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Cyclic Performance of Deep Column Moment Frames with Weak Panel Zones

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ABSTRACT

Deep columns are frequently used in steel special moment resisting frames in the United States in order to control drift and reduce construction costs. This paper presents an experimental study on welded unreinforced flange-welded web (WUF-W) steel moment connections to a deep wide-flange column. Three large-scale interior moment connection specimens with a full range of panel zone strength levels were subjected to slowly applied cyclic loads up to failure. The objective was to investigate the effect of panel zone strength on the seismic performance of WUF-W moment connections with deep column sections. All specimens satisfied the qualifying drift angle criteria for the seismic connection required in the current AISC Seismic Provisions, at least 0.04 radian story drift angle prior to failure without significant strength degradation. This paper summarizes the experimental program and key test results.

1. INTRODUCTION

Steel moment resisting frames (MRFs) are used for seismic resistant building construction in high seismic regions. A key design issue of steel MRFs is the balance of yielding between the beams and column panel zones to achieve high levels of ductility under strong earthquake motions.

One of the earliest studies on panel zone behavior was conducted by Krawinkler (Krawinkler et al. 1971; Krawinkler 1978). The study showed that panel zone shear yielding results in highly ductile behavior, with stable and repetitive hysteresis loops under cyclic loading. El-Tawil et al. (1999) indicated that excessive shear deformation of panel zone can increase the potential for brittle and/or ductile fracture despite its contribution to the connection ductility. Tests by Jones et al. (2002) showed that the specimens with weak panel zones achieved excellent performance, developing large story drifts without strength degradation.

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Experimental studies on WUF-W moment connections were conducted by Ricles at al. (2002) and Lee et al. (2005a, 2005b). The majority of column sections used in above two studies were shallow wide-flange column sections (W14 sections).

Deep columns are frequently used in steel special moment resisting frames in the United States in order to control drift and reduce construction costs. An experimental study was conducted to collect additional data on the effect of panel zone strength on the seismic performance of WUF-W moment connections with deep column sections. This paper summarizes the experimental program and key test results

2. TEST SETUP AND SPECIMENS

Tests were conducted on three large-scale interior moment connection specimens. Fig. 1 shows the test setup. Cyclic loads were slowly applied at the top of the column and the lateral supports were provided at the connection area and beam ends as shown. To simulate points of inflection under lateral load, the ends of the members were pin connected. The bottom of the column was connected to a high capacity clevis and the top of the column was rigidly attached to the loading crosshead, which can be controlled to simulate a pin. Each beam end was supported by two actuators that are intended to serve as a roller support. That is, the actuators were controlled to keep the beam ends at a constant elevation, and allow free horizontal translation and rotation of the beam ends.

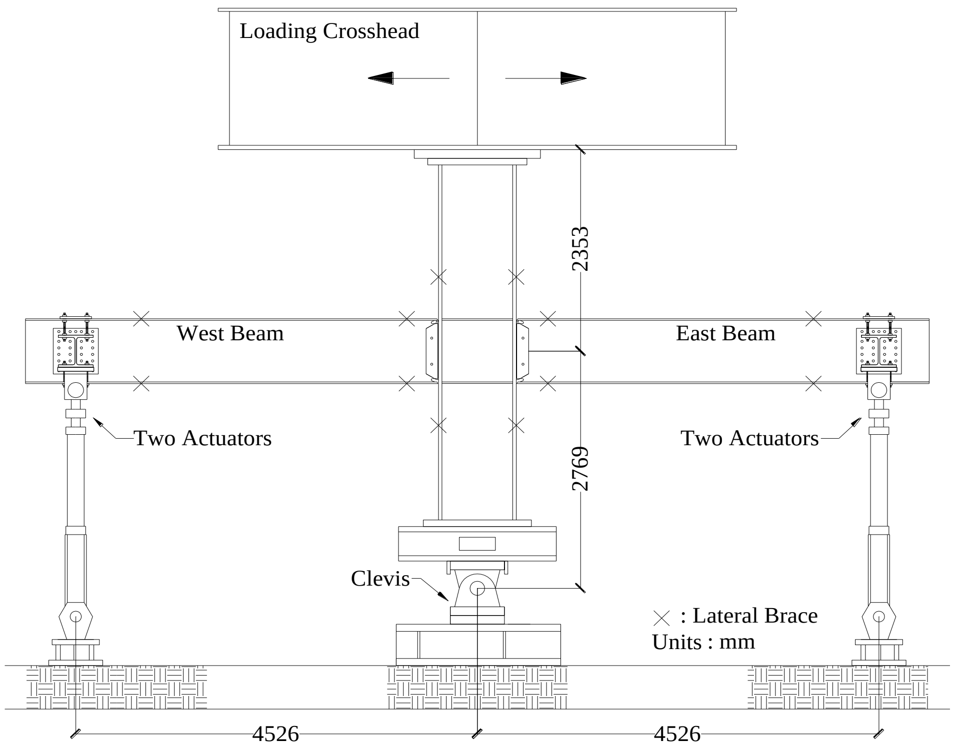

Fig. 1 Test setup

Table 1. Test specimens

Specimen	Beam	Column	Doubler Plate Thickness (mm)	Panel Zone Strength	Panel Zone Requirement of the AISC Seismic Provision
UT01	W30x108	W33x263	0	Weak	Violated
UT02	W30x108	W33x263	13	Balanced	Satisfied
UT03	W30x108	W33x263	2 at 13	Strong	Satisfied

Table 1 lists the key features of the three specimens. The specimens consisted of W30x108 beams and W33x263 column, each of A992 steel. Note that the beams and column for the Specimen UT03 were taken from different heats of steel as compared to those of the other two specimens. The material properties from tension coupon testing are not available at the time of writing. Specimens were designed with three different levels of panel zone strength. Specimen UT01 with weak panel zone was intended to promote severe inelastic deformation of the panel zone. Specimen UT03 with strong panel zone was designed so that plastic hinges would occur in the beams. Specimen UT02 with balanced panel zone was intended to share yielding between the beams and the panel zone.

Connection details for the Specimen UT02 are shown in Fig. 2. The details were identical for all three test specimens, with exception of the doubler plate thickness, as summarized in Table 1. Complete-joint-penetration (CJP) groove welds of beam flanges to column were made using the self-shielded flux core arc welding (FCAW-S) process with a 2.4 mm diameter E70T-6 (Lincoln NR-305) electrode. The backing bar was removed at the beam bottom flange groove welds and the root of CJP groove weld was reinforced with fillet welds. The backing bar was left in place at the beam top flange groove welds and reinforcing fillet welds were provided between the backing bar and the column flange. The weld tabs were removed and ground smooth. A 1.8 mm diameter E71T-8 (Lincoln NR-232) electrode was used for the CJP groove welds of beam webs. Supplemental fillet welds were provided between the shear tab and the beam web. The weld access holes were fabricated based on the research of Ricles et al. (2002). All specimens were provided with 19 mm thick continuity plates.

3. TEST RESULTS

The test specimens were subjected to cyclic loads by applying increasing levels of the total story drift angle. The pre-determined loading sequence, listed in Table 2, followed the loading protocol specified in the 2010 AISC Seismic Provisions (AISC 2010). The cyclic loading was increased until severe failure was observed at each specimen.

3.1 Overall specimen performance

The column tip load versus total story drift angle for all specimens is shown in Fig. 3. A summary of test results is listed in Table 3, including: maximum total story drift angle;

Table 2. Loading sequence

Total Story Drift Angle (rad)	Number of Loading Cycles
0.00375	6
0.005	6
0.0075	6
0.01	4
0.015	2
0.02	2
0.03	2
0.04	2
0.05	2
0.06	2
0.07	2
0.08	2
Continue loading at increments of 0.01 radian, with two cycles of loading at each increment	

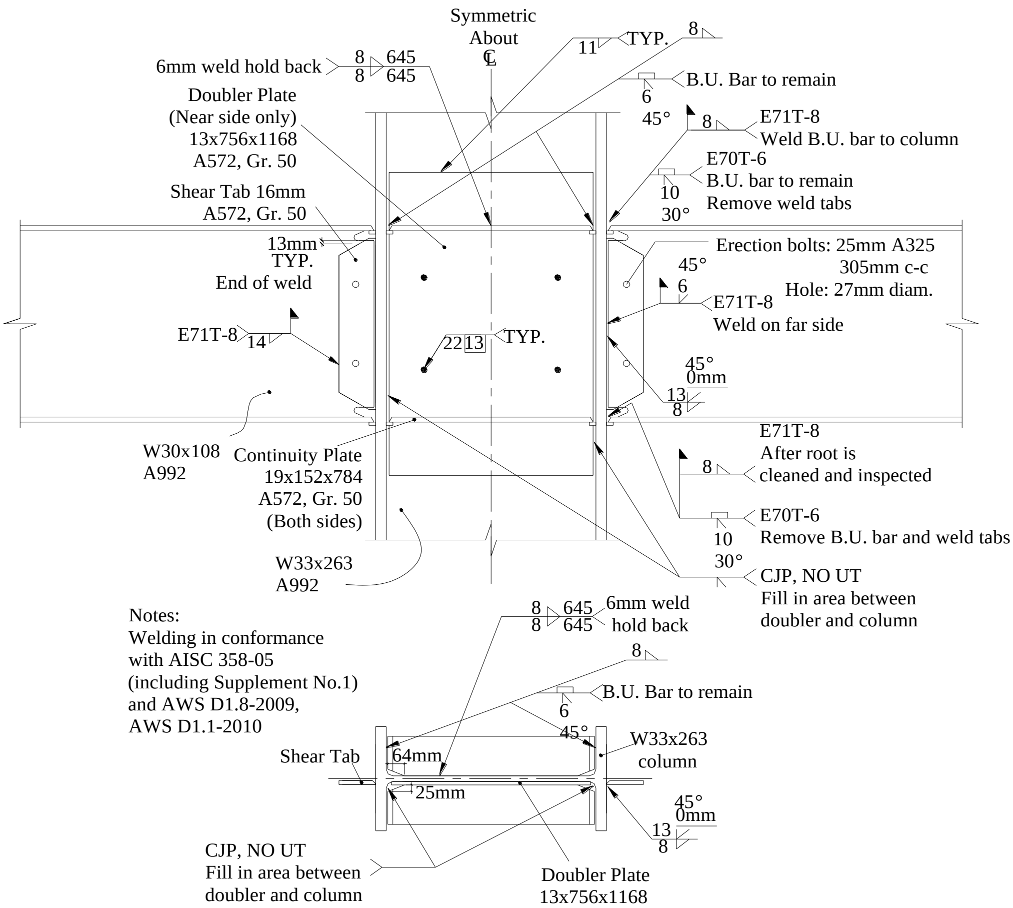

Fig. 2 Connection details for Specimen UT02

maximum total plastic rotation; maximum plastic rotation of beam, panel zone and column; and ratio of maximum panel zone shear force V_{max} to the design shear strength $\emptyset_v V_n$. The design shear strength of the panel zone was computed in accordance with the 2010 AISC Seismic Provisions (AISC 2010). The maximum total story drift angle is from at least one complete loading cycle prior to fracture or strength degradation below 80% of the nominal flexural strength of the specimen. All specimens satisfied the qualifying drift angle of 0.04 radians. Fig. 4 shows the connection region of each specimen at the end of test.

Table 3. Summary of test results

Specimen	UT01	UT02	UT03
Maximum total story drift angle (rad.)	0.04	0.05	0.04
Maximum total plastic rotation (rad.)	0.027	0.041	0.03
Maximum beam plastic rotation (rad.)	0.004	0.04	0.029
Maximum panel zone plastic rotation (rad.)	0.019	0.001	0.001
Maximum column plastic rotation (rad.)	0.005	0.002	0.002
$V_{max}/\emptyset_v V_n$	1.21	0.83	0.62

Note: Maximum total story drift angle for at least one complete loading cycle prior to fracture or significant strength degradation - less than 80% of the nominal flexural strength

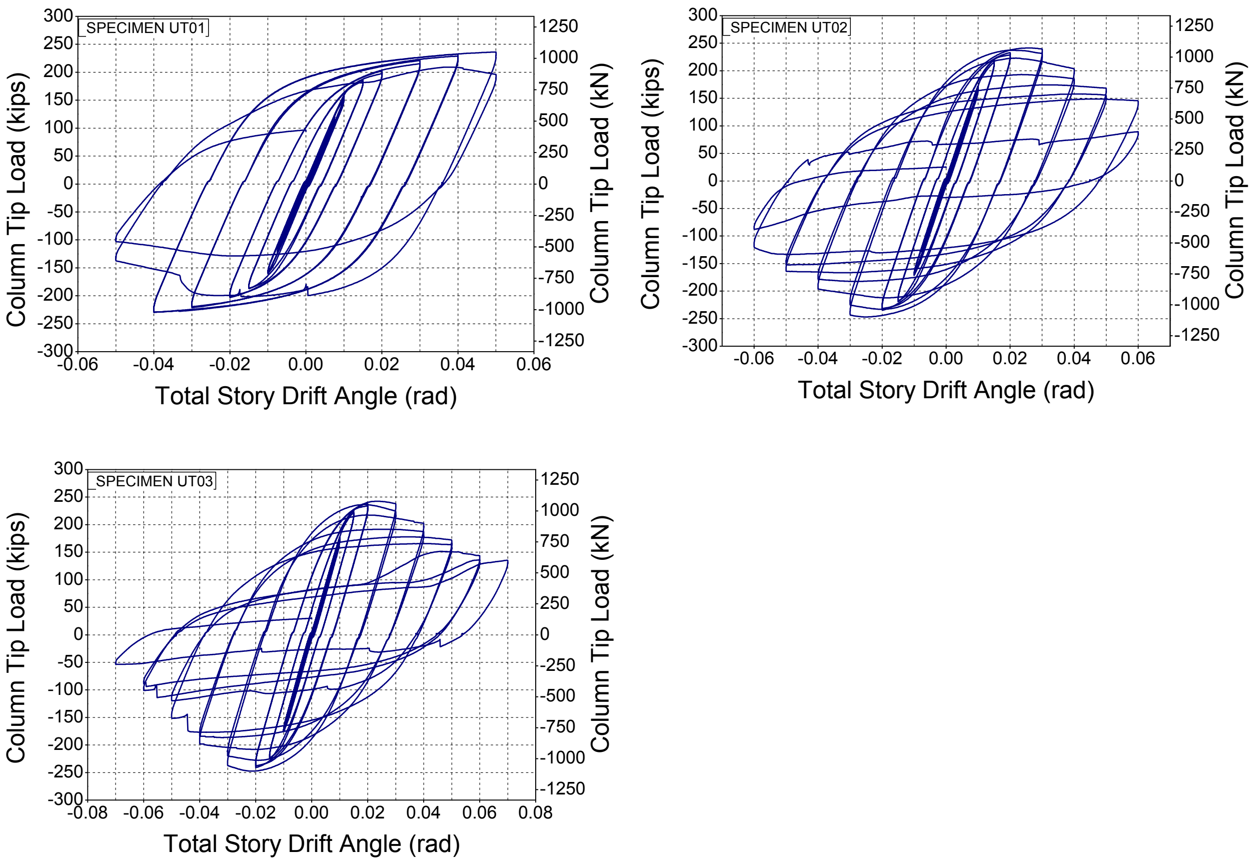

Fig. 3 Column tip load versus total story drift angle for all specimens

(a) Specimen UT01

(b) Specimen UT02

(c) Specimen UT03

Fig. 4 Photos of specimens after testing

Specimen UT01 showed stable hysteretic response through completion of the 4% story drift cycles (see Fig.3). The column panel zone began to yield during the 1% story drift cycles. At about the 4% story drift, minor local buckling occurred in the flanges of both beams and small cracks were observed in the top and bottom edges of the shear tab. A fracture in the east beam top flange occurred during the first cycle of 5% story drift. The fracture appeared to initiate in the center portion of the flange near the interface between the groove weld and the beam flange. After the east beam top flange fractured, significant local buckling occurred in the flanges and web of the west beam [see Fig. 4 (a)]. The specimen developed 0.027 rad. of maximum total plastic rotation, with 70% of panel zone contribution.

Initial minor yielding of the Specimen UT02 occurred in the top and bottom flanges of both beams during the 0.375% story drift cycles. Yielding in both beams was greater in subsequent drift levels. During the 2% drift cycles, the column panel zone slightly yielded and minor local buckling occurred in the beam flanges. During the 4% story drift cycles, more severe local buckling was observed in the flanges and web of both beams. During the first cycle of 6% story drift, cracks were observed in the bottom flange-web fillet area of both beams. These cracks occurred at the location of severe flange and web local buckling, approximately one-half the beam depth from the column face. The cracks caused ductile tearing of the bottom flange of the west beam [see Fig. 4 (b)]. Ductile tearing in the east beam bottom flange also occurred in subsequent cycle. The specimen developed 0.041 rad. of maximum total plastic rotation, with 98% of beam contribution.

In Specimen UT03, no yielding occurred on the doubler plates in the column panel zone. During the 3% drift cycles, flange and web local buckling occurred in both the beams. The local buckling became more severe in subsequent drift levels. During the first cycle of 5% story drift, the top flange of the east beam fractured. The fracture appeared to initiate at the tip of the top flange near the interface between the groove weld and the beam flange. Shortly after the first cycle of 5% drift, additional cracks were observed in three different locations. The locations were the upper weld access hole region in the east beam, the interface of the fillet weld and the east beam bottom flange on the bottom side of the flange, and the interface between the groove weld and the west beam bottom flange. During the second cycle of 5% story drift, the top flange of the east beam was completely separated from the column face and the crack near the weld access hole extended into the east beam web. The bottom flange of the west beam was torn during the first cycle of 7% story drift. The cracks initiated in the bottom flange-web fillet area at the location of severe flange and web local buckling, and then propagated into the beam bottom flange [see Fig. 4 (c)]. The specimen developed 0.03 rad. of maximum total plastic rotation, with 97% of beam contribution.

3.2 Energy dissipation

Fig. 5 shows the contribution of the beam, panel zone, and column to the accumulated energy dissipation at different story drift angle. Table 4 also lists the total dissipated energy and the contribution of the components.

The total energy dissipation in Specimen UT01 is as large as that in Specimen UT03. As intended in design, the majority of the energy dissipation in Specimen UT01 and UT03 occurred in the panel zone and the beams, respectively.

Specimen UT02 had the largest amount of energy dissipation. This can be attributed to the fact that the specimen sustained more loading cycles (i.e., 5% story drift cycles) prior to failure, compared to the other specimens. Specimen UT02, designed to have a balanced panel zone strength, exhibited strong panel zone behavior. Note that 91% of the total dissipated energy was occurred in the beams (Table 4).

Table 4. Energy dissipation of test specimens

Specimen	Total Energy Dissipation (kJ)	Energy Dissipation by Beams (kJ)	Energy Dissipation by Panel Zone (kJ)	Energy Dissipation by Column (kJ)
UT01	1814	211 (12%)	1254 (69%)	349 (19%)
UT02	2809	2553 (91%)	106 (4%)	150 (5%)
UT03	1865	1705 (91%)	52 (3%)	107 (6%)

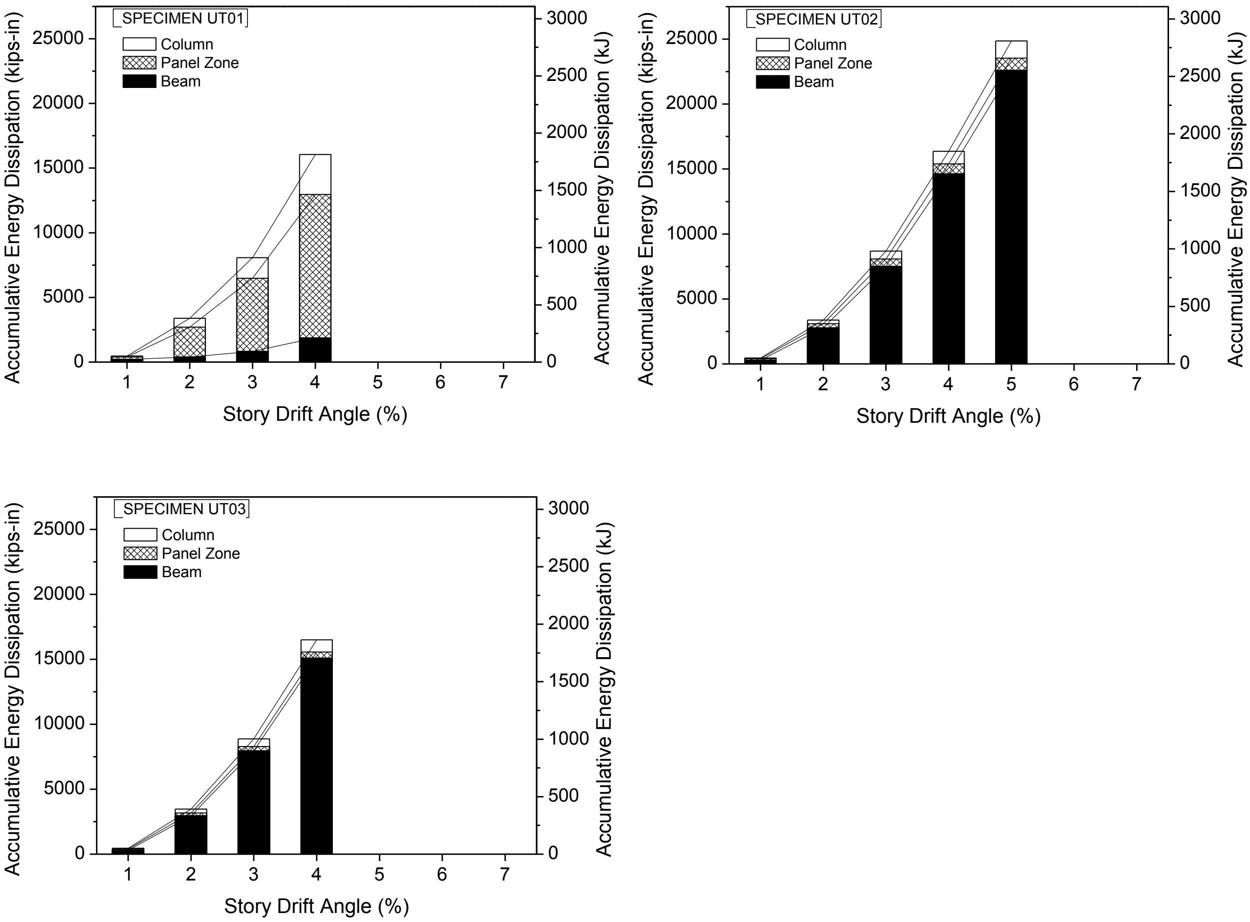

Fig. 5 Energy dissipation of test specimens

4. CONCLUSIONS

Based on the experimental results, the following conclusions can be made regarding the WUF-W moment connections to a deep column:

1. All specimens achieved 0.04 radians story drift qualification criteria for SMF connections.
2. Specimen UT01 with weak panel zone exhibited less beam buckling and therefore less strength degradation than Specimen UT03 with strong panel zone.
3. The panel zone shear strength indicated in the AISC seismic provisions (AISC 2010) is somewhat underestimated. The specimen designed with balanced panel zone strength (Specimen UT02) exhibited strong panel zone behavior.

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