Performance of Large Seismic Steel Moment Connections Under Cyclic Loads

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ABSTRACT: Cyclic behavior of recently completed 18 tests in full-size beam-to-column steel moment connections is described. Main emphasis is placed on moment connections of beams to column flanges having plastic moduli of the flanges alone less than 70 percent of the respective plastic moduli of the total beam section. Also considered, cyclic behavior of improved beam-to-column web moment connections as well as some unconventional beam-to-column flange moment connections.

INTRODUCTION

Seismic design of steel structures can now be carried out either on the basis of allowable stress design (ASD)[1,2,3] or load and resistance factor design (LRFD).[4] The second approach utilizes concepts of ideal plastic capacity of members as well as load and resistance factors. Whereas both methods permit elastic methods of analysis for determining member forces, under LRFD it is also permitted to determine such forces using methods of simple plastic analysis. Unfortunately at the present time few engineers take advantage of plastic analysis. Yet of all situations encountered in structural engineering practice, plastic analysis of frames in *seismic design* is the next logical step to take. Thereafter should come a more accurate appraisal of strain-hardening of steel to provide the designer a still better appraisal of the behavior of a structure. As matters now stand, whereas the need for ductility in seismic design is clearly recognized, the extent of the amount of such ductility is at best only roughly approximated.

Nevertheless ductility is central to all current seismic design provisions. Based on limited information, and largely arrived at from observations in the wake of major earthquakes, the codes assign *large* lateral load reduction factors (R in Ref. 5, and R_w in Ref. 2) for structural framing systems that are ductile. These factors are used to modify the estimated credible seismic loads to obtain design forces. Since steel is intrinsically a ductile material, for moment resisting frames, Ref. 5 assigns an $R = 8$, and Ref. 2 an $R_w = 12$. The difference between these modification factors is due to the difference in the allowed stress levels in ASD[2] and in NEHRP.[5] Inasmuch as these factors are large, the design seismic forces become *much smaller* than what can be expected in a *major* earthquake. Therefore the use of these factors makes it mandatory to demonstrate not only the maximum strength but also the *ductility* of a system. The anticipated ductility of a structural system plays a major role in establishing reduction factors. For this reason the main purpose of the experimental program described in this paper is to provide additional data on realistic member sizes and the extent of cyclic ductility that can be attained with moment connections commonly used in moment resisting steel frames.

In interpreting these tests it is important to recognize that the technology of joining members together is continuously changing, and that modern steel frames act virtually alone in resisting seismic forces. The columns are no longer fireproofed with concrete, and cladding as well as interior partitions have little structural value.

SPECIMEN SELECTION

All specimens were fabricated as cantilevers attached to column stubs. Thirteen of the beam specimens were connected to column flanges; five to column webs. The beam and column sizes used are given in Table 1, where the ratios of the plastic flange moduli Z_f to the respective plastic beam moduli Z are given in parentheses in the second column. The cantilever lengths for all specimens were about 60 in., requiring no change in location for the actuating hydraulic cylinder, Fig. 1.[6] The point of load application on the cantilever corresponds to the inflection point in a beam of a frame. During a seismic event these inflection points do not remain fixed in position. For beams in a perimeter frame, and other situations where the gravity load is small, the inflection point is not likely to move very much and remains approximately in the middle of a span. However, for beams carrying a substantial gravity load, such inflection points may move a great deal. For short cantilevers the inelastic action tends to be localized next to a column and therefore is more severe. The choice made for the cantilevers used in these tests is believed to be a reasonable compromise among the conflicting requirements.

Some of the specimens were fabricated as shown in Fig. 2, where one of the test specimens was attached to the column flange, and the other to the column web. Specimens 1, 2, 7, and 8 were fabricated in the University shop using shielded metal arc welding (SMAW). The remainder of the

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specimens were made up in two different local steel fabricating shops. Specimens 3, 4, 5, and 6 were fabricated in one shop, and Specimens 9 through 18 in another. A number of specimens furnished by venders were fabricated using flux-cored arc welding (FCAW). A few minor modifications to the specimens were made in the University shop.

Most of the beam-to-column flange connections were of conventional type with bolted webs and welded flanges, such as recommended in Fig. CF-1 on p. 10 of Ref. 7. On several of the specimens supplementary welds of shear tabs to beam webs per SEAOC provisions[2] were added at the University shop. Generally these welds provided the required 20 percent of the web plastic moment capacity. However only 10 percent of such welding capacity was added for Specimen 5. The Lohr Fastener System, consisting of bolts with twist-off ends for tension control in web bolts, was used on Specimens 15, 16, 17, and 18. The number and type of bolts for the connections is given in Table 1. Rib reinforcement at the column face, similar to that described later for Specimen 1, was used for Specimen 7.

In 1968 tests, six cyclic experiments on welded beam-to-column web moment connections were performed on small specimens having flange details as shown in Fig. 3. In these specimens both beam flanges and webs were welded to the connecting plates.[8] The behavior of these specimens was poor as abrupt failures of the type shown in Fig. 4 occurred prematurely in most of the specimens. The specimen with the detail shown in Fig. 4(b), where the flange connecting plates extended beyond column flanges, behaved better. This was corroborated by the *static* tests described in Ref. 9 and appears to be the basis for the recommended detail in Refs. 7 and 11. In this investigation a somewhat different approach was followed, since even the specimens having details as in Fig. 4(b) appeared to be somewhat inadequate for cyclic applications. The test specimens had the following characteristics.

Table 1.
Specimen Schedule

Specimen	Beam[1] Size	Column Size	Connecting Direction	Connection Detail	No. of Connection Bolts[2]	Remarks
1	W18×40 (0.70)	W12×133	Weak	Bolted Web Welded Flg	4	Reinforcing Ribs 20% Web Weld
2	W18×40 (0.70)	W12×133	Weak	Bolted Web Welded Flg	4	–
3	W18×35 (0.66)	W12×133	Strong	Bolted Web Welded Flg	4	Flux-Cored Arc Welding
4	W18×40 (0.70)	W12×133	Weak	Bolted Web Welded Flg	4	Flux-Cored Arc Welding 20% Web Welding
5	W21×44 (0.62)	W14×176	Strong	Bolted Web Welded Flg	5	Flux-Cored Arc Welding 10% Web Weld
6	W21×44 (0.62)	W14×176	Weak	Bolted Web Welded Flg	5	Flux-Cored Arc Welding Extended Continuity Plates
7	W21×44 (0.62)	W14×176	Strong	Bolted Web Welded Flg	5	Reinforcing Ribs
8	W21×44 (0.62)	W14×176	Weak	Bolted Web Welded Flg	5	Reinforcing Ribs Load Indicating Washers
9	W18×46 (0.71)	W12×133	Strong	All Welded	NA	Partial Penetration and Fillet Flange Welds
10	W18×40 (0.70)	W12×133	Strong	End Plate	8	Fillet Welds to End Plate with Stiffeners
11	W21×44 (0.62)	W14×176	Strong	All Welded	NA	Fillet flange Weld All Around
12	W21×44 (0.62)	W14×176	Strong	End Plate	8	–
13	W18×35 (0.66)	W14×159	Strong	Bolted Web Welded Flg	4	20% Web Weld
14	W21×44 (0.62)	W14×159	Strong	Bolted Web Welded Flg	5	20% Web Weld
15	W18×35 (0.66)	W14×159	Strong	Bolted Web Welded Flg	4	Flux-Cored Arc Welding Twist-Off Bolts
16	W21×44 (0.62)	W14×159	Strong	Bolted Web Welded Flg	5	Flux-Cored Arc Welding Twist-Off Bolts
17	W18×35 (0.66)	W14×159	Strong	Bolted Web Welded Flg	4	Twisted-Off Bolts
18	W21×44 (0.62)	W14×159	Strong	Bolted Web Welded Flg	5	Twist-Off Bolts

1. *Values in parentheses give ratios of* $\frac{Z_f}{Z}$
2. *All bolts 1 in. diam. A325-X, except 1 in. diam. A354-BD for Specimens 10 and 12.*

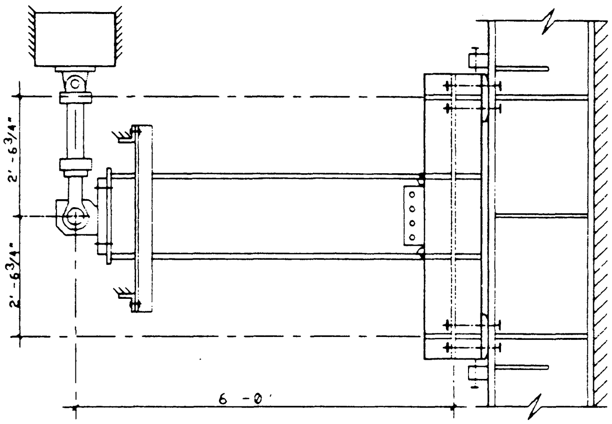

Fig. 1. Diagrammatic setup of experiments

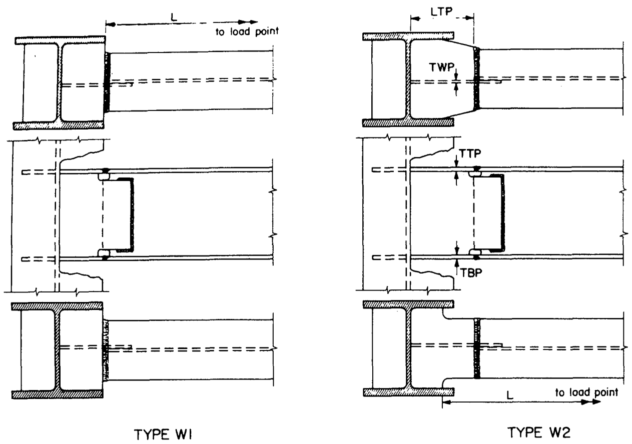

Fig. 3. Details of beam-to-column web moment connections in 1968 tests

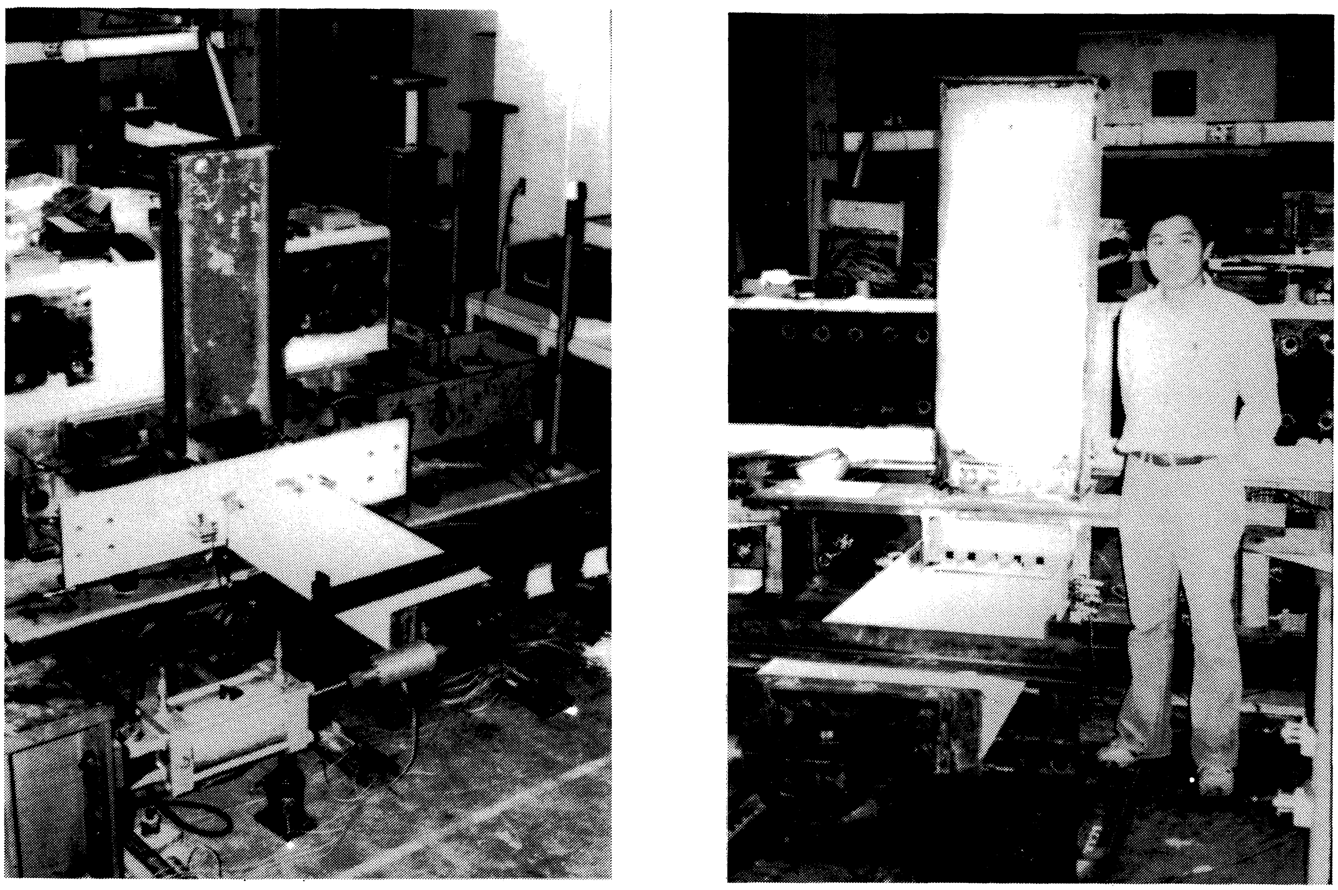

Fig. 2. Examples of specimens in testing position

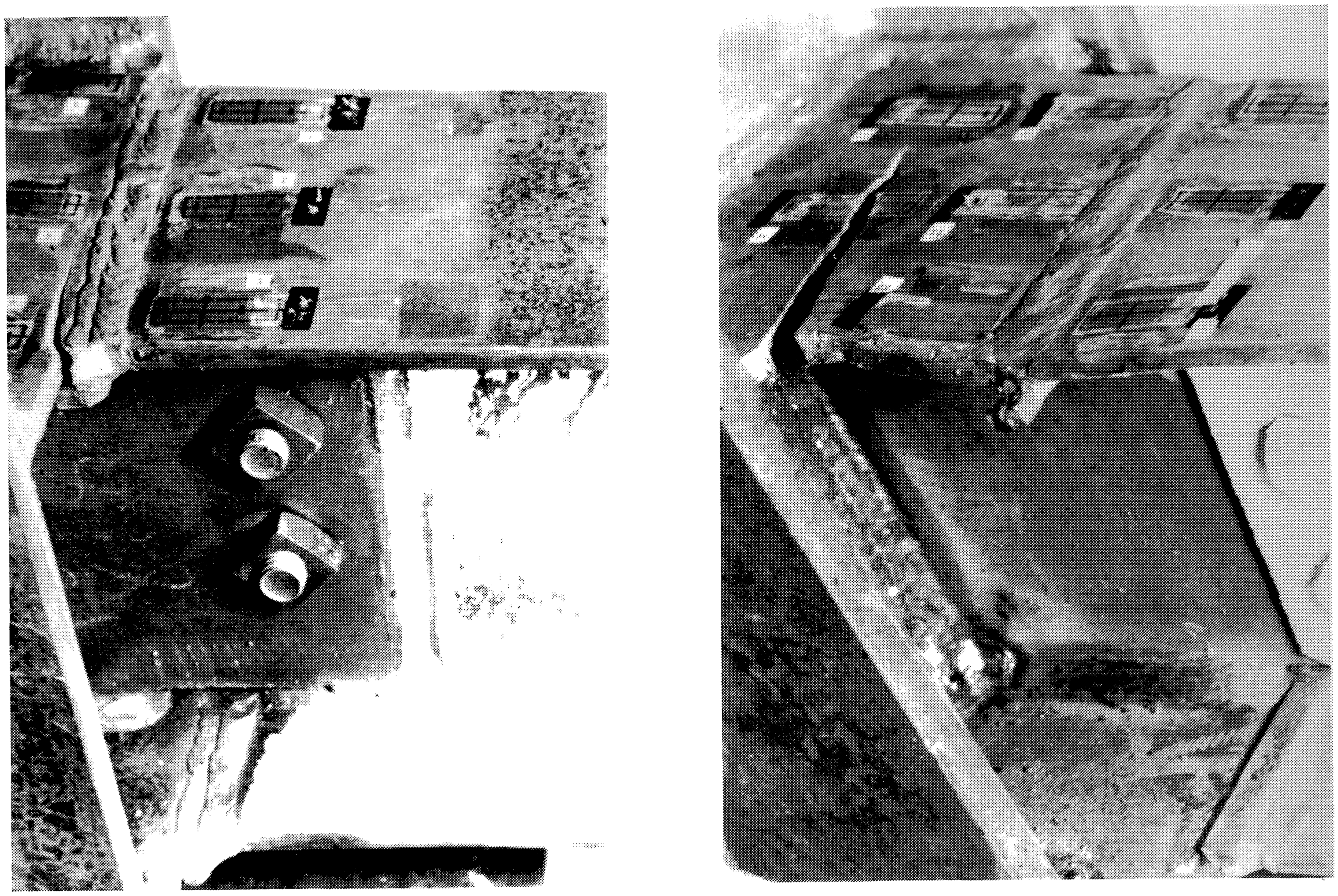

Fig. 4. Fractures in beam-to-column web all welded moment connections with simulated erection bolts

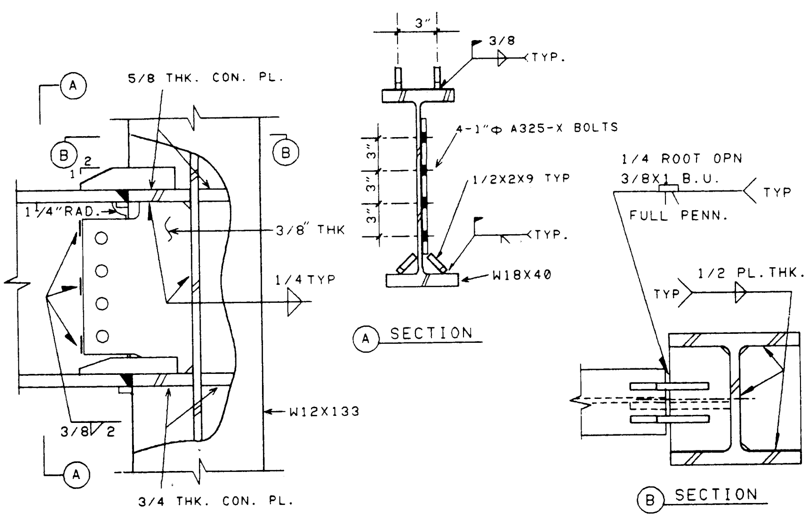

Fig. 5. Details of rib reinforcement for beam-to-column web moment connection

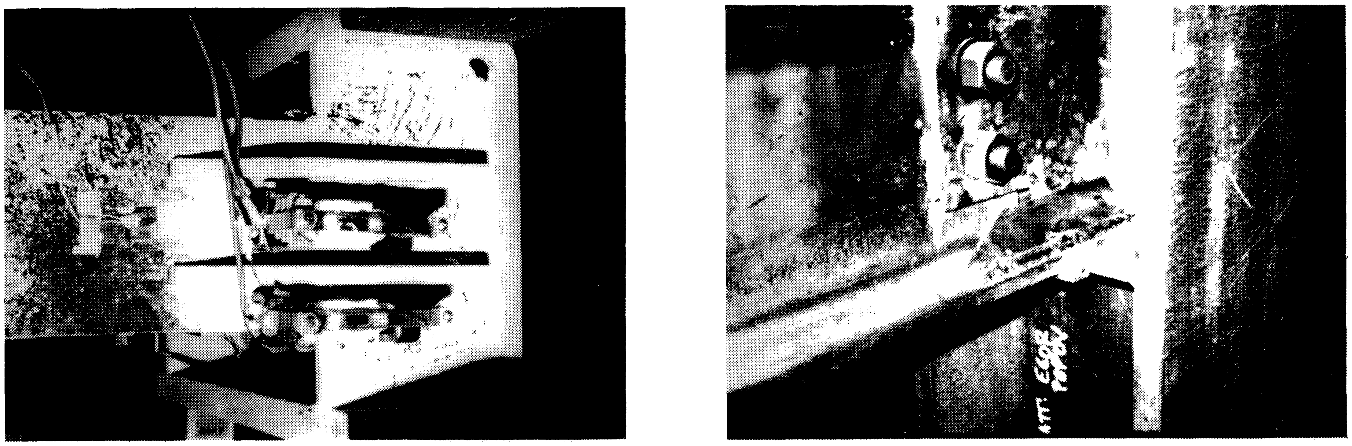

Fig. 6. Flange weld reinforcing ribs on Specimen 1

Specimen 2 represents the best in the immediate past fabrication practice of connecting a beam-to-column web. This requires the use of thicker continuity plates, both top and bottom. In this case the top plate was made ⅛ in. thicker than the beam flange, whereas the bottom one ¼ in. The additional increase in the thickness in the bottom plate accommodates possible inaccuracies in beam rolling depths. In Specimen 4 the same details were used with the connection shear capacity enhanced with a 20 percent web weld. Specimen 6 had the continuity plate extend 1 in. beyond the column flanges. The top and bottom continuity plates were, respectively, ½ in. and ¾ in. thick. These specimens were selected to provide some benchmarks for beam-to-column web connections for conventional construction under cyclic loading.

Specimens 1 and 8 were reinforced with ribs at the flange welds as shown in Fig. 5.[6] Coronet Direct Tension Indicator washers were used in Specimen 8. Note that the continuity plates were cut flush with the column flanges. The small ½×2×9 in. tapered ribs can be easily attached employing only downhand welding. In actual construction the top ribs would be fireproofed by the floor slab. At the University shop no difficulties were encountered in installing these ribs. A small crescent recess was provided in the ribs to clear the flange welds. Reinforcing ribs in place are shown in the photograph in Fig. 6. Adding these ribs gives a two-thirds increase in the flange areas, and reduces the stresses in flange welds by 40 percent. In this manner the critical section moves from the flange welds to the tips of the ribs.

The last group of specimens studied in this investigation was introduced into the test program to emphasize to designers that there are other alternatives for developing the shear and moment capacity of connections. In this paper only two of these connections are highlighted: Specimens 9 and 11. These moment connections are intended for use in beam-to-column flange joints. These connections are of an all welded type and, with some modifications, are similar to those employed in Japan. Specimen 9 represents a beam with thick flanges, whereas flanges for Specimen 11 are thin. These specimens are intended to demonstrate the improvements in the connection capacity that can be achieved when no copes are necessary for the back-up bars.

EXPERIMENTAL RESULTS

A summary of experimental results for the 18 tests is given in accompanying Table 2. Hysteretic diagrams for the test specimens are illustrated as needed and were generated by applying increasing cyclic displacements as shown in Fig. 7. For purposes of discussion of the test results, specimens will be divided into four groups. In the first two, specimens with small ratios of plastic flange moduli to plastic beam moduli, Z_1/Z, will be considered. In the first group only specimens using W18×35 beams and with $Z_1/Z = 0.66$ are included; the second group consists of W21×44 beams with $Z_1/Z = 0.62$. Three specimens with beam-to-column web connections form the third group. Two specimens having

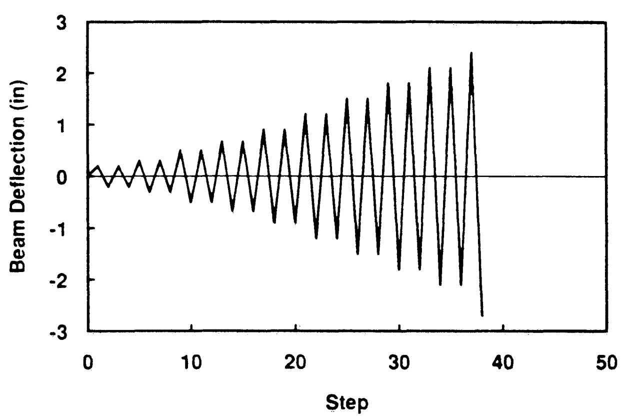

Fig. 7. Typical load-displacement sequence

direct all-welded beam-to-column flange connections are in the fourth group.

W18×35 Beam Moment Connections to Column Flanges

Specimens 3, 17, and 13, using W18×35 beams with $Z_1/Z = 0.66$, comprise the first group. The hysteretic loops for these specimens are shown in Fig. 8. In order to have these results comparable to other tests, the abscissae are plotted in percent of beam rotation determined by dividing the end deflection by the cantilever length L_c. In all cases the effect of elastic column rotation of the joint is excluded. As a guide to the lowest acceptable level of performance, the hysteretic loops are bracketed by vertical lines at ± 2 percent.

The early failure of Specimen 3 on a downward stroke can be noted from the top diagram. On manually reversing the applied load and closing the fracture, excellent ductility in the opposite direction was observed. This disparity in ductilities in the two directions illustrates the possibility of erratic behavior. The hysteretic diagram for similar Specimen 17 shows better behavior. The improved performance of this connection can be attributed to the use of tension control web bolts. The ends of these particular bolts twist off on reaching the specified bolt tension. Similar improvement in the cyclic capacities of the moment connections was attained by applying fillet welds between the shear tabs and the beam webs. These welds were designed to develop 20 percent of the plastic web capacity, per the new SEAOC requirements.[2]

The advantages in using either the tension controlled bolts or supplementary web welds can also be noted from Table 2 giving a Summary of Experimental Results. From this table it can be noted that the behavior of Specimen 3 was poor. In this specimen the flange weld fracture was abrupt. The maximum *total* beam rotation was 1.61 percent (0.0161 rad), and the maximum *plastic* rotation, θ_p, was about one percent. Even the beam plastic rotation, θ_p*, from the extreme zero intercept of the hysteretic loop on the left with the x-axis was only about 1.5 percent. This kind of beam plastic rotation capacity is inadequate for general applications for severe seismic service unless some inelastic rotation can also be depended upon in the panel zone of a column.[11]

The hysteretic loops shown in Fig. 8 for Specimens 13 and 17 are better than those for Specimen 3. The improvement in Specimen 13 can be attributed to the use of tension control bolts, whereas in Specimen 17, to the supplementary web welding. Moreover, the copes for back-up bars were ground smooth.

W21×44 Beam Moment Connections to Column Flanges

Specimens 5, 18, and 14, using W21×44 beams, form the second group. The hysteretic loops for these specimens are shown in Fig. 9. Virtually the same remarks as above are

Table 2.
Summary of Experimental Results

Specimen [1]	L_c [2] (in.)	M_p^* [3] (kip-in.)	P_p^* [4] = [3]/[2] (kip)	P_{36} [5] (kip)	P_{max} [6] (kip)	P_{max}/P_{36} [7] = [6]/[5]	P_{max}/P_p^* [8] = [6]/[4]	θ_{max} [9] (%)	θ_p [10] (%)	θ_p^* [11] (%)
1	61.3	3280	53.5	40.2	67	1.67	1.25	5.84	5.24	8.66
2	65.8	3280	49.8	37.4	61	1.63	1.22	2.53	1.95	3.29
3	64.8	3540	54.6	32.0	61	1.91	1.12	1.61	0.94	1.53
4	65.8	3807	57.9	37.4	62	1.66	1.07	1.65	0.92	1.71
5	62.9	4905	77.9	46.7	81	1.73	1.04	1.31	0.68	1.39
6	61.6	4905	79.7	47.7	79	1.66	1.01	1.33	0.73	1.41
7	59.8	5031	84.1	49.1	90	1.83	1.07	2.11	1.46	2.42
8	58.8	5014	85.3	50.0	95	1.90	1.11	2.43	1.75 (4.82)	3.39 (7.60)
9	62.9	3487	55.4	45.1	75	1.66	1.35	5.03	4.49	6.93
10	56.6	3782	66.8	39.8	80	2.01	1.20	3.33	2.73	4.76
11	63.2	4388	69.0	46.5	72	1.55	1.04	1.77	1.26 (3.21)	1.95 (5.65)
12	61.9	4388	70.9	46.5	77	1.66	1.09	2.01	1.51	2.65
13	63.2	3683	58.3	32.8	69	2.10	1.18	2.02	1.27	2.12
14	63.2	4298	68.0	46.5	76	1.63	1.12	2.80	2.39	4.16
15	63.2	3683	58.3	32.8	33	1.01	0.57	0.43	0.04	0.04
16	63.2	4298	68.0	46.5	57	1.23	0.84	0.83	0.39	0.78
17	63.2	3291	52.1	32.8	54	1.65	1.04	2.12	1.48	2.70
18	63.2	4006	63.4	46.5	70	1.51	1.10	1.81	1.36	2.63

L_c = Distance from applied load to face of column or edge of stiffener
M_p^* = Plastic moment capacity based on tensile coupon strength
P_{36} = Nominal cantilever load at yield stress of 36 ksi
P_{max} = Maximum attained load during test
θ_{max} = Maximum beam rotation before failure occurs or moment capacity M_p^* is exhausted
θ_p = Maximum beam plastic rotation before failure occurs or moment capacity M_p^* is exhausted
θ_p^* = Beam plastic rotation measured from zero intercept to the same point as defined in θ_p
Values of θ_p and θ_p^* in parentheses give rotations at M_p where M_p = plastic moment at 36ksi

applicable to these specimens. Again erratic behavior was observed for the first specimen. The performance of the connections was significantly improved either by using tension control web bolts or 20 percent supplementary welding as required in Ref. 2. In Specimen 18 the copes were ground smooth. These results are quantified in Table 2. It is to be noted that the rib welds for Specimen 7 were inadequate.

Beam Moment Connections to Column Webs

Specimens 1, 2, and 8 were selected for illustrating cyclic behavior of beam moment connections to column webs. The hysteretic loops for these specimens are shown in Fig. 10. Ribs were used to reinforce the beam-to-column connections for Specimens 1 and 8. The cyclic behavior of these connections was excellent. With regard to these tests

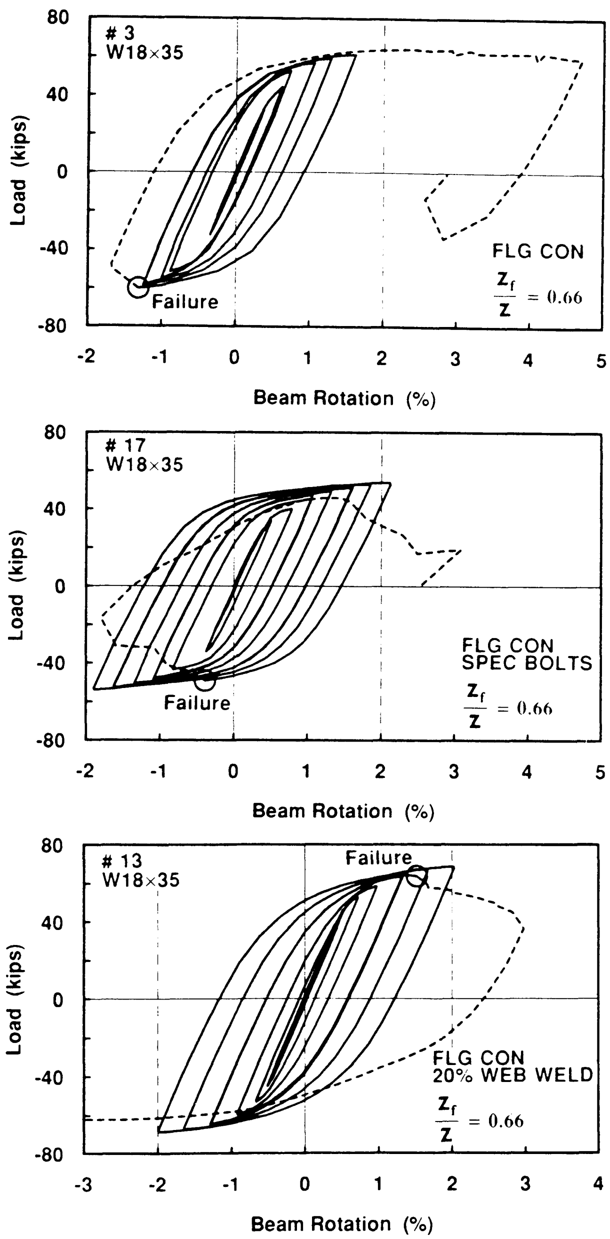

Fig. 8. Hysteretic loops for W18×35 beams connected to column flanges

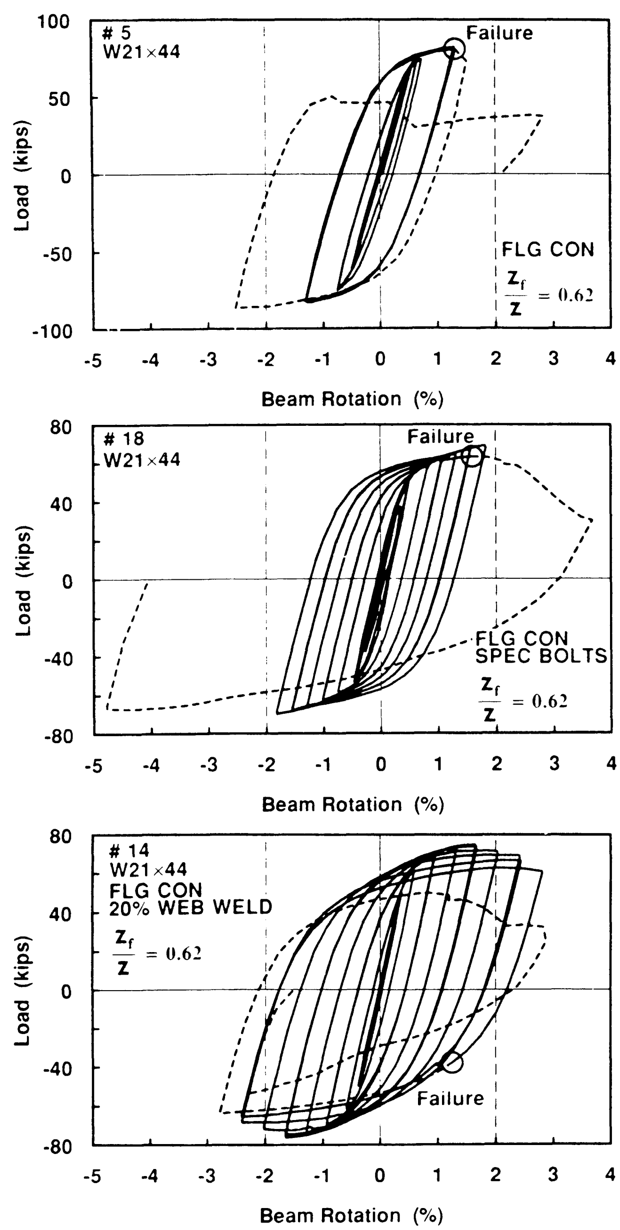

Fig. 9. Hysteretic loops for W21×44 beams connected to column flanges

it is useful to note the following:

The test on Specimen 1 with rib reinforcement was discontinued after reaching a very large ductility. Specimen 8, also with rib reinforcement, exhibited excellent ductile behavior and no weld fractures occurred. However, since the flange-thickness ratio was relatively large, at big cantilever displacements, significant buckling of flanges developed.

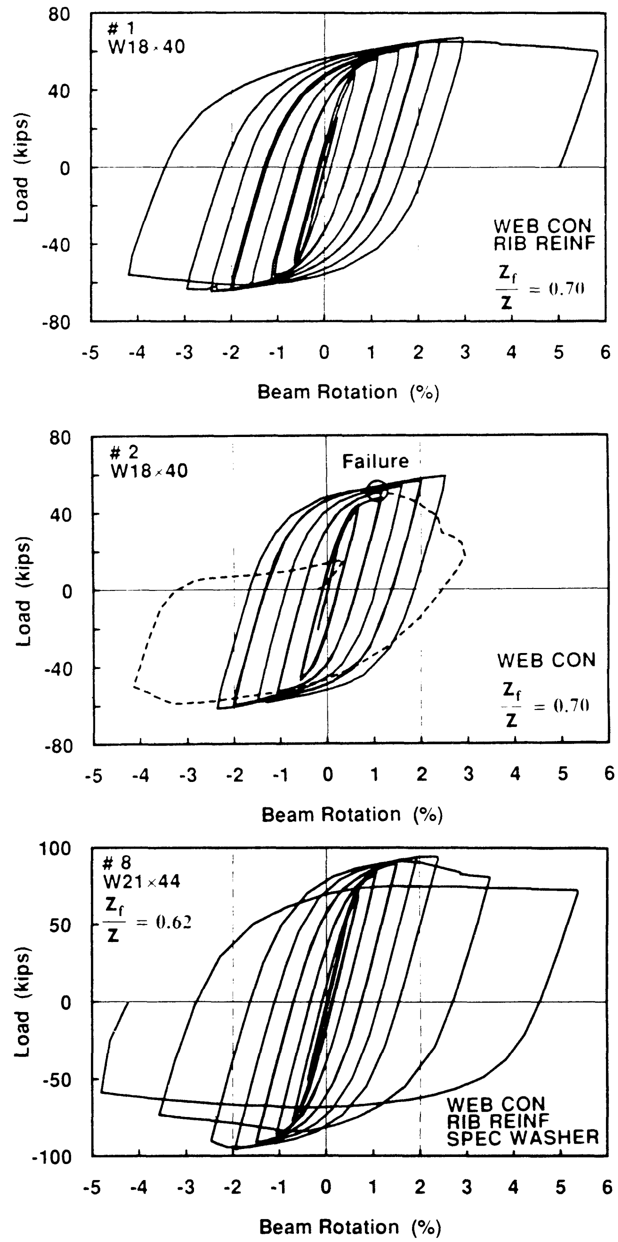

Fig. 10. Hysteretic loops for selected beam specimens connected to column webs

Nevertheless a substantial moment continued to be carried by the beam. In order to record this observation in Table 2, the beam plastic rotation limits at *nominal* plastic moments are given in parentheses in columns 10 and 11. For Specimen 2, although the beam plastic rotation angles, θ_p and θ_p*, were also large, an abrupt weld fracture was the cause for terminating the test.

All Welded Beam Moment Connections to Column Flanges

Hysteretic loops for Specimens 9 and 11, shown in Fig. 11, are the last group considered in this paper. As noted earlier both of these connections were fabricated by welding the beams directly to the column flanges. Continuous fillet welds were used on both sides of the beam cross section for Specimen 11. For Specimen 9 partial penetration welds from the outside and fillet welds from the inside were used to attach the flanges. Fillet welds were applied on both sides of the beam web. No copes in the webs were used in

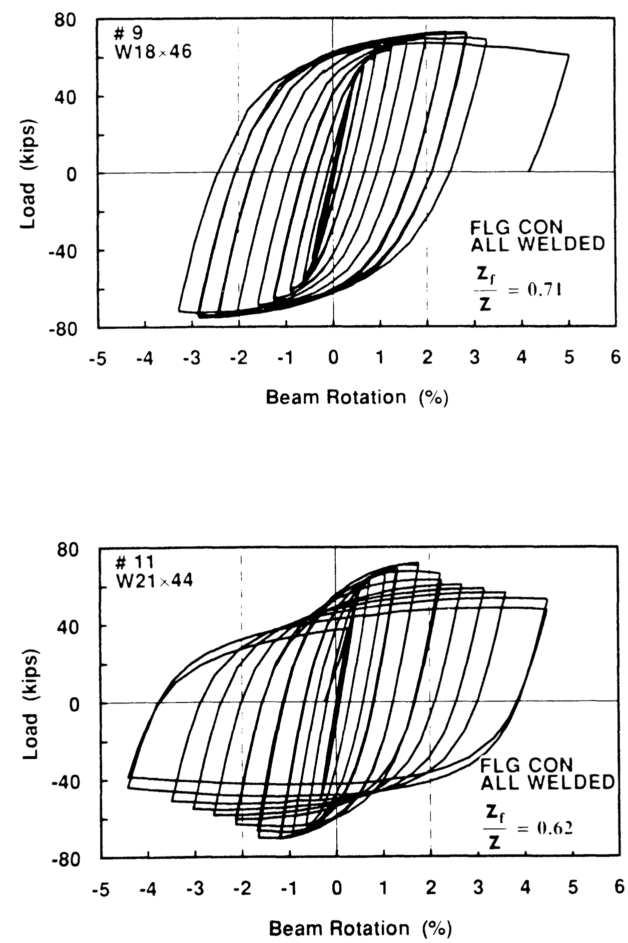

Fig. 11. Hysteretic loops for two different beams welded directly to column flanges

this assembly of members. Neither were any stress relief holes introduced at the junctures of the beam flanges with the webs.

The behavior of these specimens was intended to illustrate the improvement in joint ductility obtained when no holes for back-up bars are provided. An examination of the hysteretic loops in Fig. 11 shows that the ductility of these connections is excellent. A slight decay in capacity occurred in Specimen 11 due to flange buckling of the type already described in connection with Specimen 8. As can be noted from Table 2, the maximum beam rotation angles were very large. No weld failure occurred in either one of these two specimens. The same excellent cyclic behavior was observed for Specimens 10 and 12 where a similar welding scheme was used in attaching beams to thick end-plates.

A few remarks on Specimens 15 and 16 are now in order. These were made up by a vendor generally using SMAW rather than flux-cored arc welding. In conformity with the AWS Specifications,[12] the joint was not preheated during fabrication. The appearance of the welds was good, but they were not inspected ultrasonically. The tests showed that the ductility of these connections was poor as recorded in Table 2.

In this series of experiments only flange welds in Specimens 3 through 6 were inspected ultrasonically and were found to be satisfactory.

COMPARISONS WITH EARLIER TESTS

It is instructive to relate the obtained results to some available test data. Perhaps the most suitable data for this purpose were first published in Ref. 13 and are now available in suitably reduced form in Ref. 14. These are reproduced here in Table 3. Column 9 of this table can be directly compared with Column 11 in Table 2, bearing in mind only that the beam plastic rotations in one table are given in radians, and in the other in percent.

On scanning the above two columns in these tables, the beam plastic rotations for Specimens 1, 2, 7, 8, 9, 10, 11, 12, 14, 17, and 18 are comparable with the earlier work, and that for Specimen 13 is marginal. On the other hand, the rotations for Specimens 3, 4, 5, and 6, and certainly for the poorly prepared Specimens 15 and 16, are unacceptable. In situations *where it can be shown* that the column panel zones can effectively participate with the beams in dissipating energy, some of the restrictions noted above may be rescinded. Experiments with that type of joint are described in Ref. 11.

CONCLUSIONS

Steel seismic beam-to-column moment connections must attain a reliable required strength *and* ductility. Beam plastic rotation in the conclusions stated below will be taken as a measure of ductility. Weld fractures at connections are particularly dangerous. This series of tests provides additional information on several kinds of connections. Inasmuch as complete load reversals were applied in all cases, judgment is required in the interpretation of results. The following observations may be helpful to the reader in evaluating the reported results on this limited series of tests, bearing in mind that the number of parameters is very large.

1. All 18 specimens attained their *strength* at a maximum nominal 36 ksi yield stress (Table 2, col. 7). Sixteen of the specimens also exceeded their yield strengths based on flange coupon tests, however the two poorly prepared Specimens 15 and 16 did not attain such strength (Table 2, col. 8).
2. *Ductility* in beam-to-column flange moment connections in Specimens 7, 8, 9, 10, 11, 12, 14, 17, and 18 was comparable with the earlier tests[13,14] considered satisfactory (Table 2, cols. 10 and 11). Specimen 13 may perhaps be classified with this group, although its perfor-

Table 3. Summary of Joint Behavior[14]

Specimen No. (1)	Connection Type (2)	P_{36} [kips] (3)	P_p [kips] (4)	P_p [ksi] (5)	P_u/P_p (6)	P_{36}/V_u [%] (7)	$P_u/\bar{V}_u$ [%] (8)	θ_p^* [rads] (9)	Bolt Slip At 36 ksi (in.) (10)
	W18×50 Series								
1	5⅞ in. bolts	38.6	54.8	71.2	1.30	30	42	0.033	0.0028
2	All welded			75.4	1.38		45	0.059+	No slip
3	5¾ in. bolts			74.9	1.37		45	0.042	0.0031
4	4¾ in. bolts			74.6	1.36		45	0.034	0.0024
	W24×76 Series								
5	7⅞ in. bolts	76.4	87.2	126.3	1.45	37	58	0.053+	0.0172
6	7½ in. bolts			112.9	1.29		52	0.024	0.0212 (estim.)
7	All welded			138.0	1.58		63	0.056+	No slip

P_{36} = Calculated Cantilever Tip Load at 36 ksi Max. Beam Stress
P_p = Calculated Cantilever Tip Load for Max. Beam Moment = M_p
P_u = Av. Measured Ultimate Tip Load (Corrected for 7% XY error)
V_u = 0.55 $F_y dt_w$ with F_y = 36 ksi
$\bar{V}_u$ = 0.55 $F_y dt_w$ with F_y from Web Coupons (47 ksi and 36 ksi)
θ_p^* = Inelastic (Plastic) Beam Rotation at P_u

mance was very marginal. In some of these specimens the requisite ductility was achieved using either rib reinforcements at the connections, tension control web bolts, or 20 percent supplementary web welds.

3. Variability in ductility of moment connections was observed in many instances. This can and should be reduced by the following means:

   (a) Careful inspection of the connections. This includes welds as well as bolts.

   (b) Copes in beam webs for back-up bars as well as gouges caused by cutting should be as smooth as possible.

   (c) From these limited tests the use of either tension control for web bolts or supplementary web welds appears to enhance the performance of bolted web connections for beams with Z_f/Z of 0.70 or less. Bolt slippage reported earlier in Ref. 13 appears to be one of the main causes for scatter in connection capacity. This indicates the need for strict compliance with the procedures for tightening bolts in slip critical connections as specified by AISC and the Research Council on Structural Connections.

4. *If* it can be demonstrated on the basis of *inelastic* analysis that the column panel zone in a beam-to-column flange moment connection can effectively participate in providing ductility, the requirements for beam connection ductility can be reduced.[11]

5. Ductility in beam-to-column web moment connections was found to be satisfactory. For Specimens 1 and 8 this was achieved with the aid of ribs across the flange welds. Specimen 2 was fabricated with exceptional care using the SMAW process, and cannot necessarily be duplicated in the field.

6. Specimens 9 and 11, with fully welded beam-to-column flanges having no copes in the beam webs, provided exceptionally high ductilities.

7. In many instances fracture of beam specimens initiated near web copes or at tack welds for back-up bars. It is important to have the copes ground smooth and to tack weld back-up bars on the inside of weld preparation.

8. From the limited number of tests and under laboratory conditions no essential difference in performance was noted between the two types of tension control devices used for web bolts.

9. The ductility requirements discussed in this paper pertain to applications in moment resisting frames and not for eccentrically braced frame links where the requirements may be more severe.

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