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Cyclic tests on large scale steel moment connections

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ABSTRACT: Cyclic loading tests were conducted on eight large scale welded flange-bolted web type steel moment connections. The objective was to investigate the influence of various web connection details on the overall connection performance. The eight specimens showed highly variable performance, and developed plastic rotations that were judged as poor to marginal for severe seismic applications. Variability in the performance of the beam flange groove welds dominated the response of the specimens. This paper summarizes the test program and compares results to those of previous U.S. test programs. Concerns are expressed about the reliability of this connection detail for seismic applications.

1 INTRODUCTION

The beam-to-column connection is a critical element of a steel moment resisting frame. The most widely used moment connection detail in current U.S. practice for seismic-resistant framing is the welded flange-bolted web detail. The beam flanges are welded to the column using complete penetration single bevel groove welds, with the welds typically made in the field using the self-shielded flux core arc welding (FCAW) process. The beam web is field bolted to a single plate shear tab which is shop welded to the column.

Recent tests by Tsai and Popov (1988) have indicated that this detail may not provide satisfactory performance when used for beams in which the web accounts for a substantial portion of the beam's flexural strength. This observation has been attributed to the inability of the bolted web connection to transfer bending moment, resulting in excessive demands on the beam flange connections. As a result of this concern, recent U.S. seismic code provisions (Recommended lateral force requirements 1988, Seismic provisions 1990, Uniform building code 1988) require that in addition to web bolts, supplemental welds be provided between the beam web and the shear tab for beam sections with $Z_f/Z \leq 0.70$. Z is the beam's plastic section modulus, and Z_f is the plastic section modulus of the beam flanges only. The supplemental web welds must be designed to develop at least 20 percent of the beam web's flexural strength. These provisions recognize that the web connection must be capable of carrying not only shear force, but also a portion of the beam's bending moment. These new provisions are based on rather rather limited data, and according to the Recommended lateral force requirements (1988), were chosen "until additional data are available."

An experimental investigation was undertaken to collect additional data on the effect of the Z_f/Z ratio and the web connection detail. The specific objective of the research sponsor was to determine if a relaxation of the supplemental web welding requirements could be justified based on additional experimental data. This paper summarizes results of this experimental program.

2 TEST PROGRAM

Tests were conducted on cantilever type test specimens in which a beam was connected to the flange of 3.65 m long pin ended column. Cyclic loads were applied to the beam at a distance of 2.44 m from the column face. A total of eight specimens with different beam sections and web connection details were used, as summarized in Table 1. All beams were of A36 steel (specified F_y = 250 MPa). W12x136 sections of A572 Gr. 50 steel (specified F_y = 345 MPa) were used for the columns of all specimens. This section was chosen to provide a very strong panel zone, so that inelastic deformation of the specimens occurred primarily as flexural yielding of the beam rather than shear yielding of the column panel zone.

Connection details for two of the test specimens are illustrated in Figures 1 and 2. Similar details were used for all specimens. Complete penetration single bevel groove welds were used to connect the beam flanges to the column flange in all specimens. This weld was detailed with a 10 mm root opening and a 30 degree bevel. Backup strips were used and remained in place

Table 1. Test Specimens.

Spec.	Beam	Z_f/Z	Web Connection
1	W24x55	0.61	6 - A325 bolts
2	W24x55	0.61	6 - A490 bolts
3	W24x55	0.61	6 - A325 bolts + web welds
4	W18x60	0.75	4 - A325 bolts
5	W18x60	0.75	4 - A325 bolts
6	W21x57	0.67	5 - A325 bolts
7	W21x57	0.67	5 - A325 bolts + web welds
8	W21x57	0.67	all welded

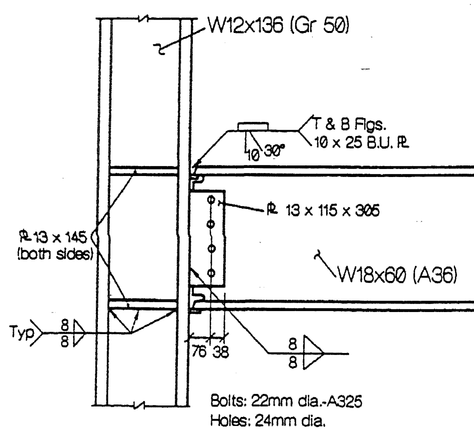


Figure 1. Connection details for Specimen 4.

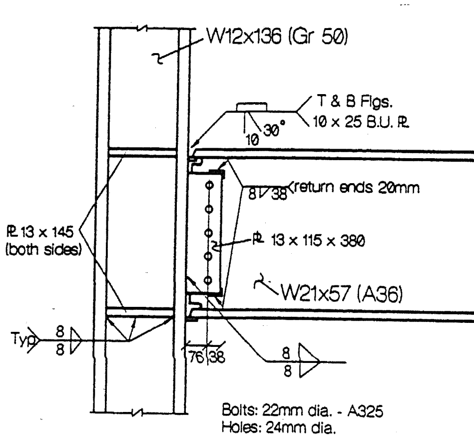


Figure 2. Connection details for Specimen 7.

after the welds were completed. Welding was by self-shielded FCAW process, using an E70T-7, 2.4 mm diameter electrode (specified F_u = 485 MPa).

The web connection details (Table 1) included all bolted connections, bolted connections with supplemental web welds, and an all welded web connection. All bolts were 22 mm diameter, grade A325 (specified F_u = 830 MPa) or grade A490 (specified F_u = 1035 MPa). The A325 bolts were tensioned to 182 kN and the A490 bolts to 227 kN, using a carefully calibrated torque control method. For each of the beam sections used in the test program, three 22 mm diameter A325 bolts would have been sufficient to satisfy connection shear strength requirements of the Recommended lateral force requirements (1988). It was judged, however, that a larger number of bolts would be used for these beam sections in current California practice. The number of bolts was therefore increased as indicated in Table 1 to more closely reflect current detailing practice.

For the specimens with supplemental web welds (Nos. 3 and 7), these welds were sized to provide a nominal strength of 20% of M_{pweb}. M_{pweb} was computed as 250 MPa$*(d-2t_f)^2*t_w/4$, where d, t_f and t_w are the overall beam depth, flange thickness and web thickness. Weld strength was computed using 1.7 times the usual allowable stress, per the Recommended lateral force requirements (1988). The supplemental web welds were placed at the top and bottom edges of the shear tab (Figure 2) to provide the maximum flexural web connection capacity with the smallest possible welds.

For Specimen 8, the beam web was directly welded to the column flange using a single bevel complete penetration groove weld, with three A325 bolts provided to simulate erection bolts. All-welded connections such as Specimen 8 have shown excellent performance in past test programs (Popov and Stephen 1972, Tsai and Popov 1988).

All specimens were provided with 13 mm thick continuity plates, as shown in Figures 1 and 2. According to the Recommended lateral force requirements (1988), continuity plates were not required for any of the test specimens. These plates were provided, however, to eliminate the possible influence of local column flange bending as an additional variable affecting test results. No doubler plates were required for any of the test specimens, and none were provided.

The connection design and detailing for Specimens 3, 4, 5, 7, and 8 met or exceeded all current U.S. seismic code requirements. Specimens 1, 2, and 6 intentionally violated code requirements for supplemental web welds, but satisfied or exceeded all other requirements.

The fabrication sequence was identical for all specimens. First, the continuity plates and shear tab were welded to the column. Bolts were then installed and fully tensioned. The top beam flange was then welded, followed by the bottom flange. After the beam flange welds cooled, the supplemental web welds were made on Specimens 3 and 7, and the complete web weld was made on Specimen 8. All groove welds were made with the specimens in an upright position to simulate field welding positions. The fabrication and welding sequence were chosen to simulate typical U.S. field construction practice.

All specimens were constructed by a commercial structural steel fabricator. Welders were certified per the Structural welding code (1988). Inspection was provided by a commercial welding inspection firm and included ultrasonic testing of the column flanges for laminations prior to fabrication, a pre-weld fitup inspection, and ultrasonic testing of all complete penetration groove welds. The individual conducting the inspections had all appropriate certifications from the American Welding Society and the American Society for Nondestructive Testing. The ultrasonic inspections revealed a defect in a groove weld of Specimen 3, which was then repaired and successfully retested ultrasonically.

3 TEST RESULTS

Each specimen was subject to cyclically increasing displacements at the tip of the cantilever. When specimens entered the inelastic range, yielding was typically concentrated in the beam flanges. Specimens with bolted webs only (Nos. 1, 2, 4, 5, and 6) showed virtually no yielding in the beam web, reflecting the lack of flexural participation of the web connection. Specimens with web welding (Nos. 3, 7 and 8) typically showed significant beam web yielding. Some specimens also showed minor yielding in the column panel zone.

Cyclic loading was continued for each specimen until connection failure. Failure for all specimens occurred as fracture at a beam flange groove weld. For Specimens 1, 2, and 4, failure occurred unexpectedly early in the loading program. For each of these three specimens, the beam had barely entered the inelastic range when a sudden failure occurred at the bottom flange weld. Complete fracture occurred nearly instantaneously at the interface of the weld and the column flange. For each of these specimens, after failure at the bottom flange occurred, the load was reversed for one final monotonic half-cycle of loading, which was continued until failure of the top flange occurred. The top flange failures occurred by a gradual tearing of the beam flange well outside of the groove weld. The specimens all developed substantial ductility during this final monotonic half-cycle of loading. In fact, it was not possible to fail the top flange of Specimen 4 before running out of stroke on the hydraulic loading ram.

The hysteretic response of Specimen 4 is illustrated in Figure 3. Results are plotted as moment versus plastic rotation. Moment was computed as the load multiplied by the distance to the face of the column. Rotation was computed as the tip displacement divided by the distance to the face of the column. Plastic rotation was obtained by subtracting out the measured elastic portion of the rotation. This method of computing plastic rotation is consistent with previous experimental investigations, and therefore provides a meaningful basis for comparison.

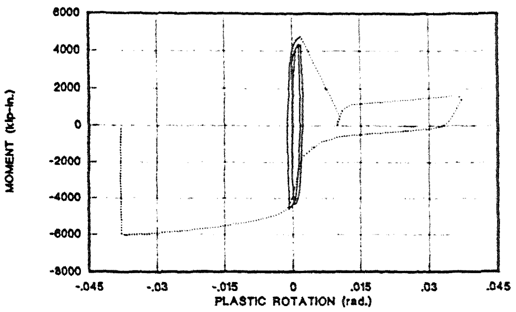


Figure 3. Hysteretic response of Specimen 4 (1 kip-in = .113 kN-m).

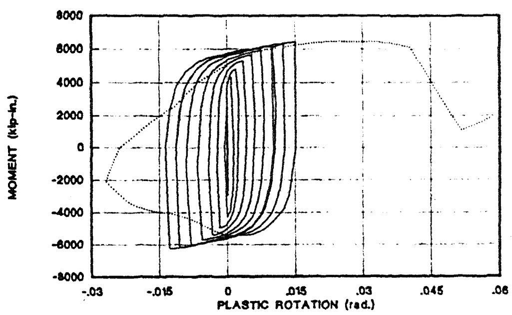


Figure 4. Hysteretic response of Specimen 7 (1 kip-in = .113 kN-m).

The dotted lines in Figure 3 represent the response during the final monotonic half-cycle of loading, after failure at the bottom flange connection had occurred.

The remaining specimens (Nos. 3 and 5-8) showed somewhat better performance. These specimens all failed by the development of a fracture at or near the interface of the groove weld and the beam flange. These fractures generally initiated at the edge of the beam flange, and gradually spread across the width of the flange as the loading progressed. In many cases, the fracture occurred at the interface of the weld and the beam flange, running along the bevel in the flange.

Figure 4 displays the hysteretic response of Specimen 7. Failure of this specimen occurred at the top flange weld in the manner described above. The dotted lines represent the specimen's response during the final monotonic half-cycle of loading, after failure of the top flange connection had occurred. This final monotonic loading was continued until failure also occurred at the bottom flange connection.

Of the eight specimens tested in this program, Specimen 7 provided the best performance in terms of plastic rotation, whereas Specimen 4 (Figure 3) provided the worst performance.

Table 2. Summary of plastic rotations and failure modes.

Spec.	θ_p (rad)	Failure Mechanism
1	0.004	sudden fracture at weld-column interface at bottom flange
2	0.003	sudden fracture at weld-column interface at bottom flange
3	0.009	gradual fracture through bottom beam flange, outside of weld
4	0.002	sudden fracture at weld-column interface at bottom flange
5	0.013	gradual fracture at weld-beam interface at bottom flange
6	0.013	gradual fracture at weld-beam interface at bottom flange
7	0.015	gradual fracture at weld-beam interface at top flange
8	0.012	gradual fracture at weld-beam interface at bottom flange

Table 3. Maximum moments.

Spec.	M_{max}	M_p	M_p^*	M_{max}/M_p	M_{max}/M_p^*
1	612	545	694	1.12	0.88
2	610	545	694	1.12	0.88
3	722	545	694	1.33	1.04
4	540	500	627	1.08	0.86
5	673	500	627	1.34	1.07
6	664	524	594	1.27	1.12
7	716	524	594	1.37	1.21
8	737	524	594	1.40	1.24

Notes: moments in kN-m
M_{max} = maximum measured moment in beam at column face, prior to connection failure
M_p = beam plastic moment based on nominal specified dimensions and yield stress
M_p^* = beam plastic moment based on measured dimensions and yield stress

Table 2 summarizes the plastic rotation, θ_p, and failure mechanism for each test specimen. The tabulated value of θ_p is the maximum plastic rotation attained before failure, as measured from the beam's original undeformed position. Most specimens were loaded until fracture occurred at both the top and bottom flange connections. However, the point at which the first flange connection fractured is taken as the actual "failure" for each specimen, since beyond this point, the ability to resist cyclic loading is exhausted.

The maximum moment, M_{max}, developed by the beam at the column face prior to connection failure is listed in Table 3. For comparison, the nominal specified plastic moment, M_p, and the estimated actual plastic moment, M_p^*, are also listed. In the final monotonic cycle after failure, some specimens developed bending moment substantially higher than the value of M_{max} listed in Table 3.

4 DISCUSSION OF TEST RESULTS

4.1 *Weld performance*

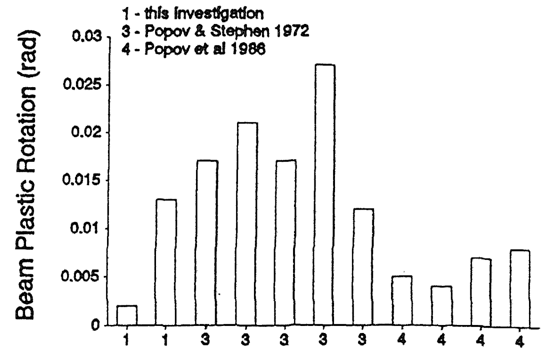
As described above, the failures of Specimens 1, 2, and 4 occurred unexpectedly early in the cyclic loading program. These three specimens failed in a nearly identical manner: a sudden fracture at the interface of the column and the groove weld at the beam's bottom flange. Examination of the failure surfaces revealed a small region of incomplete fusion near the lower central portion of each of the welds, suggesting poor welding as the cause of the failures. Further investigations and consultation with welding specialists supported this observation. A different welder was used for Specimens 5 to 8, and similar premature failures were not observed.

As described earlier, failures in Specimens 5 to 8 were generally observed at the weld-beam interface. These failures occurred after the beams had achieved substantially higher moments and plastic rotations as compared to Specimens 1, 2, and 4. Clearly, the welds were of higher quality in these later specimens. Nonetheless, subsequent studies of these failures indicated that these welds likely did not develop the full tensile strength of the beam flanges. Thus, the groove welds for Specimens 5 to 8 also do not appear to have performed in a completely satisfactory manner. There were no obvious defects in these welds, and possible causes of their somewhat unsatisfactory performance is still under investigation.

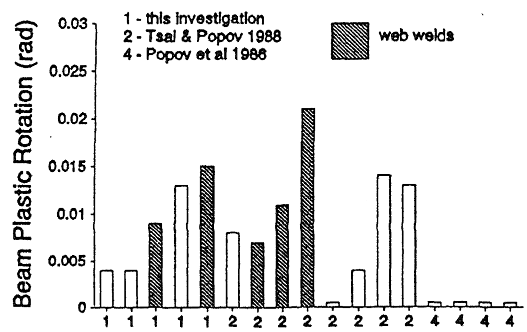
It is important to recall that all test specimens in this program were constructed by a commercial structural steel fabricator. All welders were certified, and all groove welds were ultrasonically tested by an independent welding inspection firm. Nonetheless, a number of welds were found to be inadequate when the connections were tested to destruction. This suggests the possibility that current industry practices in the U.S. for welding and quality control may not be adequate for this moment connection detail when subject to large cyclic deformations.

4.2 *Strength and plastic rotation capacity*

Based on the data in Table 3, all specimens devel-



(a) Specimens with $Z_f/Z > 0.70$.



(b) Specimens with $Z_f/Z \leq 0.70$.

Figure 5. Comparison of beam plastic rotations with past tests.

oped their nominal specified plastic moments. Further, all specimens, except Nos. 1, 2, and 4, developed their estimated actual plastic moments. The detrimental effect of the flange weld failures on the strength of Specimens 1, 2, and 4 is clear from Table 3.

A few comparisons between the performance of specimens with the same Z_f/Z ratios can also be made. The most interesting trend in behavior can be observed by comparing Specimens 6 to 8. These specimens were provided with increasing degrees of flexural capacity in their web connections. The data in Table 3 show increasing strengths progressing from Specimen 6 to 8. This suggests that when reasonably good quality flange welds are provided, the web connection details may have a significant effect on overall connection strength.

The primary criteria for judging the performance of the test specimens was their plastic rotations. A value of ±0.015 radian was chosen as a reasonable estimate of beam plastic rotation demand in steel moment resisting frames subject to severe earthquakes. This is based on a review of recent inelastic dynamic frame analyses and appears to be consistent with criteria used in past test programs (Popov et al 1986, Tsai and Popov 1988). At joints in which the column panel zone can effectively take part in developing inelastic deformations, the plastic rotation demand on the beam can be relaxed (Tsai and Popov 1988).

Based on the criterion of ±0.015 radian of beam plastic rotation at the connection measured from the original undeformed position, the performance of the eight specimens was unsatisfactory overall. Based on the data in Table 2, the performance of Specimens 1, 2, and 4 was particularly poor. The other specimens performed significantly better. However, Specimen 7 was the only one to achieve ±0.015 radian of plastic rotation. All others were judged as marginally acceptable, with plastic rotations varying from ±0.009 radian to ±0.013 radian.

Although the test results showed some effect of web connection details on strength, there is no clear evidence from the test data that the Z_f/Z ratio or the web connection details had a significant influence on ductility. Specimens 5 and 6, for example, both had bolted webs, but had different Z_f/Z ratios (0.75 for Specimen 5 and 0.67 for Specimen 6). Yet, the plastic rotation was nearly identical for these two specimens. Compare also Specimens 6, 7, and 8. These specimens had the same Z_f/Z ratio, but had increasing degrees of web participation. Specimen 7 with the supplemental web welds did develop greater plastic rotation than Specimen 6, which had only a bolted web. However, Specimen 8, with a fully welded web, developed less plastic rotation than either Specimens 6 or 7.

Overall, variability in the performance of the beam flange welds appears to have had a much greater influence on plastic rotation capacity than Z_f/Z ratio or web connection detail. This variability is clearly evident when comparing Specimens 4 and 5. These specimens had nominally identical beam sections and connection details. Yet, Specimen 5 achieved approximately 6 times greater plastic rotation.

5 COMPARISON WITH EARLIER TESTS

Results of this investigation were compared with results of tests conducted by Popov and Stephen (1972), Popov et al (1986), and Tsai and Popov (1988). Although this is not an exhaustive database, it is believed to represent a significant portion of cyclic tests performed on large scale specimens in the U.S. Figure 5 shows beam plastic rotations achieved by specimens in these test programs, divided according to Z_f/Z ratio. Only beam to column flange connections with bolted webs, with or without supplemental web welds, are included. Specimens with supplemental web welds are highlighted. The beam plastic rotations as shown in this figure were calculated as half of the maximum plastic rotations measured from the total width of the hysteretic loops, as a basis for comparing results from many different tests. The tests by Popov et al (1986) included significant panel zone deformations. However, only the plastic rotation attributable to the beam is included in the bar charts in Figure 5.

Several observations can be made from the data in Figure 5:

1. The results of the current test program, in terms of the magnitude and variability of plastic rotations, are similar to previous test programs.
2. There is a large variability in performance of test specimens in both the current test program as well as in previous test programs.
3. A significant number of specimens have not achieved plastic rotations of .015 radian. A rather large number have not even achieved .005 radian.
4. Specimens with $Z_f/Z > 0.70$ have shown better performance, on average, than those with $Z_f/Z \leq 0.70$. Both groups of specimens, however, show variable performance.
5. Specimens with supplemental web welds (all have been on beams with $Z_f/Z \leq 0.70$) have performed somewhat better on average than their counterparts with no web welding. There is again, however, considerable variability.

6 CONCLUSIONS

The results of this test program revealed no clear influence of the Z_f/Z ratio or web connection detail on the performance of welded flange-bolted web moment connections. Rather, variability in the performance of the beam flange groove welds dominated the response of the specimens. One of the objectives of this investigation was to determine if current U.S. seismic code requirements for supplemental web welds could be relaxed. Although the influence of web welds was masked by variability in the flange welds, specimens with the supplemental web welds performed somewhat better than their counterparts without web welds. Consequently, no relaxation in current code requirements is recommended.

An important observation from this and previous test programs is that the welded flange-bolted web connection detail has shown highly variable performance. While some specimens have performed satisfactorily, a significant number have demonstrated poor or marginal performance in cyclic test programs. Some of this variability can be attributed to the influence of Z_f/Z ratio and web connection details. However, a great deal of the variability also appears to be related to the performance of the beam flange groove welds.

The final recommendation of this investigation calls for a thorough review of current U.S. industry practice for seismic steel moment connections. The results of this and previous tests programs leads to questions on the reliability of the current welded flange-bolted web detail for severe seismic applications. A careful review of design and detailing practices, as well as welding and quality control issues is needed.

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REFERENCES

Popov, E.P., Amin, N.R., Louie, J., and Stephen, R.M. 1986. Cyclic behavior of large beam-column assemblies. *Engineering Journal.* First Quarter. American Institute of Steel Construction Inc.

Popov, E.P. and Stephen, R.M. 1972. Cyclic loading of full-sized steel connections. Bulletin No. 21. American Iron and Steel Institute.

Recommended lateral force requirements and tentative commentary. 1988. Seismology Committee, Structural Engineers Association of California.

Seismic provisions for structural steel buildings, load and resistance factor design. 1990. American Institute of Steel Construction, Inc. Chicago, Illinois.

Structural welding code - steel, AWS D1.1-88. 1988. American Welding Society. Miami, Florida.

Tsai, K.C. and Popov, E.P. 1988. Steel beam-column joints in seismic moment resisting frames. 1988. *Report No. UCB-EERC 88/19.* Earthquake Engineering Research Center. University of California at Berkeley. Berkeley, California.

Uniform building code. 1988. International Conference of Building Officials, Whittier, California.