ECONOMICAL WELDING DETAILS AND DESIGN FOR CONTINUITY AND DOUBLER PLATES IN STEEL SPECIAL MOMENT FRAMES

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Abstract

Steel Special Moment Frames (SMF) are regularly used in seismic force-resisting systems when more architectural freedom over braced frames is desired. These systems have excellent ductility employing the highest ASCE response modification factor with no limits on building height. Current AISC Seismic Provisions require the use of continuity plates when a proportion between the beam flange breadth and column flange thickness is not realized. Additionally, it is required that the weldments between the continuity plates and column flange be a complete-joint-penetration (CJP) groove weld. The required thickness of continuity plates is specified as either 75% or 50% of the adjacent beam flange thickness for interior and exterior connections, respectively. Full-scale testing of 10 moment frames was performed to investigate the design of these continuity plates and their weldments. Six of these frames were exterior connections utilizing the prequalified Reduced Beam Section (RBS) connection, while the remaining four were interior connections utilizing the prequalified Welded Unreinforced Flange–-Welded Web (WUF-W) connection. While violating the current continuity plate requirements, all 10 connections surpassed the 0.04 rad story drift requirement of SMF according to the prequalification criteria of AISC Seismic Provisions. The testing has demonstrated that (1) the AISC proportion criteria requiring the use of a continuity plates based on the ratio of the beam flange breadth and the column flange thickness may be relaxed, (2) the weldments connecting the continuity plate-to-column flange may be made with fillet welds sized to develop the tensile strength of the continuity plates, (3) the continuity plate thickness may be determined based on a plastic interaction equation, and (4) that hot-dip galvanization of the members in a moment frame prior to welding does not impair the performance. In addition to the economizing of the continuity plate welds, it was found that sizing the doubler plate weldments for the shear developed according to the relative doubler plate stiffness was adequate to fasten the doubler plate. This may result in significant savings for lightly loaded doubler plates, which currently require welds to develop the shear strength of the doubler plate. Additionally, a new width-to-thickness limit is proposed to prevent instability of the continuity plate.

Keywords: Moment Frame; Moment Connection; Continuity Plate; Fillet Weld; Doubler Plate.

1. Introduction

Steel moment frames are a common Seismic Force-Resisting System (SFRS) due to the architectural freedom they offer. They permit open bays and eliminate the need for braced frames or shear walls. These systems develop plastic hinging through the plastification of the beams and the base of the first-story column. These SFRS have excellent levels of ductility which allow designers significant reductions of the required elastic seismic design forces. However, after the 1994 Northridge Earthquake, significant damage to steel moment frames was observed at drift levels far below their assumed capacity.

The observed damage instigated a significant research effort, which made substantial changes to the detailing of steel moment frames. The research found that a combination of low fracture toughness weld metals, a lack of control of base metal properties, and connection geometries susceptible to high localized strain conditions were the leading cause of the fractures. After these findings were acknowledged, strict control of the use of steel moment frames was imposed through AISC 341, Seismic Provisions for Structural Steel Buildings [1], AISC 358, Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications [2] and AWS D1.8, Structural Welding Code-Seismic Supplement [3]. These controls involve mandatory use of notch-tough weld electrodes for welds designated as Demand Critical (DC), modified access hole geometries, and weld root treatments to minimize sharp discontinuities. However, the most important provision requires that Special Moment Frames (SMF) and Intermediate Moment Frames (IMF) match the dimensions and detailing of previously experimentally qualified connections with limited extrapolation.

The Prequalified Connections document summarizes the geometry limitations and detailing requirements of prequalified connections since connection testing would be prohibitively expensive to perform on a project-by-project basis. A number of these connections are proprietary, wherein the intellectual property is licensed during the design phase. Two popular non-proprietary connections are the Reduced Beam Section (RBS) and the Welded Unreinforced Flange with Welded Web (WUF-W). Some of the prescriptive detailing requirements used in these connections, enacted after the Northridge Earthquake, are recognized to be potentially conservative—specifically, the welding requirements of continuity plates and doubler plates for SMF and IMF. These plates are installed between the column flanges to stiffen the connection and ensure the desired inelastic behavior of the frame. The stiffening elements accomplish this by preventing excessive out-of-plane column flange deformation which would otherwise lead to premature failure of the connection, and by reinforcing the high shear panel zone such that plastic hinging occurs in the beam.

2. Current Design Provisions and Background Research

The Seismic Provisions have two requirements dictating when a continuity plate shall be used in a connection. This first requirement pertains to when the available strength of the column as computed for the Web Local Yielding (WLY) [see Eq. (1)] or the Flange Local Bending (FLB) [see Eq. (2)] limit states of Section J10 of the Specification for Structural Steel Buildings (AISC 360) [4] are insufficient to resist the beam flange force. The second requirement is when the column flange thickness is less than the beam flange breadth divided by 6 [see Eq. (3)].

$$R_n = (5k + t_{bf})F_y t_{cw} \tag{1}$$

$$R_n = 6.25 t_{cf}^{\ 2} F_y \tag{2}$$

$$t_{cf} \geq b_{bf} / 6 \tag{3}$$

where k, t_{bf}, t_{cw}, t_{cf}, and b_{bf} are geometric properties of the beam and column shapes used to form the connection, and F_y is the yield strength of the material. When either of these requirements dictates the use of a continuity plate, the plate thickness shall be 50% of the adjacent beam flange thickness for exterior (one-sided) connections or 75% of the thicker adjacent beam flange for interior (two-sided) connections. Another limit state, Web Local Crippling, is found to rarely govern the design of continuity plates in rolled shapes.

The current requirement for the weld between the continuity plate and the column flange requires the weld to be a CJP groove weld; the use of a CJP weld rather than a fillet weld has significant economic implications. These welds require additional fabrication to bevel the edge of the plates and install a backing bar, additional weld volume, and more stringent inspection requirements. This inspection requirement for CJP welds significantly increases the cost of fabricating the continuity plates. Considering these factors, the mandatory use of CJP welds imposes an increase in cost so significant that many designers prefer to increase the size of the column to mitigate the need for additional stiffening elements.

Adequately designing the fillet welds for continuity plates would require the reconciliation of the flow of forces through the joints. A CJP weld does not possess this requirement as the weld develops the strength of adjacent plates—implying that failure of the plate would occur before the weld. The use of fillet welded continuity plates has been explored by other researchers [5, 6, 7]. More recently, a proposed methodology using the elastic flexibility of the continuity plates was proposed [8] and validated [9]. In this elastic approach, the continuity plate-to-column flange weld is sized based on the estimated proportion of strain hardened beam flange force that flows into each continuity plate. The research presented herein aims to extend beyond an elastic design philosophy for the continuity plates by using a plastic distribution of forces.

The column proportion limit of Eq. (3) originates from a study on WUF-W connections shortly after the Northridge Earthquake [10]. The original derivation of the equation was derived using a low-cycle fatigue analysis and solving for the requisite column flange stiffness to preclude ductile fracture of the beam flange-to-column flange weld. The subsequent adoption of this expression resulted in a significant simplification of the expression to eventually arrive at Eq. (3). This equation is applied without prejudice to all SMF connections, including RBS which typically have much lower flange forces than a WUF-W connection. Fig. 1 shows that this limit may be triggered for several cases where a strength limit state does not govern. The band of connections shown in pink indicate where Eq. 3 requires a continuity plate while green connections do not require continuity plates. Connections shown with a white color are not permitted per AISC 341 due to violation of the Strong Column Weak Beam requirement. Several specimens of this research program investigate if this requirement is warranted for RBS connections.

Intimately linked to the continuity plate is the doubler plate. When present, this plate acts to double up the web to resist the high shear forces that develop within the panel zone of the moment connection. The high shear force is a result of the concentrated flange forces which resolve the beam moment as a force-couple. These flange forces flow through the column flanges into the continuity plates (if present) before ultimately loading the panel zone in shear. According to the Seismic Provisions, vertical weldments of the doubler plates to the column flanges are required to develop the shear strength of the plate—irrespective of the demand that may exist for the plate. Part of the explanation for this level of conservatism is observed high levels of stress in finite element models of the doubler plate attributed to a complex flow of forces and strain hardening of the doubler plate [11]. Some research has explored minimizing doubler plate reinforcement to encourage hysteretic energy dissipation through panel zone yielding which can lower the overall expense by minimizing the required doubler plate thickness [12]. However, the research herein aims to economize the welding of doubler plates by validating a new design approach, which determines the doubler plate shear flow based on the relative panel zone plate proportions.

3. Specimen Design Principles and Test Matrix

The design of the continuity plates is based on Section J10 of AISC 360 [4] which assumes the force entering the plate, P_{cp}, is equal to the concentrated force less the governing column limit state:

$$P_{cp} = \frac{1}{2}\left[P_f - \min\left(WLY, FLB\right)\right] \quad (4)$$

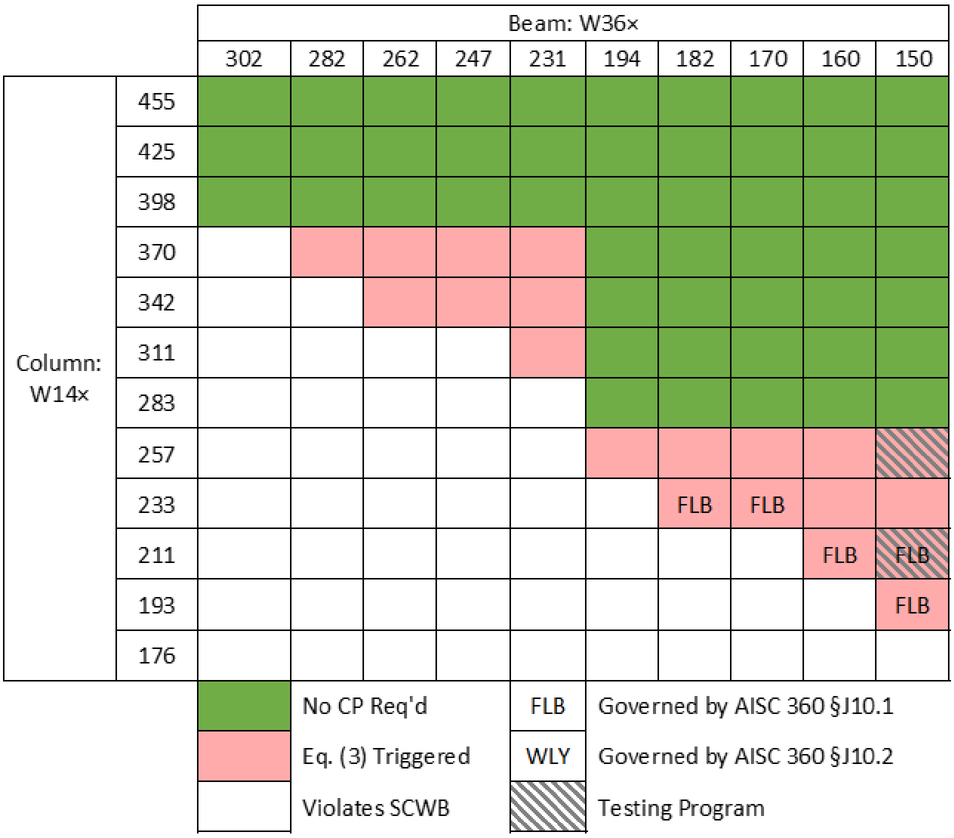


Fig. 1 – One-Sided RBS Continuity Plate Limit State Matrix

where P_f is the hardened flange force as per AISC 358. To complete the design of the continuity plate, a shear force, V_{cp}, based on the equilibrium of the continuity plate is established (see Fig. 2). The axial component of the continuity plate is assumed to be centered on the net continuity plate edge, b_n. Finally, the continuity plate is designed based on a plastic interaction equation [13]:

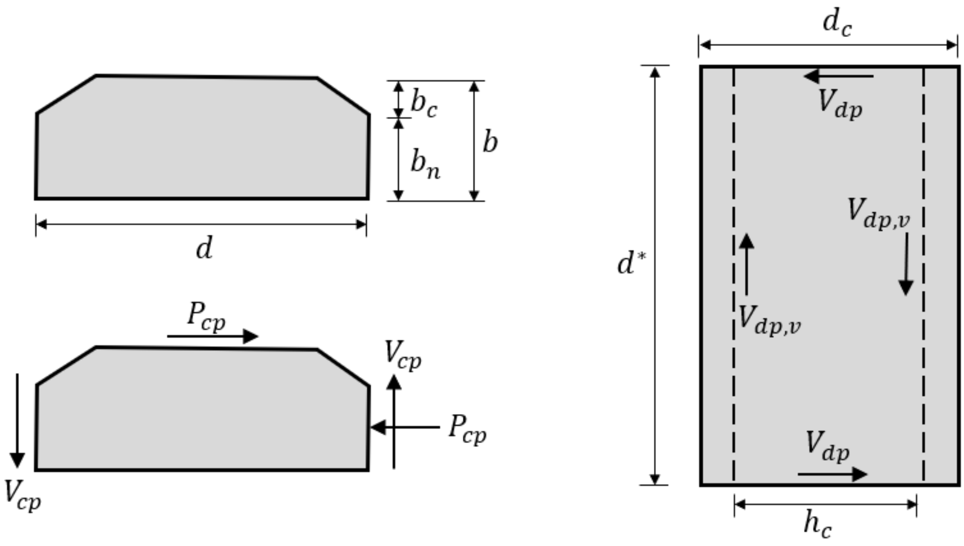
$$\left(\frac{P_{cp}}{P_c}\right)^2 + \left(\frac{V_{cp}}{V_c}\right)^2 \leq 1.0 \tag{5}$$

where P_c and V_c are the axial and shear strengths of the continuity plate. The continuity plate-to-column fillet welds are designed such that the weld size, w, for each weld of a fillet weld pair is equal to 3/4 the thickness of the continuity plate. This is similar to the ubiquitous ‘5/8’ rule commonly used to develop the yielding limit state of a plate, but has been increased such that the fillet welds are capacity protected for the force that develops after yielding of the continuity plate.

Although the current requirements for continuity plates include a minimum thickness, AISC 341 imposes no formal limits on the width-to-thickness ratio of a continuity plate. Finite element analysis has confirmed that the appropriate width-to-thickness ratio of the stiffener is:

$$\frac{b}{t} = 0.56\sqrt{\frac{E}{F_y}} \tag{6}$$

When continuity plates were used, all except Specimen C5 met the above criterion. When a doubler plate is required, it is designed based on a required shear force V_{pz} in the panel zone:



(a) Continuity Plate (b) Doubler Plate

Fig. 2 –Free Body Diagrams

$$V_{pz} = M_f / d^* \tag{7a}$$

where M_f and d^* are the beam flange moment and distance between flange centers, respectively. The design strength of the panel zone is given by:

$$R_n = 0.60 F_y d_c t_{cw} \left(1 + \frac{3 b_{cf} t_{cf}{}^2}{d_b d_c t_{cw}} \right) \tag{7b}$$

The vertical welds of a doubler plate are designed to resist the appropriate proportion of the panel zone shear based on the relative elastic shear stiffness of the doubler plate:

$$V_{dp} = \frac{G t_{dp}}{G t_{dp} + G t_{cw}} = \frac{t_{dp}}{t_{pz}} \tag{8}$$

where t_{dp} and t_{pz} are the thicknesses of the doubler plate and total panel zone, respectively, and G is the shear modulus. Vertical force in the doubler plate, $V_{dp,v}$, is calculated from moment equilibrium:

$$V_{dp,v} = V_{dp} d^* / d_c \tag{9}$$

The corresponding shear flow equals $V_{dp,v}$ divided by the total weld length between beam flanges, d^* (see Fig. 2). Since the panel zone strength includes extra strength [the second term in Eq. (7b)] from the hinging of the column flanges [14], this equation may result in a predicted shear force in excess of the shear strength of the doubler plate. In this scenario, the shear flow is set to the yielding shear flow of the doubler plate and the design philosophy converges to the existing provisions in AISC 341. Specimen C7 utilized this design basis with a fillet weld of 11 mm versus the 22 mm required fillet weld to develop the shear strength of a 19 mm plate.

The experimental testing program involved 10 full-scale specimens and consisted of six one-sided RBS connections and four two-sided WUF-W subassemblies. Three specimens (C3, C4, and C7) were designed to challenge Eq. (3). The remaining specimens utilized continuity plates designed as per Eqs. (4) and (5). Most specimens, except Specimens C6 and C6-G, used a pair of fillet welds with a proposed weld size equal to 3/4 of the continuity plate thickness. Specimen C5 intentionally used a continuity plate with a width-to-thickness of 16 to violate Eq. (6). Fig. 3 shows a representative continuity plate and RBS detail. Specimen C6-G was a nominally identical specimen to C6, except that the members were hot-dip galvanized prior to welding the specimen together. Two of the WUF-W specimens used a conventional vertical groove weld for the vertical doubler plate weld, while the other two used fillet welds. An important consideration for fillet welded doubler plates is maintaining the fillet weld throat through the edge of the plate which has been beveled to clear the radius of the column web-to-column flange junction. Fig. 4 demonstrates the typical fillet welding of the doubler plate and continuity plate. Table 1 shows the test matrix.

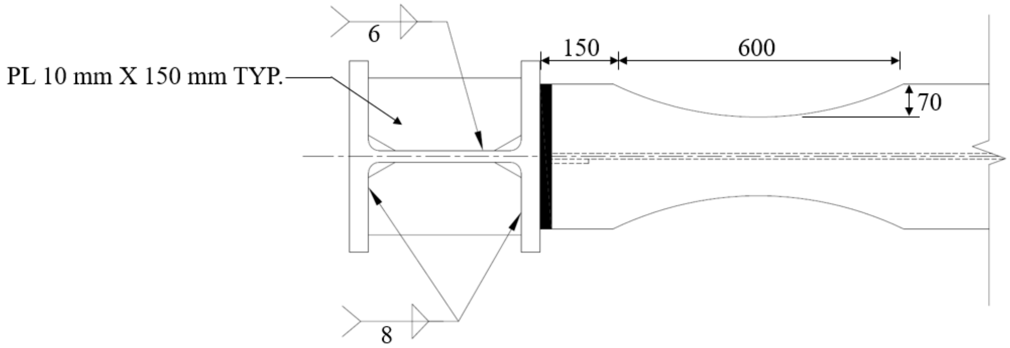
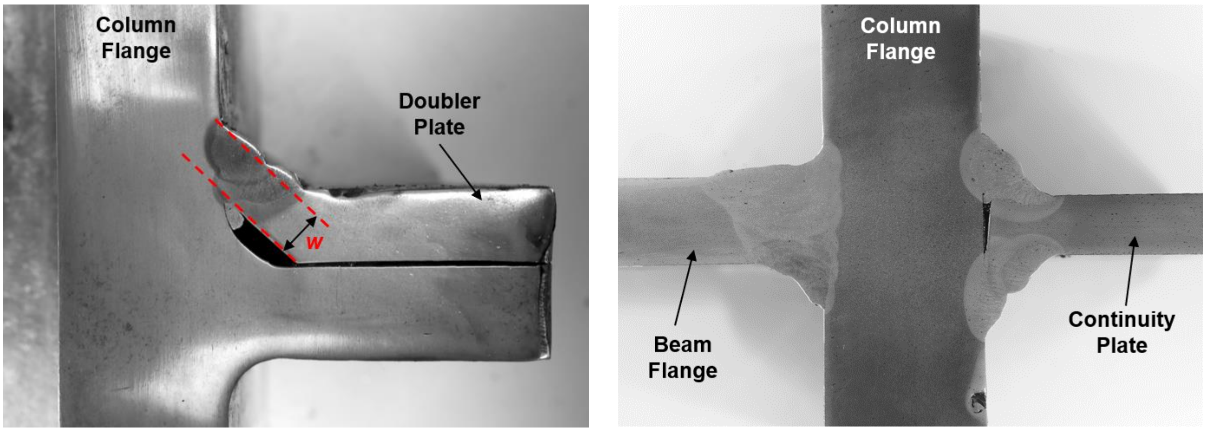


Fig. 3 – Specimen C5 Continuity Plate and RBS Detail

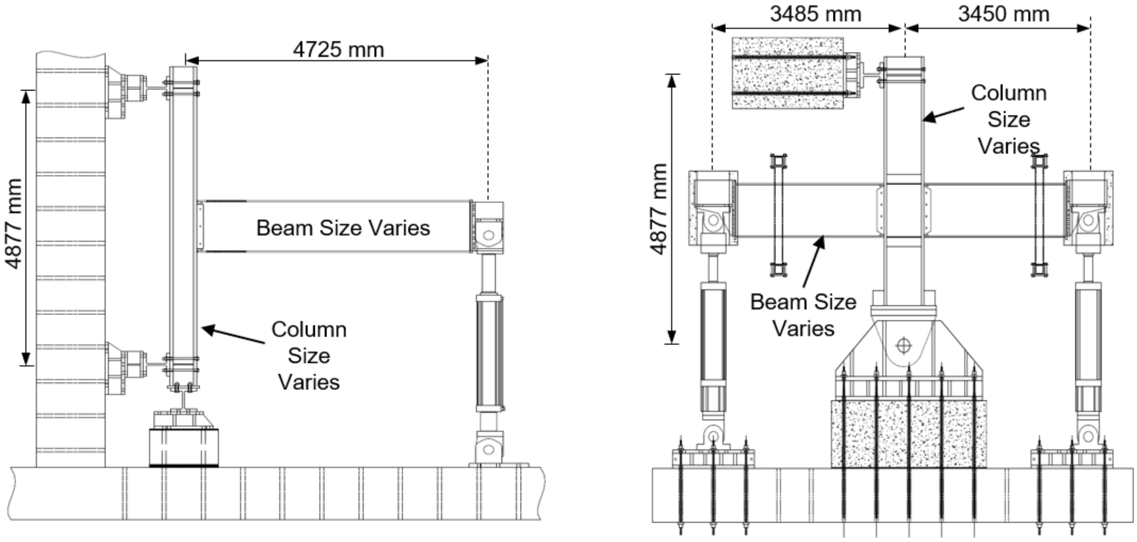


(a) Doubler Plate (b) Continuity Plate

Fig. 4 – Fillet Weldments

4. Test Program

The test setup for the exterior and interior specimens is shown in Fig. 5. The testing was conducted in accordance with Section K2 of AISC 341 using the standard loading protocol (see Fig. 6). The exterior connections were tested in the upright position with a single 1000-kN hydraulic actuator. Frame inflection points were assumed to exist at the mid-height of each story which were simulated by W-shape hinges. The interior connections were tested in the horizontal position with a single W-shape hinge at the top of the column and a steel clevis on the bottom end. In this setup, two 2200-kN hydraulic actuators were attached to the ends of both beams. In both test setups, the actuator is affixed to the beam through a bolted loading corbel. The actuators are loaded in a displacement-control mode to achieve the target drift levels of Section K2. In the case of the interior connection, the actuators displace in opposite directions. Lateral bracing is provided to both setups as per the requirements of AISC 341 for highly ductile members.



(a) One-Sided Specimens (Elevation View) (b) Two-Sided Specimens (Plan View)

Fig. 5 – Test Setup

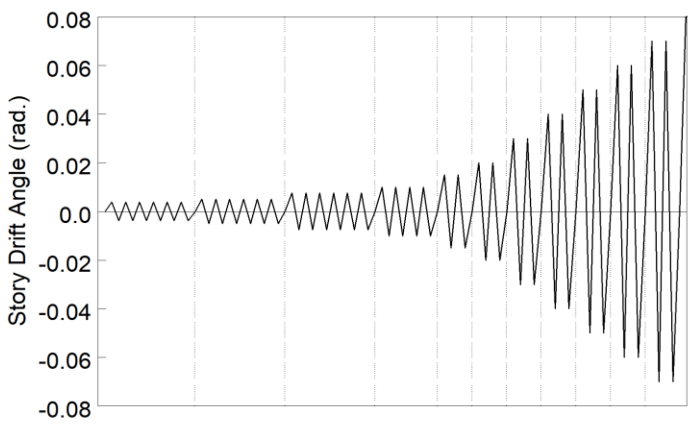


Fig. 6 – Load Protocol

The test specimens were fabricated by a commercial fabricator from A992 steel for the rolled shapes and A572 Gr. 50 plates for the stiffeners. The beam-to-column welding was performed as a simulated field weld to replicate actual conditions. Beam flange CJP welds used an E70T-6 (Lincoln Electric NR-305) electrode, while the beam web used an E71T-8 (Lincoln Electric NR-232) electrode. The continuity and doubler plates were welded with an E70T-9C (Lincoln Electric OSXLH-70) electrode. All electrodes satisfy the requirements of AWS D1.8 for Demand Critical Welds.

A combination of displacement transducers, strain gauge rosettes, and uniaxial strain gauges were used to measure global and local responses. Measurement devices were arranged such that the beam tip displacement could be divided into the three main components (1) transducers across the panel zone measure the shear deformation of the panel zone, (2) transducers coplanar with the column web on the surface of the column flange measure the column rotation, and (3) a string potentiometer measures the beam tip displacement directly.

Table 1 – Test Matrix

Spec. No.	Beam	Column	Connection Type	Continuity Plate Thickness (mm)	Research Objective
C3	W36×150	W14×257	One-sided RBS	-	Specimen violates Eq. (3). Strength Limit states predict plate not required.
C4	W30×116	W27×235	One-sided RBS	-	Specimen violates Eq. (3). Strength Limit states predict plate not required.
C5	W36×150	W14×211	One-sided RBS	9.5	Size of continuity plate designed as per Eq. (5). Column designed to have a weak panel zone to exacerbate column kinking. Continuity plate violates Eq. (6). Continuity plate welds designed as the per the w = (3/4) t rule.
C6	W30×116	W24×176	One-sided RBS	12.7	Size of continuity plate designed as per Eq. (5). Welds conservatively designed such that $w = t$
C6-G	W30×116	W24×176	One-sided RBS	12.7	Identical to Specimen C6, except that all members were hot dip galvanized prior to welding.
C7	W30×116	W24×192	One-sided RBS	-	Specimen violates Eq. (3). Size of doubler plate to satisfy WLY limit state. FLB limit state satisfied without stiffening. Welds designed according to Eq. (7) to Eq. (9).
W1	W36×150	W27×258	Two-sided WUF-W	12.7	Size of continuity plate designed as per Eq. (5). Continuity plate welds designed as per the w = (3/4) t rule.
W2	W33×141	W27×217	Two-sided WUF-W	19.0	Size of continuity plate under designed as per Eq. (5). (DCR=1.16). Continuity plate welds designed as per the w = (3/4) t rule.
W3	W30×116	W24×207	Two-sided WUF-W	12.7	Size of continuity plate designed as per Eq. (5). Extended doubler plate welded with vertical fillet welds to develop shear capacity. Continuity plate welds designed as per the w = (3/4) t rule.
W4	W24×94	W24×182	Two-sided WUF-W	19.0	Size of continuity plate under designed as per Eq. 2. Doubler plate welded with vertical fillet welds to develop shear capacity. Continuity plate welds designed as per the w = (3/4) t rule.

5. Specimen Performance

All the specimens completed the AISC prequalification for SMF. Specifically, all the specimens completed at least one cycle of 0.04 rad drift without the beam flexural strength at the column face degrading below $0.8M_{pn}$, where M_{pn} is the nominal plastic moment of the beam. Fig. 7a depicts the typical response of a specimen showing the successful completion of the prequalification. A second axis on the right hand side of the figure shows the beam moment at the face of the column normalized by M_{pn}. Fig. 7b demonstrates the inelastic response of the specimen after removing the elastic component.

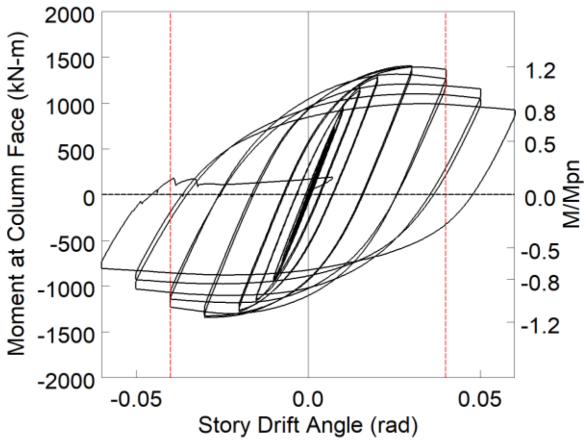
Table 2 – Test Results

Spec. No.	Beam	Column	Cycle at Failure	Failure Mode
C3	W36×150	W14×257	1st of 0.05 rad	Beam Top Flange CJP Weld
C4	W30×116	W27×235	1st of 0.06 rad	RBS Fracture
C5	W36×150	W14×211	2nd of 0.05 rad	Beam Top Flange CJP Weld
C6	W30×116	W24×176	1st of 0.05 rad	Beam Top Flange CJP Weld
C6-G	W30×116	W24×176	1st of 0.06 rad	RBS Fracture
C7	W30×116	W24×192	2nd of 0.05 rad	RBS Fracture
W1	W36×150	W27×258	2nd of 0.04 rad	Beam Top Flange CJP Weld
W2	W33×141	W27×217	2nd of 0.06 rad	Beam Top Flange CJP Weld
W3	W30×116	W24×207	2nd of 0.06 rad	Beam Top Flange CJP Weld
W4	W24×94	W24×182	1st of 0.05 rad	Beam Top Flange CJP Weld

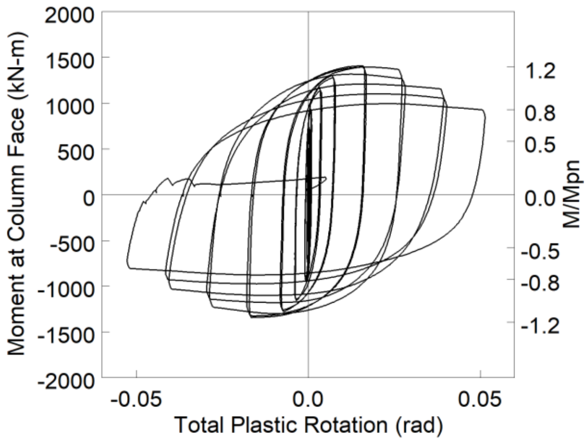
Specimens C6 and C6-G had nearly identical responses until fracture of Specimen C6 during the first cycle of 0.05 rad drift. In contrast, Specimen C6-G fractured during the first cycle of 0.06 rad drift. Table 2 documents the performance of each connection. No damage to any of the continuity or doubler plate weldments occurred during testing. The high width-to-thickness (16.0) of the continuity plates used in Specimen C5 demonstrated instability (see Fig. 8).

The one-sided connections failed either by fracture of the beam flange within the reduced beam section or failure of the top flange CJP weld. Specimens, including those with and without continuity plates, which ultimately failed due to weld fracture demonstrated early signs of ductile weld tearing during the initial 0.03 rad cycle drifts. During each negative excursion where the top flange was in tension, the weld tear progressed until the complete fracture of the weld. The weld tears started in the center of the beam flange at the toe of a prominent weld pass in the reentrant corner. The typical fracture was a ductile shear fracture that propagated at a 35-degree angle through the weld metal until a cleavage fracture occurred.The exterior specimens which ruptured through the beam flange at the reduced beam section developed fractures in the vicinity of the largest local buckling amplitudes.

The interior connections all fractured through the beam top flange CJP weld. This fracture developed at the CJP weld root where a notch condition exists at the junction between weld metal and steel backing. The initiation of this fracture occurred during the 0.03 rad drift cycles, and its gradual progression occurred through the weld metal along the CJP weld bevel. Fig. 9 shows the beam top flange fractures of the one-sided and two-sided specimens. The final fracture surfaces resulted in a mixture of shear fracture and cleavage. Following fractures of the beam flange CJP weld, the interior specimens developed tears at the weld access holes propagating into the beam web. The normalized energy dissipation and the distribution between yielding components of each specimen are shown in Fig. 10. Specimen C5 intentionally used a weak panel zone which violated Eq. (7b) to exacerbate column kinking. Comprehensive reporting for the test program described herein is available [15].



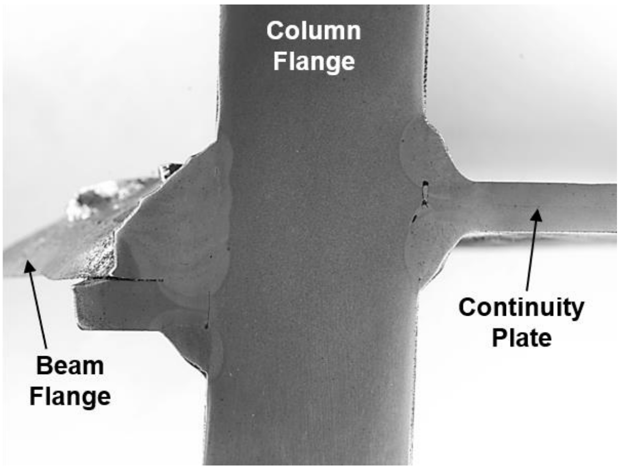
(a) Load-Displacement



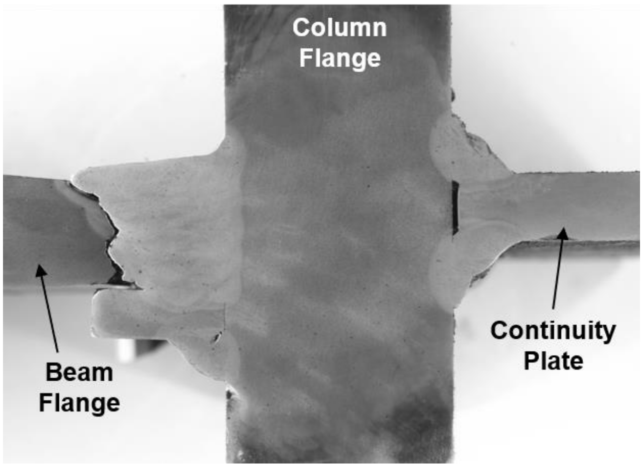
(b) Plastic Rotation

Fig. 7 – Specimen C4 Response

Fig. 8 – Continuity Plate Instability (Specimen C5)



(b) One-sided Connection (Specimen C5)



(b) Two-sided Connection (Specimen W1)

Fig. 9– Beam Flange CJP Weld Failures

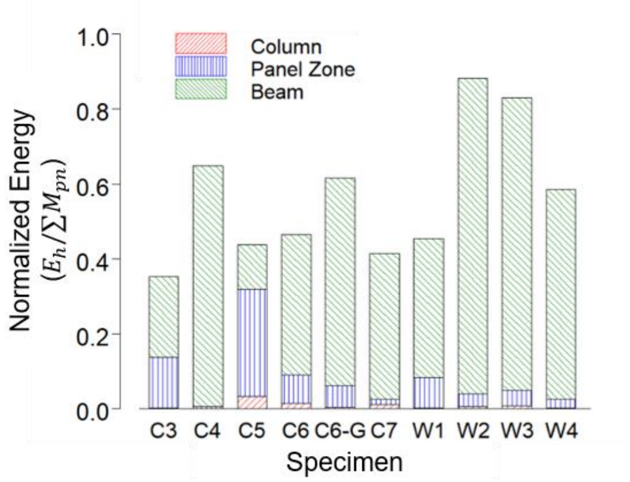


Fig. 10– Normalized Energy Dissipation Capacity

6. Conclusions

Cyclic testing of ten full-scale steel moment connections was conducted to evaluate the efficacy of economized continuity and doubler plate weld details. Six of the specimens were exterior (one-sided) RBS connections, and the remaining four were interior (two-sided) WUF-W connections. Both of these connections are currently prequalified in AISC 358. All ten specimens violated the current detailing requirements of these connections while successfully satisfying the prequalification criteria of SMF. Including other recently tested SMF connections [9], a total of nine subassemblies, including both RBS and WUF-W connections, have been tested with fillet welded continuity plates. No damage to the fillet welds has been observed. From the experimental testing program, the following conclusions can be drawn.

1. Continuity plate fillet welds sized to develop the yield strength of the continuity plate such that the weld size, *w*, is equal to 3/4 the thickness of the continuity plate are adequate. This result has been tested for interior and exterior connections utilizing either an RBS or WUF-W connection.

2. Experimental results and parametric finite element analysis confirm that using the current design approach in AISC 341 to size the continuity plate leads to the yielding of the continuity plate. This justifies the (3/4)*t* design rule which capacity protects the fillet welds from yielding of the continuity plate.

3. Based on the single specimen (Specimen C7) the efficacy of using fillet welds sized for the proportion of shear flow in the panel zone for the vertical doubler plate welds has been shown. These welds are significantly smaller than the current requirement that these welds develop the shear strength of the doubler plate. The design of these welds was based on Eqs. (8) and (9).

4. The comparison between Specimens C6 and C6-G reveals no adverse effects due to hot-dip galvanization of the members prior to welding.

5. Continuity plates are prone to buckling if the width-to-thickness ratio of Eq. (6) is not satisfied.

6. Based on three specimens (C3, C4, and C7) the minimum column flange thickness of Eq. (3) appears to be conservative for the relatively lightly loaded RBS style of connection.

7. Acknowledgments

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