in. ($\approx l/2000$) at center span in Specimen No. 2, and the average initial sweep was 0.05 in. ($\approx l/3600$). This is considerably less than the deviation from straightness allowed by the AISC Specification ($l/1000 = 0.18$ in.).

In reviewing the tension test data, the most interesting result was the widely varying loads at fracture. Table 1 gives the fracture loads and the average weld sizes, which were measured after fracture occurred. The spread in fracture loads was from 200 kips for Specimen No. 4 to 1070 kips for Specimen No. 3. Some of this discrepancy can be attributed to the extent of overall and local buckling which developed at the splices during compression, as well as the actual size of the welds and their general quality.

CONCLUSIONS

The results of these tests indicate that the lack of perfect contact at compression splices of columns may not be important, provided that the gaps are shimmed and welding is used to maintain the sections in alignment. It is noted that the spliced columns, rather independently of the gaps introduced at mid-height, behaved very much like the unspliced column, and their column strengths were essentially the same. It must be pointed out, however, that the Kl/r ratio of the columns tested was low, being on the order of 30. A slenderness ratio in the range 25–30, however, is not at all uncommon in actual buildings. It may be desirable to extend the research to include columns with higher slenderness ratios, to determine the effect of the splicing on a wider range of columns. The significance of the splice location for longer columns than those tested may have to be considered.

The data obtained on the behavior of initially buckled members, subsequently loaded in tension, should prove useful in evaluating the behavior of such members in aseismic design. Columns and braces in buildings, as well as in offshore structures, may buckle in compression under extreme excitation and then become subjected to tensile forces. Some data from experiments simulating this type of behavior on small members are available.[9] This investigation provides data on the behavior of full-size members. It is to be noted that for the two loading conditions of compression and tension, the boundary conditions were not identical.

If the designer anticipates tensile forces developing in the spliced columns, one must be certain that the specified weld size is obtained. As ultrasonic inspection at partial penetration welds is questionable, visual inspection at first root pass is recommended.

ACKNOWLEDGMENTS

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