A Flexibility-Based Formulation for the Design of Continuity Plates in Steel Special Moment Frames

ANDY T. TRAN, PATRICK M. HASSETT and CHIA-MING UANG

ABSTRACT

This paper introduces a rational approach for the design of continuity plates and associated welds in steel special moment frame (SMF) connections. The current AISC *Seismic Provisions* require welds attaching continuity plates to develop the full strength of the plate, resulting in the need to use complete-joint-penetration (CJP) groove welds. The combination of continuity plate thickness requirements, welding process and weld inspection often leads to costly detailing that may be overly conservative. The proposed design procedure, which is based on the relative flexibility between the column flange and continuity plate, aims to quantify the seismic force demand on continuity plates, thus allowing designers to efficiently size both the continuity plate thickness and the associated welded joints. In addition, the design procedure may allow the use of fillet welds or partial-joint-penetration groove welds as opposed to CJP welds, leading to a more economical design and fabrication. Formulation of the design procedure through analytical studies, including finite element analysis, is outlined.

Keywords: special moment frame, continuity plate, relative flexibility, RBS, WUF-W.

INTRODUCTION

Continuity plates are commonly used in moment connections of steel moment frames to increase the local strength and stiffness of the column flange and web (see Figure 1). These transverse plates are attached to the column. They are utilized in both gravity and lateral load applications. The basic design requirements for continuity plates have been provided in the AISC *Specification for Structural Steel Buildings* (AISC 2005a, 2010a), hereafter referred to as AISC *Specification*. For seismic applications, additional requirements are provided in the AISC *Seismic Provisions for Structural Steel Buildings* (AISC 2005b, 2010b) and the AISC *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (AISC 2005c, 2010c), hereafter referred to as AISC *Seismic Provisions* and AISC 358, respectively.

Current seismic design provisions take a conservative approach for sizing and attaching continuity plates to the column due to the lack of a rational procedure to apportion the beam flange force to the continuity plate welded joints and limitations in tested geometries. For seismic design applications, this leads to continuity plates that maybe thicker than necessary and require complete-joint-penetration (CJP) welds to the column flanges, detailing that adds cost to fabrication and quality control. Thus, there is a need for the development of a rational design methodology such that a more economical continuity plate and weld design can be achieved.

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BACKGROUND

Section J10 of the AISC *Specification* provides requirements for continuity plate design for gravity, wind, and low seismic load applications. AISC *Seismic Provisions* and AISC 358 provide additional requirements for seismic load applications. The application of these design requirements are well documented in AISC Design Guide 13, *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* (Carter, 1999). These documents contain equations to compute the force demand and column limit state capacities that determine whether continuity plates are required. If required, these documents also provide requirements to size and detail continuity plates. A review of these equations and requirements are presented in the following sections.

Concentrated Beam Flange Force

Figure 2 shows concentrated beam flange forces acting on a column. Consider cases (a) and (b) where beams frame into the column from both sides. Assuming $P_{uf,1} = P_{uf,2} = P_{uf}$ under the gravity load case, the flow of stress from beam 1 to beam 2 is relatively direct and uniform (see Figure 3a). The continuity plate welded joint to the column flange (defined hereafter as "flange weld") is required to transmit part of the tensile force, P_{uf}; however, there is a negligible force transferred from continuity plate to the column web. In Figure 3b, a different flow of stress is shown under a lateral load case. Because one loaded edge of the continuity plate is in tension while the other edge is in compression, shear forces exist along the web edge. To satisfy equilibrium, two additional shear forces along the loaded column flange edges also result. These transverse shear forces, which are not trivial in magnitude, are not numerically addressed in current design.

According to AISC Design Guide 13 (Carter, 1999), the concentrated beam flange force, P_{uf}, is calculated as shown in Equation 1:

$$P_{uf} = \frac{M_u}{d_m} \tag{1}$$

where M_u = beam end moment and d_m = moment arm between the flange centroids ($d_b - t_{bf}$). Note that Equation 1 assumes the entire beam moment is transferred to the column through the beam flanges. While this assumption may be reasonable for the more flexible, bolted beam web connection, it may be too conservative for moment connections where the beam web is welded directly to the column flange. This issue will be discussed later. The demand from the beam flange force is checked against a series of limit states to determine the need for continuity plates.

Limit States and Design Strengths

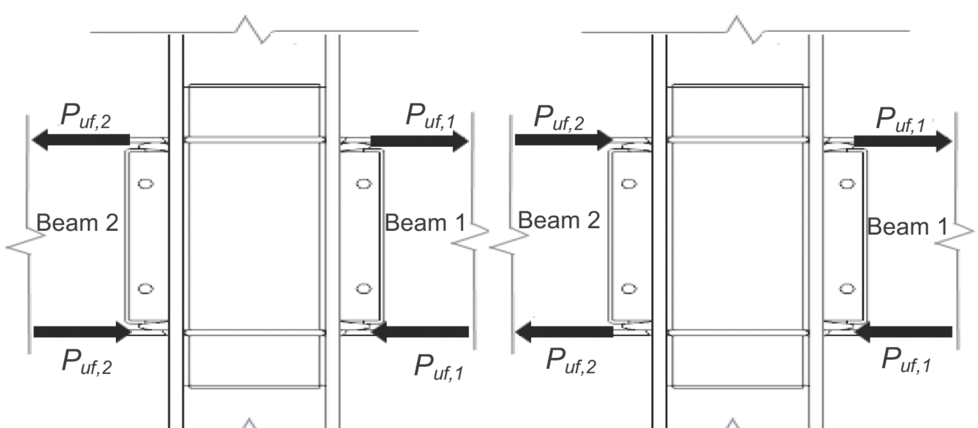
In continuity plate design for a lateral load case, the following two limit states need to be checked per AISC *Specification*.

(1) *Flange Local Bending (FLB) of Column*

When an unstiffened column flange is pulled out-of-plane by the tensile beam flange force, P_{uf}, stress concentrations in the beam flange weld will occur due to the differential

(a) (b)

Fig. 1. RBS moment connection: (a) with continuity plates; (b) without continuity plates.



(a) (b)

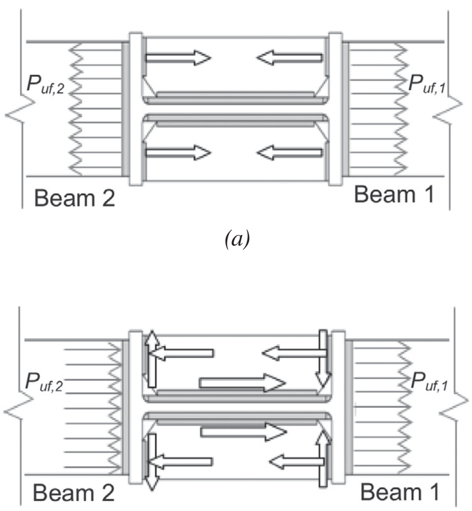
Fig. 2. Moment connection flange forces: (a) gravity load case; (b) lateral load case (adapted from Carter, 1999).

stiffness across the column flange width (see Figure 4a). To minimize the effect of stress concentration, AISC *Specification* specifies the following design strength for FLB beyond which continuity plates are required:

$$\phi R_n = \phi 6.25 t_{cf}^2 F_{yc} \tag{2}$$

where $\phi = 0.9$, t_{cf} = column flange thickness and F_{yc} = specified minimum yield stress of the column.

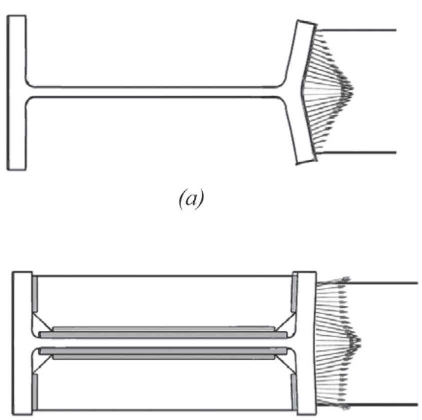
To prevent weld fracture due to stress concentrations, Equation 2 was derived based on a limit state defined by a ¼-in. relative deformation between two opposing column flanges. A recent study by Hajjar et al. (2003) observed that this equation is conservative for design.



(a)

(b)

Fig. 3. Stress flow in continuity plates: (a) gravity load case; (b) lateral load case.



(a)

(b)

Fig. 4. Beam flange stress distribution: (a) unstiffened column flange; (b) stiffened column flange.

(2) *Web Local Yielding (WLY) of Column*

AISC *Specification* assumes the concentrated beam flange force, P_{uf}, is transmitted to the web of an unstiffened column as shown in Figure 5. The associated design strength is

$$\phi R_n = \phi(5k + N) F_{yc} t_{cw} \tag{3}$$

where ϕ = 1.00, k = distance from the outer face of the column flange to the web toe of the fillet, N = beam flange thickness and t_{cw} = column web thickness.

Required Strength for Continuity Plates and Welds

When the concentrated beam flange force, P_{uf}, exceeds either the FLB or WLY limit state strengths, a pair of continuity plates is needed to strengthen and stiffen the column. Here, $\phi R_{n(\min)}$ is denoted as the lesser of the design strengths for FLB and WLY limit states. The AISC *Specification* specifies the required strength for a pair of continuity plates as

$$R_{u(st)} = P_{uf} - \phi R_{n(min)} \tag{4}$$

AISC Design Guide 13 notes that Equation 4 is a simplified approach, whereby only the force in excess of the governing limit state strength is assumed to be transmitted to the continuity plates. In an exact solution, the design guide also noted that "…this force would be apportioned between the web and transverse stiffeners on the basis of relative stiffness and effective area" (Carter, 1999). It will be shown in subsequent sections that the simplified approach can lead to a large difference between the required design strength, $R_{u(st)}$, and the actual force transmitted to the continuity plates.

Each full-depth continuity plate is welded to the column on three sides, with two flange welds and one web weld. It is critical to evaluate the required strength of these welds

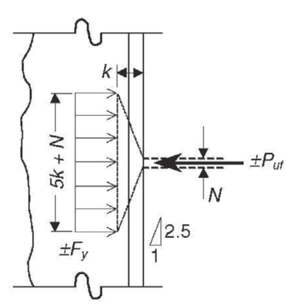


Fig. 5. Local force transfer for WLY limit state (Carter, 1999).

properly because a conservative estimate can lead to expensive welds (e.g., CJP groove welds). The past and current procedures for welding of continuity plates detailed in the AISC *Specification* is summarized later.

Section K1 of the AISC LRFD *Specification* (AISC, 1999) specifies a conservative approach to design continuity plate flange welds that require them to develop the welded portion of the stiffener. This conservatism is unavoidable when using this simplified approach because the actual force transmitted to the continuity plates is not calculated by a method that considers the relative stiffness. Section K1 of the AISC LFRD *Specification* requires the web weld to transmit the unbalanced force in the stiffener to the web.

The requirement for flange weld design is relaxed in the AISC *Specification* (2005a, 2010a) for nonseismic applications, which stipulates in Section J10 that the required strength is the difference between the beam flange force and available strength (i.e., Equation 4). Thus, the required design strengths for both continuity plates and flange welds are the same; flange welds, therefore, do not have to develop the flange welded portion of the continuity plates.

Seismic Design Provisions for Continuity Plates

For seismic applications, additional provisions are presented in the AISC *Seismic Provisions*. For pre-Northridge-type moment connections that feature welded beam flanges and a bolted web, the 1992 AISC *Seismic Provisions* assume the maximum beam moment developed at the face of the column is 1.3 times the nominal plastic moment of the beam. The 1.3 factor is used to account for the effect of material overstrength and cyclic hardening. Furthermore, it assumes that the flexible, bolted beam web is ineffective in transferring moment to the column. Assuming the flange-only plastic sectional modulus, $Z_f\left(\approx b_{bf}t_{bf}d_b\right)$, is approximately 70% of the beam plastic sectional modulus, Z_x, the concentrated tensile beam flange force is thus computed as

$$
\begin{aligned}
P_{uf} &= \frac{M_u}{d_m} \approx \frac{1.3M_p}{d_b} = \frac{1.3\left(Z_x F_{yb}\right)}{d_b} = \frac{1.3\left(Z_f / 0.7\right)F_{yb}}{d_b} \\
&= \frac{1.3\left(b_{bf}t_{bf}d_b/0.7\right)F_{yb}}{d_b} = 1.8b_{bf}t_{bf}F_{yb}
\end{aligned}
\tag{5}
$$

To check FLB, the 1992 AISC *Seismic Provisions* use Equation 5 as the required strength, P_{uf}, and Equation 2 as the available strength, but with $\phi = 1.00$. Therefore, continuity plates are not required for the FLB limit state if the following condition is satisfied:

$$6.25t_{cf}^2F_{yc} \geq 1.8b_{bf}t_{bf}F_{yb} \tag{6a}$$

or

$$t_{cf} \geq 0.4\sqrt{\frac{1.8b_{bf}t_{bf}F_{yb}}{F_{yc}}} \tag{6b}$$

$$= 0.54\sqrt{\frac{b_{bf}t_{bf}F_{yb}}{F_{yc}}} \tag{6c}$$

Because of the damage of moment connections observed after the 1994 Northridge, California, earthquake, the 1997 and 2002 AISC *Seismic Provisions* simply stated, "[C]ontinuity plates shall be provided to match the tested specimens." Based on a study conducted after the Northridge earthquake by the SAC Joint Venture (a project headed by SEAOC, ATC, and CUREE), FEMA-350 (2000a) recommends that Equation 7, which is a slightly modified form of Equation 6b to account for the difference between nominal and expected yield stresses, be used for special moment frame (SMF) design:

$$t_{cf} \geq 0.4\sqrt{\frac{1.8b_{bf}t_{bf}R_{yb}F_{yb}}{R_{yc}F_{yc}}} \tag{7}$$

where R_{yb} and R_{yc} are the ratio of the expected yield stress to specified minimum yield stress for the beam and column, respectively. That is, Equation 7 implies a beam flange force as in Equation 5 except that F_{yb} is replaced by $R_{yb}F_{yb}$. It is interesting to note that FEMA-355D (FEMA, 2000c) commented that Equation 7 is "…not a precise indicator of the need for continuity plates or of connection performance. There is room for considerable improvement in the continuity plate design requirements." In the SAC study on continuity plates, one welded unreinforced flange-welded (WUF-W) web moment connection tested by Ricles et al. (2000) provided satisfactory performance, although Equation 7 was not satisfied.

Equation 7 is the same as Equation 6b if the same grade of steel (e.g., ASTM A992 steel) is used for both the beams and columns. Both equations are based on a conservative assumption that the beam web is ineffective in transferring moment, and the beam flanges transfer 1.3 times the nominal plastic moment at the face of the column. For application to prequalified SMF moment connections (AISC, 2010c) where the web is fully welded to the column flange [e.g., reduced beam section (RBS) and WUF-W connections], the implied concentrated beam flange force used to determine the need for continuity plates may be too high. It is also noted that FEMA 350 (2000a) recommends another requirement based on the research of Ricles et al. (2000):

$$t_{cf} \geq \frac{b_{bf}}{6} \tag{8}$$

For SMF design, the required force for continuity plate design is not quantified as in Equation 4. Instead, a prescriptive procedure is used to determine the thickness of the continuity plates. When required, FEMA 350 (2000a) recommends that the thickness of continuity plates satisfy the following requirements:

- For one-sided (exterior) connections, continuity plate thickness should be at least one-half of the thickness of the two beam flanges.
- For two-sided (interior) connections, the continuity plates should be equal in thickness to the thicker of the two beam flanges on either side of the column.

The recommended design procedure outlined in FEMA 350 (2000a) has been adopted by the 2005 and 2010 AISC 358 and is promulgated in the 2010 AISC *Seismic Provisions* for SMF design.

AISC 358 takes a conservative approach for weld design, requiring that the welds develop the strength of the continuity plates. The requirements are as follows:

- Continuity plates, if provided, shall be welded to column flanges using CJP groove welds.
- Continuity plates shall be welded to column webs using groove welds or fillet welds. The required strength of the sum of the welded joints of the continuity plates to the column web shall be the smallest of the following:
  - The sum of the design strengths in tension of the contact areas of the continuity plates to the column flanges that have attached beam flanges.
  - The design strength in shear of the contact area of the plate with the column web.

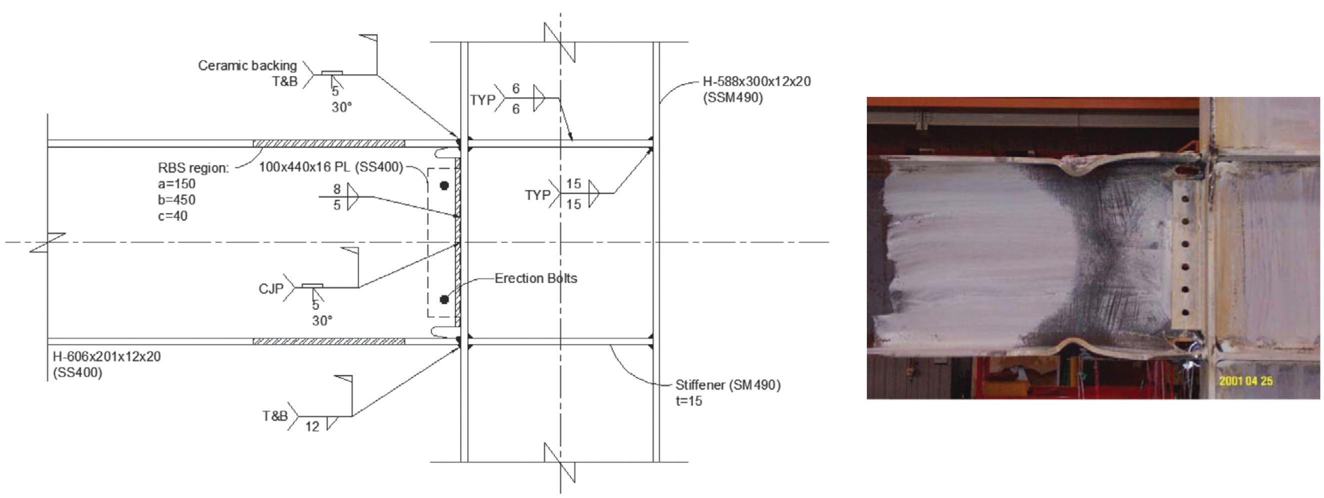
The current continuity plate and weld design requirements indicate that the design procedure may be more conservative than necessary. Thus, a rational approach that considers the relative stiffness to apportion the concentrated beam flange force to the continuity plates such that the welded joint can be properly and economically designed is desirable.

Experimental Evidence

A significant number of full-scale moment connections have been tested as a result of the 1994 Northridge, California, earthquake. A comprehensive summary of moment connection testing programs (pre- and post-Northridge) that feature either fillet or CJP welded continuity plates are provided by Hajjar et al. (2003). Recommendations from these studies suggest that fillet-welded continuity plates may provide adequate performance for seismic and nonseismic applications. In addition, a testing program conducted by Lee et al. (2005) featured two out of a total of eight full-scale RBS specimens with continuity plates that were fillet welded to the column (see Figure 6). Both specimens achieved an interstory drift angle of 0.04 radian with no observed failure in the continuity plate welds. Therefore, it is not always necessary to use CJP welds to connect the continuity plates to the column.

FORCE DEMAND ON CONTINUITY PLATES: A PARAMETRIC STUDY

To identify significant factors affecting the force demand on continuity plates, parametric nonlinear, finite element analyses (FEAs) were performed for an interior (two-sided) and exterior (one-sided) WUF-W moment connection (Uang, Tran and Hassett, 2011). The nonlinear FEA software



(a) (b)

Fig. 6. RBS moment connection with fillet-welded continuity plates: (a) connection details (in milimeters); (b) yielding pattern and deformed configuration (Lee et al., 2005).

ABAQUS was used (ABAQUS, 2005). Two base models were first established, then an additional nine interior connection cases and seven exterior connection cases were created (see Table 1) in which the continuity plate, column flange or column web were varied in thickness. For these analysis cases, four-node, thick-shell brick elements were used with mesh size ranging from 0.5 in. in the connection region to about 2 in. in the outer regions. A piecewise linear material model from FEMA (2000b) was used with a yield stress, F_{yn} = 50 ksi (see Figure 7).

A WUF-W specimen tested by Ricles et al. (2000) was used for the interior base model. The test specimen featured W36×150 beams along with a W14×398 column, all of A572 Grade 50 steel. The specimen simulated an interior connection in a SMF with a bay width of 29.5 ft and a story height of 13 ft. Column reinforcement included two ¾-in.-thick doubler plates and 1-in.-thick continuity plates. Figure 8 shows the finite element mesh (FEM) of the specimen, and Figure 9 shows the mesh at the connection. Assuming inflection points at the midspan of the beams and mid-height of the column, the free end of the beams were supported by simulated horizontal rollers. The base of the column was pin supported, and the top end of the column was loaded by a horizontal actuator to impose a monotonic displacement load. Lateral bracing of the beams was provided 10 ft from the column centerline.

The exterior WUF-W base model was designed in accordance with the AISC *Seismic Provisions*. A W33×130 beam was selected with a span of 14.75 ft (equal to half of the 29.5-ft bay width). A 13-ft-long W24×192 deep column was chosen to investigate the effects of thinner flanges. The connection features a pair of ¼-in.-thick doubler plates along with ⅞-in.-thick continuity plates to satisfy the FLB limit state. The column was pin-supported at both ends, and a load was applied to the end of the beam. The beam included lateral bracing of the flanges 10 ft from the column centerline. ASTM A992 steel was specified for the beam and column.

For each of the 16 parametric cases, the beam flange force, P_{uf}, at the face of the column is computed by integrating the tensile stresses of the bottom beam flange (beam 1 for interior connections) across its width. The forces acting on the flange weld edge of the continuity plates are also computed in a similar manner. The percentage of the beam flange force allocated to a pair of continuity plates are computed at 0.5 and 3% interstory drift to compare elastic and inelastic force distribution, respectively (inelastic force demand at 4% interstory drift tends to be lower for the beams used in this parametric study and thus were not used).

Effect of Continuity Plate Thickness

The continuity plate thickness for the interior base model was 1 in. Two additional cases corresponding to 33 and 67% of the base model thickness were considered. The continuity plate thickness for the exterior base model was ⅞ in. Three additional exterior cases were analyzed with a continuity plate thickness equal to 50, 75 and 125% of the exterior base model.

Figure 10 shows the percentage of the concentrated beam flange force that is transmitted to the continuity plates for the interior and exterior cases. It is observed that the percentage of the normal force acting on the flange weld increases with an increase continuity plate thickness; that is, thicker continuity plates attract more force from the beam flange.

Effect of Column Flange Thickness

The thickness of the column flange was 2.85 and 1.46 in. for the interior and exterior base models, respectively. Three

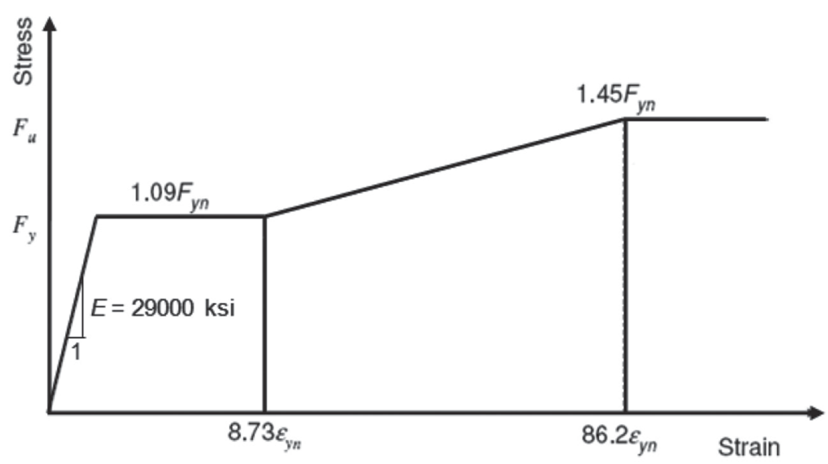


Fig. 7. Assumed steel stress-strain relationship.

Table 1. Cases for Parametric Study								
(a) Interior WUF-W Moment Connections								
	Continuity Plate Thickness		**Column Flange Thickness**		**Column Web Thickness**		**Strong Column–Weak Beam**	**Panel Zone Strength**
Case	(in.)	(%)	(in.)	(%)	(in.)	(%)	$\sum M^*_{pc} / \sum M^*_{pb}$	$\phi R_n / R_u$
I-1†	1.00	100	2.85	100	1.77	100	1.08	0.96
I-2	0.67	66.7	2.85	100	1.77	100	1.08	0.96
I-3	0.33	33.3	2.85	100	1.77	100	1.08	0.96
I-4	1.00	100	3.56	125	1.77	100	1.32	0.96
I-5	1.00	100	2.14	75	1.77	100	0.85	0.96
I-6	1.00	100	1.43	50	1.77	100	0.62	0.96
I-7	1.00	100	2.85	100	2.21	125	1.11	1.10
I-8	1.00	100	2.85	100	1.33	75	1.06	0.83
I-9	1.00	100	2.85	100	0.89	50	1.04	0.69
(b) Exterior WUF-W Moment Connections								
	Continuity Plate Thickness		**Column Flange Thickness**		**Column Web Thickness**		**Strong Column–Weak Beam**	**Panel Zone Strength**
Case	(in.)	(%)	(in.)	(%)	(in.)	(%)	$\sum M^*_{pc} / \sum M^*_{pb}$	$\phi R_n / R_u$
E-1†	0.88	100	1.46	100	0.81	100	1.68	1.37
E-2	1.09	125	1.46	100	0.81	100	1.68	1.37
E-3	0.66	75	1.46	100	0.81	100	1.68	1.37
E-4	0.44	50	1.46	100	0.81	100	1.68	1.37
E-5	0.88	100	1.83	125	0.81	100	2.02	1.37
E-6	0.88	100	1.10	75	0.81	100	1.35	1.37
E-7	0.88	100	0.73	50	0.81	100	1.02	1.37

† Refers to base model.

additional cases were considered for each base model with a column flange thickness equal to 125, 75 and 50% of the base model thickness. It is recognized that according to AISC *Seismic Provisions,* reducing the column flange thickness may violate some requirements (e.g., strong column–weak beam condition), and increasing the column flange thickness may result in a condition where continuity plates are no longer required. However, the purpose of this parametric study was primarily to evaluate the effect of column flange thickness on the force demand in continuity plates.

Figure 11 shows the percentage of the beam flange force that is transmitted to the continuity plates for the interior and exterior cases. The results show that an increase in column flange thickness decreases the force demand to continuity plates. It is also observed that at 3% interstory drift, the forces apportioned into the continuity plate varied from 35 to 87% of the beam flange force for the interior case and from 71 to 88% for the exterior case.

Effect of Column Web Thickness

The thickness of the column web was 1.77 in. for the interior base model. Three additional cases were considered with a column web thickness of 125, 75 and 50% of the interior base model web thickness. The results are shown in Figure 12. As expected from Equation 3 for the WLY limit state, a thicker column web will reduce the force demand on continuity plates. The percentage of force to the continuity plate, however, is shown to not be sensitive to variance in the column web thickness at the elastic level (0.5% interstory drift) and only marginally sensitive at inelastic levels (3% interstory drift). Thus, no similar parametric study was conducted for the exterior base model.

STRESS DISTRIBUTION ON FLANGE AND WEB WELDS

Figure 13a shows the normal stress distribution along the flange weld at 3% drift for the interior base model (case I-1). The figure shows that the normal stress is the highest near the column flange tip. Because the stress distribution is not uniform, to compute the maximum tensile stress, f_{max}, occurring near the column flange tip, it is necessary to idealize the distribution. A trapezoidal distribution varying from $0.25f_{max}$ at the web to f_{max} at the column flange tip is proposed (see Figure 13a). This idealized stress distribution has a resultant force located at $0.6b$ from the column web. For an exterior case, the stress is more uniform, varying from $0.40f_{max}$ near the web to f_{max} at the column flange tip (Uang et al., 2011). For the proposed design procedure that follows, it is conservative to set the location of the resultant force at $0.6b$ from the column web for both the interior and exterior cases. Figure 13b shows the stress distributions along the web weld for the interior control case. The shear stress is high but relatively uniform along this edge.

REVISED BEAM FLANGE FORCE DEMAND

It was shown in the presentation of Equation 7 that the current seismic codes (AISC 2005c, 2010c) assume all moment in the beam is transferred to the column by the beam flanges only. While this may be more consistent with the pre-Northridge-type connections that feature a bolted beam web and welded beam flanges, post-Northridge SMF connections with a welded beam web have been shown to reduce force demand on the beam flanges. Nonlinear FEA also demonstrated that welded beam webs of RBS and WUF-W

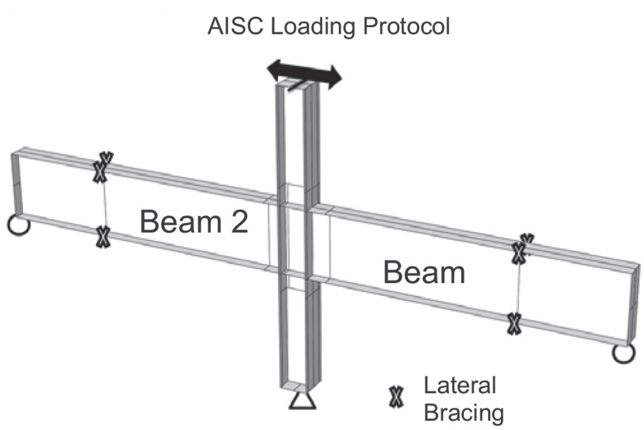


Fig. 8. Two-sided WUF-W model.

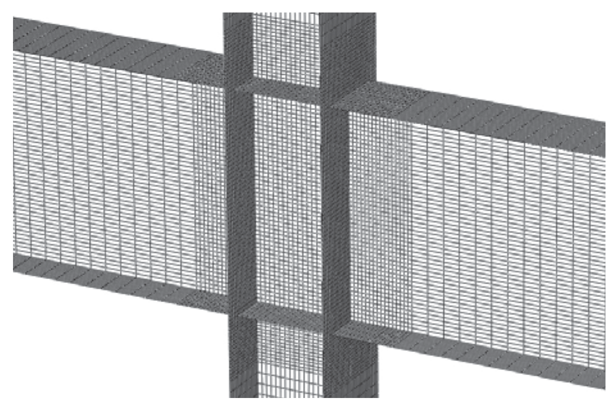
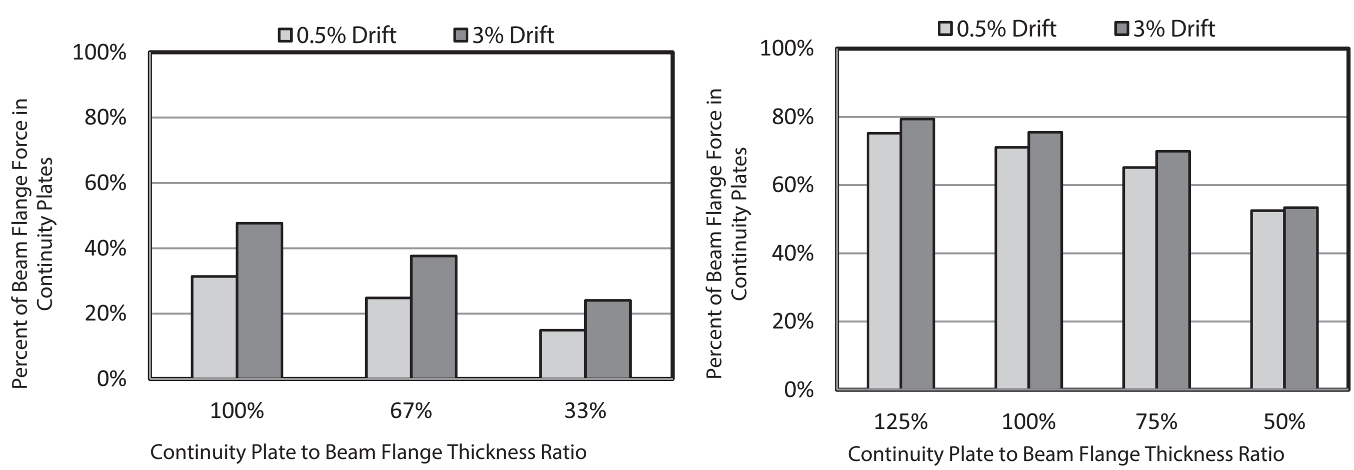


Fig. 9. Finite element mesh in the connection region.



(a) interior connection

(b) exterior connection

Fig. 10. Continuity plate thickness effect on continuity plate normal force demand: (a) interior connection; (b) exterior connection.

connections can transfer a significant portion (up to 15 to 20%) of the beam moment at the column face.

Therefore, in lieu of using a factor of 1.8 in Equation 5 to compute the beam flange force, a reduced value can be used for moment connections with a welded beam web. The following is proposed to replace Equation 5:

$$P_{uf} = C_{pf} R_y F_{yb} b_{bf} t_{bf} \tag{9}$$

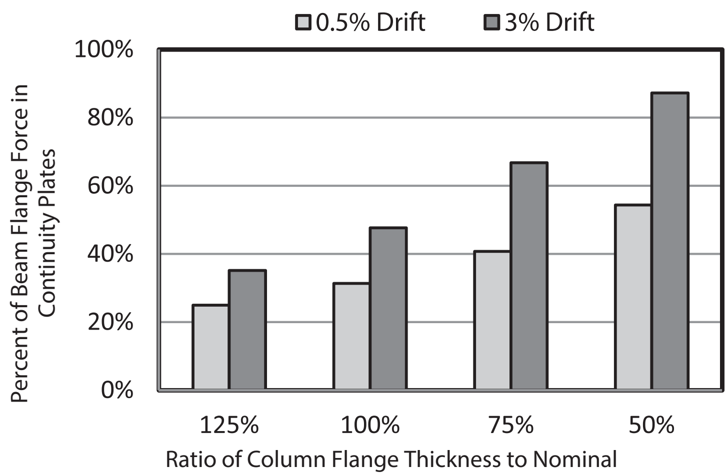
where C_{pf} is the beam flange force adjustment factor. Note that C_{pf} is different from C_{pr} used in AISC 358. The former is used to compute the expected beam flange force, while the latter is for computing the expected plastic hinge moment. A derivation of the C_{pf} factor for both the RBS and WUF-W connections is presented later.

RBS Moment Connection

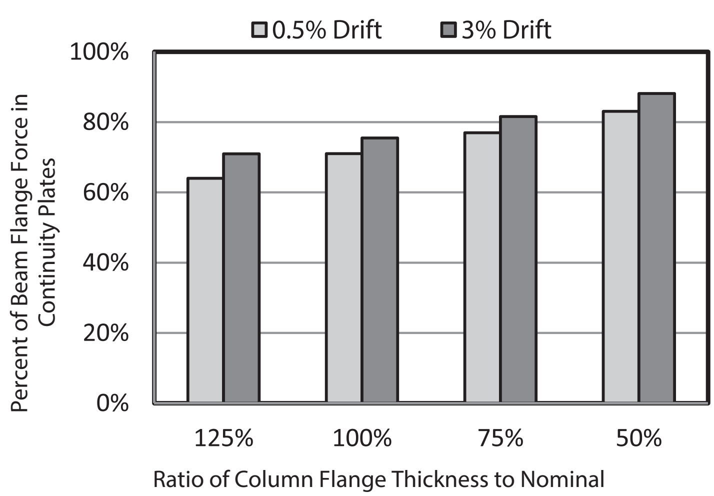
Based on AISC 358, the beam moment at the column face is limited to

$$M_f = R_y F_{yb} Z_x \tag{10}$$

where Z_x is the plastic section modulus of the beam. Based



(a) interior connection



(b) exterior connection

Fig. 11. Column flange thickness effect on continuity plate normal force demand: (a) interior connection; (b) exterior connection.

on previously conducted FEA, it is conservative to assume that the beam web resists only 15% of M_f (i.e., the beam flange resists 85% of M_f). Thus, the beam flange force is

$$P_{uf} = \frac{0.85M_f}{d_b - t_{bf}} = \frac{0.85R_yF_{yb}Z_x}{d_b - t_{bf}} \tag{11}$$

Multiply both the numerator and denominator of Equation 11 by the beam flange area:

$$\begin{aligned} P_{uf} &= \frac{0.85R_yF_{yb}Z_x\left(b_{bf}t_{bf}\right)}{\left(d_b - t_{bf}\right)\left(b_{bf}t_{bf}\right)} \\ &= 0.85\left(\frac{Z_x}{Z_f}\right)R_yF_{yb}\left(b_{bf}t_{bf}\right) \end{aligned} \tag{12}$$

where Z_f [$= (d_b - t_{bf})b_{bf}t_{bf}$] is the plastic section modulus of the beam flanges.

AISC 358-10 prequalifies the RBS moment connection with the following limitations for the beam size and weight:

1. Beam depth is limited to W36 for rolled shapes.
2. Beam weight is limited to 300 lb/ft.
3. Beam flange thickness is limited to 1¾ in.

Figure 14a shows the values of the Z_x/Z_f ratio for all seismically compact rolled shapes of W12 or deeper that satisfy the preceding beam size limitations. If an upper-bound value of Z_x/Z_f taken as 1.47, only 10 out of the 127 shapes in the figure exceed this value; 6 shapes exceed the upper-bound value of 1.47 by less than 3%, and 4 shapes (W21×44, W21×50, W24×55 and W24×62) exceed this upper-bound value by a range of 5 to 11%. With this upper-bound value, Equation 12 can be taken as follows:

$$\begin{aligned} P_{uf} &= 0.85\left(1.47\right)R_yF_{yb}\left(b_{bf}t_{bf}\right) \\ &= 1.25R_yF_{yb}\left(b_{bf}t_{bf}\right) \end{aligned} \tag{13}$$

This represents a 30% reduction in beam flange force as compared to that implicitly assumed in Equation 7. That is, using Equation 7 as a criterion to determine the need for continuity plates is very conservative because it does not recognize the significant reduction of beam flange force by introducing the reduced section in the beam.

WUF-W Moment Connection

Based on AISC 358, the beam moment at the column face is

$$M_f = 1.4R_yF_{yb}Z_x \tag{14}$$

Assuming that the beam flanges resist 85% of M_f, the beam flange force is

$$\begin{aligned} P_{uf} &= \frac{0.85\left(1.4R_yF_{yb}Z_x\right)\left(b_{bf}t_{bf}\right)}{\left(d_b - t_{bf}\right)\left(b_{bf}t_{bf}\right)} \\ &= 1.19\left(\frac{Z_x}{Z_f}\right)R_yF_{yb}\left(b_{bf}t_{bf}\right) \end{aligned} \tag{15}$$

AISC 358-10 prequalifies the WUF-W moment connection with the following limitations for the beam size and weight:

1. Beam depth is limited to W36 for rolled shapes.
2. Beam weight is limited to 150 lb/ft.
3. Beam flange thickness is limited to 1 in.

Figure 14b shows the values of the Z_x/Z_f ratio for all

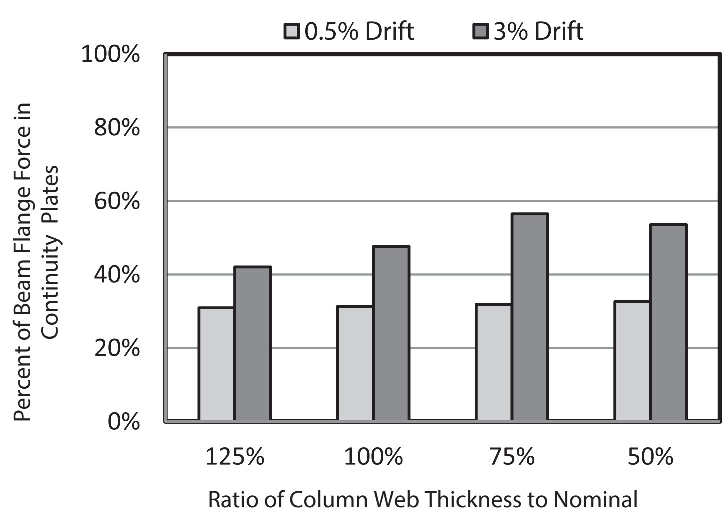
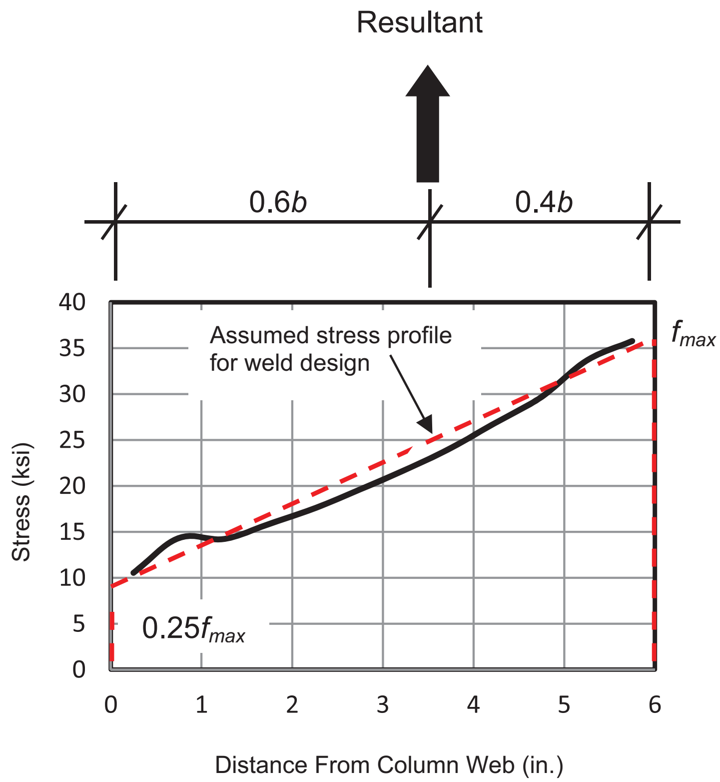
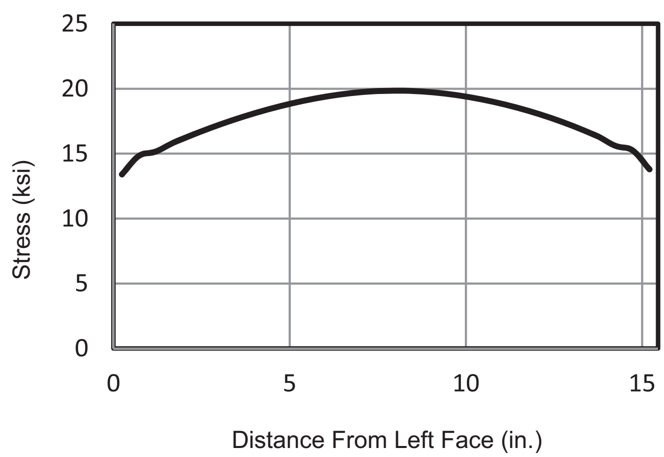


Fig. 12. Column web thickness effect on continuity plate normal force demand (interior connection).



(a)



(b)

Fig. 13. Stress distribution along continuity plate welds at 3% drift (interior base model): (a) normal stress along flange weld; (b) shear stress along web weld.

seismically compact rolled shapes of W12 or deeper that satisfy the preceding beam size limitations. Again, with 1.47 as the upper-bound value for Z_x/Z_f, Equation 15 can be conservatively taken as follows:

$$\begin{aligned} P_{uf} &= 1.19(1.47) R_y F_{yb} \left(b_{bf} t_{bf}\right) \\ &= 1.75 R_y F_{yb} \left(b_{bf} t_{bf}\right) \end{aligned} \tag{16}$$

Because the beam section is not reduced, the preceding beam flange force is about 40% higher than that in Equation 13. This beam flange force is also similar to that implicitly assumed in Equation 7.

To summarize, the beam flange force can be expressed as Equation 9, where the beam flange force adjustment factor, C_{pf}, is

For RBS connection: $C_{pf} = 1.25$ (17)

For WUF-W connection: $C_{pf} = 1.75$ (18)

RELATIVE FLEXIBILITY OF COLUMN FLANGES AND CONTINUITY PLATES

Results from the parametric studies indicate that the seismic force demands on continuity plates depend not only on the beam flange force, but also on the relative flexibility (or stiffness) between the continuity plate and column flange. This section describes the formulation of an analysis procedure for computing the amount of beam flange force allocated to continuity plates by considering the relative flexibility between the column flange and the continuity plate. The flexibility coefficients for both components are established from analytical studies, including FEA of individual components.

Figure 15 depicts the force flow from the beam flange to the column web for an exterior connection with continuity plates. A portion of the beam flange force in line with the column web is transferred directly into the column web. The remaining force is distributed between the continuity plates and column flange based on their relative flexibility. Force allocated to the column flange is transferred to the column web mainly through out-of-plane bending of the column flange, while force allocated to the continuity plates is transferred to the column web mainly through shear. Equation 19 computes the force allocated to one continuity plate, P_{cp}, from the beam flange force, P_{uf},

$$P_{cp} = \frac{P_{uf}}{2} \left(\frac{b_{bf} - t_{pz} - 2t_{cf}}{b_{bf}} \right) \left(\frac{B_{cf}}{B_{cf} + B_{cp}} \right) \tag{19}$$

where b_{bf} is the beam flange width, t_{pz} is the panel zone thickness and t_{cf} is the column flange thicknesses.

The term in the first parentheses accounts for the portion of the beam flange force, which is in line and transferred directly through to the column panel zone; it includes a 45-degree projection through the column flange thickness. The term in the second parentheses accounts for the flexibility of the continuity plate (B_{cp}) relative to the total flexibility ($B_{cf} + B_{cp}$) of the column flange and continuity plate, where B_{cf} is the flexibility of the column flange. The ½ term represents one of the two continuity plates at each beam flange level. The formulation of the column flange and continuity plate flexibility coefficients follows.

Flexibility Coefficient of Column Flange

When continuity plates are used, it is reasonable to assume the beam flange applies a uniform line load across its width and causes the column flange to deform out-of-plane (see Figure 16). Each column flange can be treated as a long, cantilever plate with a support along the column web. Considering symmetry, only half of the width of the column flange

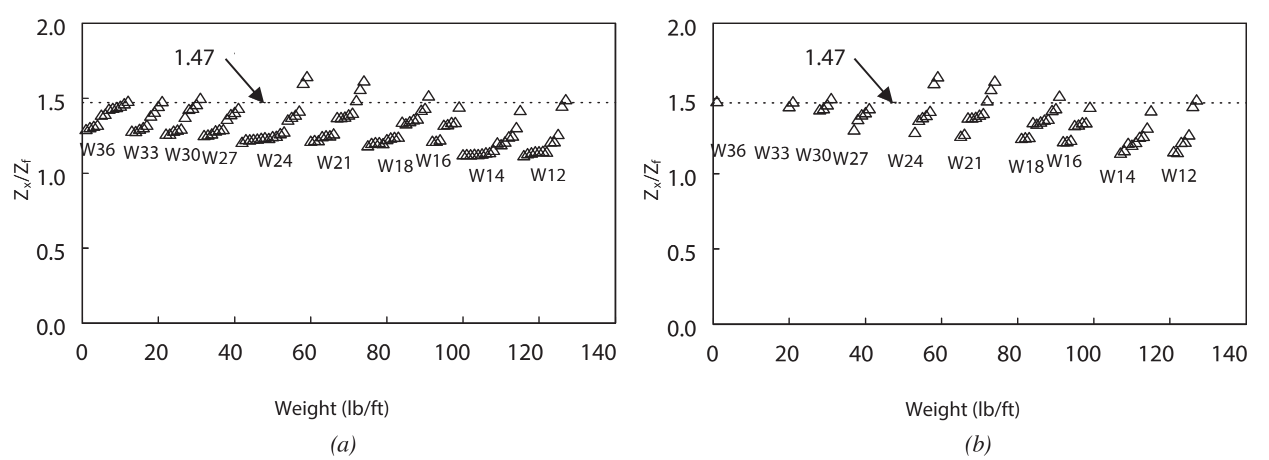


Fig. 14. Z_x/Z_f ratios for seismically compact sections: (a) RBS beam sections; (b) WUF-W beam sections.

needs to be considered in the analysis with a fixed support. The line load acts on the cantilever plate transversely with a loaded length of b, as defined in Figure 16. The flexibility of the column flange is defined as the out-of-plane displacement at the mid-width of the dimension b produced by a total line load of unity. Using the analogy of two springs in series, the column flange flexibility coefficient is

$$B_{cf} = f_{cf,b} + f_{cf,s} \tag{20}$$

where $f_{cf,b}$ and $f_{cf,s}$ are the flexibility coefficients due to bending and shear deformations, respectively.

(1) Flexibility Coefficient Due to Bending

Based on elastic plate theory (Timoshenko and Woinowsky-Krieger, 1959), the flexibility coefficient due to flexure can be expressed in the following form:

$$f_{cf,b} = C_1 \frac{b^2}{Et_{cf}^3} \tag{21}$$

To establish the constant C_1, the flexibility of long plates with varying thickness (t_{cf}) and width (b) was analyzed using FEA. Figure 17a shows the correlation of Equation 21 with $C_1 = 0.26$. Equation 21 correlates well for slender plates with a larger b/t_{cf} ratio, but for stockier plates, the effect of shear becomes significant and must be accounted for.

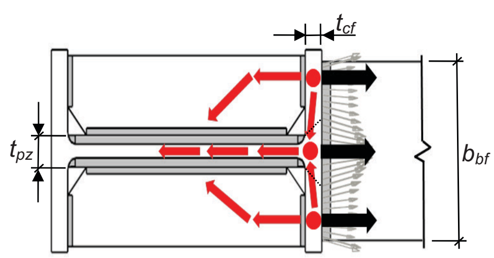


Fig. 15. Flow of beam flange force to column.

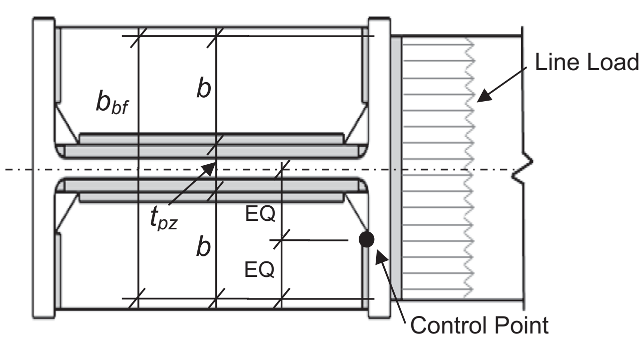


Fig. 16. Definition of column flange flexibility.

(2) Flexibility Coefficient Due to Shear

The flexibility coefficient due to shear can be expressed as

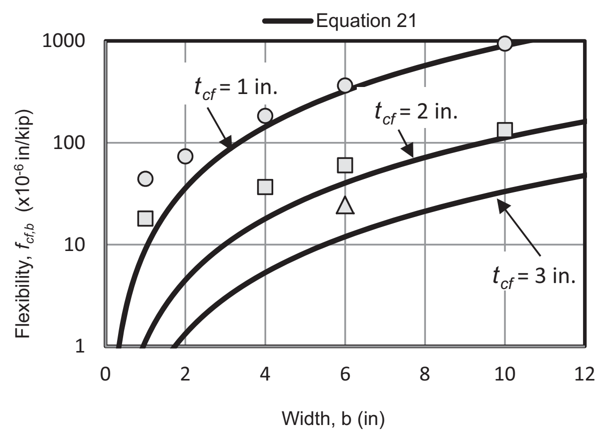
$$f_{cf,s} = \frac{C_2}{Gt_{cf}} \tag{22}$$

where C_2 is a constant.

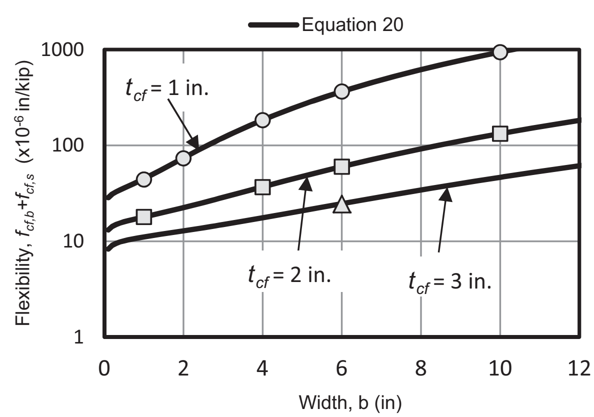
Curve fitting shows that the following expression for C_2 produces good correlation with FEA results:

$$C_2 = 0.4\left[1 + 0.09\ln\left(\frac{b}{t_{cf}}\right)\right] \tag{23}$$

Figure 17b shows the correlation of Equation 20 with FEA results. Equation 20, which accounts for both bending and



(a)



(b)

Fig. 17. Correlation of column flange flexibility: (a) flexural component only; (b) flexural and shear components.

shear deformations, provides good correlation for both stocky and slender column flanges.

Flexibility Coefficient of Continuity Plate

Finite element analysis shows that the normal force distribution varies almost linearly along the flange weld of a continuity plate (see Figure 13a). A simplification is made by assuming that the normal force is uniform, and the flexibility, B_{cp}, is defined as the deflection at the mid-width of the continuity plate due to a total edge load of unity. A continuity plate under the assumed edge load can be treated as a deep beam cantilevered from the column web (see Figure 18). For the purpose of computing the flexibility coefficient, the width of the continuity plate is taken to be equal to the length of the line load, b, as defined in Figure 16.

The applied unit load produces both shear and flexural deformations (shear being the dominant component), which are the shear ($f_{cp,s}$) and flexural ($f_{cp,b}$) flexibilities, respectively. Figure 19a shows the combined deformation from shear and flexure. Note that because the continuity plate has been idealized as a cantilever plate, the edge opposite the load also deforms by an amount, $f_{cp,r}$. In reality, the continuity plate is bounded by both column flanges. FEA shows that the rigidity of the nonloaded column flange restrains the opposite edge from deforming (see Figure 19c). A deformation pattern accounting for the restraint from the free flange is defined as having a magnitude $f_{cp,r}$ in the opposite direction, as shown in Figure 19b.

The superposition of the deformed shapes shown in Figures 19a and 19b results in a deformation pattern shown in Figure 19c. Therefore, the total flexibility coefficient of one continuity plate is:

$$B_{cp} = f_{cp,s} + f_{cp,b} - f_{cp,r} \tag{24}$$

(1) Flexibility Coefficient Due to Shear

Applying the beam theory to the cantilever plate in Figure 18, the shear flexibility is

$$f_{cp,s} = C_3 \frac{b}{Gdt} \tag{25}$$

However, for very small aspect ratios, the shear force does not transfer to the full depth (d) of the plate, but instead to an effective depth proportional to dimension b. Substitution of d with an effective depth proportional to b results in the following expression:

$$f_{cp,s} = C_4 \frac{1}{Gt} \tag{26}$$

The value of constant C_4 is determined by correlating with the FEA data. With $C_4 = 0.42$, Figure 20a shows that Equation 26 provides a good correlation for aspect ratios (b/d) less than 0.4. Above this value, however, the FEA data diverges from Equation 26 due to flexural deformation, which is considered next.

(2) Flexibility Coefficient Due to Bending

Applying the beam theory, the flexibility due to a unit total load is

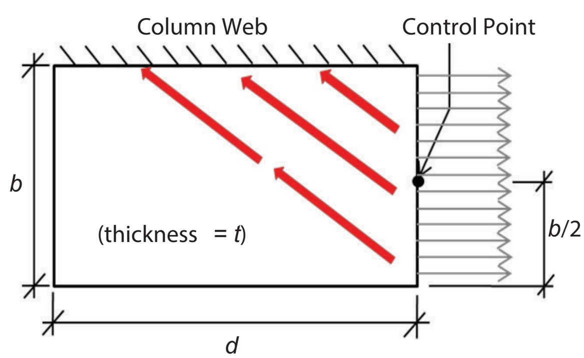


Fig. 18. Definition of continuity plate flexibility.

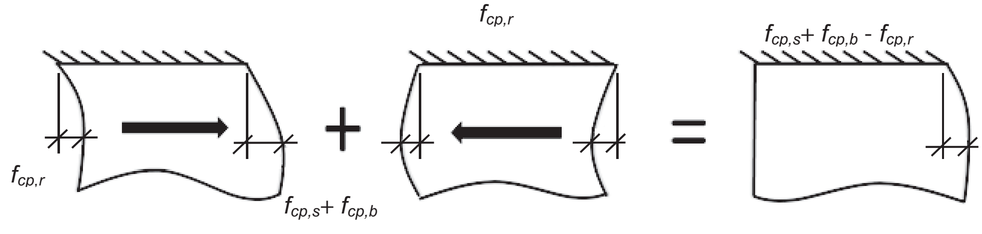


Fig. 19. Superposition of flexibility components for continuity plate: (a) shear and bending flexibility; (b) subtract restraint; (c) total flexibility.

$$f_{cp,b} = C_5 \frac{b^3}{Ed^3t} \quad (27)$$

FEA data is again used to determine the constant C_5. In the curve-fitting process, the bending flexibility is computed as the difference between the total flexibility determined from FEA and the shear flexibility from Equation 26. By including the bending flexibility term with $C_5 = 1.0$, a satisfactory correlation of continuity plate flexibility is achieved with results from FEA over a wider range of aspect ratios, as shown in Figure 20b.

(3) Flexibility Coefficient Due to Restraining Effect

For an exterior connection, resistance by the opposite column flange decreases the flexibility by $f_{cp,r}$ (see Figure 19). The empirical Equations 28a and 28b are correlated with results from FEA:

$$f_{cp,r} = \frac{C}{Gt} \quad (28a)$$

where

$$C = 0.6\left(\frac{b}{d}\right) - 0.14 \geq 0 \quad (28b)$$

Equation 28b implies that the opposite edge does not deform when the aspect ratio is less than 0.23; that is, the effect of shear is negligible for low-aspect ratios.

(4) Total Flexibility

Combining Equations 26, 27, and 28a, the total flexibility of the continuity plate is

$$B_{cp} = f_{cp,s} + f_{cp,b} - f_{cp,r} = \frac{0.42 - C}{Gt} + \frac{b^3}{Ed^3t} \quad (29)$$

where $C = 0$ for interior connections, and C is defined in Equation 28b for exterior connections.

PROPOSED DESIGN PROCEDURE

The proposed design procedure incorporates requirements of AISC 358 with some modifications to determine the need for continuity plates. If required, an iterative process is used to ensure that the design strength of the continuity plates is sufficient to transfer load from the beam flange to column web; the force apportioned to the continuity plates is determined based on the relative flexibility of the column flange and continuity plate. Welds connecting the continuity plates to the column are also sized according to the expected force calculated from this flexibility-based procedure. The proposed design procedure is suitable for moment connections where the beam web and flange are fully welded to the column flange. These include RBS and WUF-W moment connections.

Step 1. Continuity plates need not be provided if

$$t_{cf} \geq 0.4\sqrt{\frac{C_{pf} b_{bf} t_{bf} R_{yb} F_{yb}}{R_{yc} F_{yc}}} \quad (30)$$

$$t_{cf} \geq \frac{b_{bf}}{6} \quad (31)$$

where the beam flange force adjustment factor, C_{pf}, is

For RBS connection: $C_{pf} = 1.25$ (32)

For WUF-W connection: $C_{pf} = 1.75$ (33)

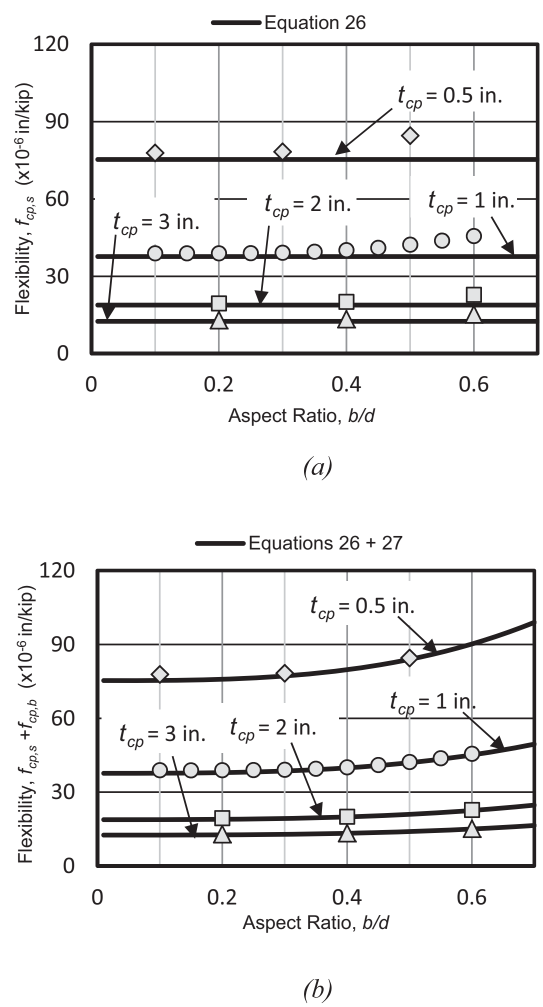


Fig. 20. Correlation of continuity plate flexibility: (a) shear component only; (b) flexural and shear components.

Note: Equation 30 is derived based on the following beam flange force:

$$P_{uf} = C_{pf} R_{yb} b_{bf} t_{bf} F_{yb} \tag{34}$$

It was previously demonstrated that the upper-bound C_{pf} values can be conservatively used for all seismically compact rolled shapes satisfying the AISC 358 size and weight limitations, except for four sections (W21×44, W21×50, W24×55 and W24×62).

Step 2. If continuity plates are needed, perform preliminary sizing based on the following:

$$R_{u(st)} = P_{uf} - \phi R_n \tag{35}$$

where ϕR_n is the design strength of the governing limit state (e.g., FLB or WLY). The required continuity plate cross-sectional area is:

$$A_{cp(req'd)} = \frac{R_{u(st)}}{F_{ycp}} \tag{36}$$

where and F_{ycp} is the specified minimum yield stress of the continuity plate.

The width-thickness ratio also needs to satisfy the following requirement:

$$\frac{b_{cp}}{t} \le 0.75\sqrt{\frac{E}{F_{ycp}}} \tag{37}$$

where b_{cp} and t are the actual (not effective) continuity plate width and thickness, respectively.

Note: AISC 358 requires the continuity plate thickness to be at least equal to one-half of the beam flange thickness for exterior connections and full beam flange thickness for interior connections. In this design procedure, it is suggested that the continuity plate thickness be at least equal to one-half of the beam flange thickness for both exterior and interior connections.

The limiting width-thickness ratio in Equation 37 is the same as that in Section B4 of 2005 AISC *Specification* for the stem of a tee in uniform compression because one edge of the continuity plate is free. Use of this limiting ratio for a continuity plate check is judged to be conservative.

The width of the continuity plate should be selected to extend at least to the end of the beam flange. It may be necessary to extend the continuity plate beyond the beam flange width to increase contact area with the column flange and account for the loss of contact area due to clipped corners to clear the *k*-area. Clipping of corners should be detailed in accordance with Section 3.6 of AISC 358. The net contact width used to calculate the net contact area should not extend a distance one column flange thickness beyond the end of the beam flange (see Figure 21). The width of the column flange may also limit the maximum net contact width. Equation 38, which is used to compute the net width of the continuity plate, takes into account cases where either the beam flange or column flange width is the limiting dimension.

$$b_n = \min\begin{cases} \dfrac{b_{bf} - t_{pz}}{2} - b_{clip} + t_{cf} \\ \dfrac{b_{cf} - t_{pz}}{2} - b_{clip} \end{cases} \tag{38}$$

where t_{pz} = thickness of panel zone and b_{clip} = continuity plate clipped corner dimension parallel to column flange.

Equation 38 assumes that doubler plates, if used, extend beyond the continuity plates. In the case where doubler plates are detailed to stop at the continuity plates, set t_{pz} to the width of the column web.

Step 3. Design continuity plates.

1. Calculate the column flange out-of-plane flexibility coefficient, B_{cf}.

$$B_{cf} = 0.26\frac{b^2}{Et_{cf}^3} + \frac{0.4\left[1 + 0.09\ln\left(\dfrac{b}{t_{cf}}\right)\right]}{Gt_{cf}} \tag{39}$$

2. Calculate the continuity plate in-plane flexibility coefficient, B_{cp}.

$$B_{cp} = \frac{0.42 - C}{Gt} + \frac{b^3}{Ed^3t} \tag{40}$$

where $C = 0$ for interior connection, and for exterior connection:

$$C = 0.6\left(\frac{b}{d}\right) - 0.14 \ge 0 \tag{41}$$

In Equations 39, 40 and 41, $b = b_n + b_{clip}$, and d and t are defined as the depth and thickness of the continuity plate, respectively.

3. Apportion the beam flange force to one continuity plate:

$$P_{cp} = \frac{P_{uf}}{2}\left(\frac{b_{bf} - t_{pz} - 2t_{cf}}{b_{bf}}\right)\left(\frac{B_{cf}}{B_{cf} + B_{cp}}\right) \tag{42}$$

4. Check if FLB and WLY limit states are satisfied with the addition of continuity plates:

$$\phi R_n \geq P_{uf} - 2P_{cp} \tag{43}$$

where ϕR_n is the design strength of the governing limit state (e.g., FLB or WLY). Resize the continuity plates if the preceding condition is not satisfied.

Note: The continuity plates are initially sized for the required force $R_{u(st)}$ in Equation 35. When continuity plates are added, the force transferred into a pair of continuity plates is $2P_{cp}$. Equation 43 ensures that FLB and WLY limit states of the column are satisfied with a reduce beam flange force demand due to the alternative load transfer mechanism provided by the continuity plates.

Step 4. Design continuity plate flange welds.

Refer to Figure 22 for the free-body diagrams of the continuity plate for the interior and exterior connection configurations.

1. Calculate the required shear force in the flange weld:

$$V_{cp} = \left(\frac{0.6b}{d} \right) \Sigma P_{cp} \tag{44}$$

Note: Based on an idealized trapezoidal normal stress distribution shown in Figure 13a, the resultant normal force, P_{cp}, is located at a distance $0.6b$ from the column web. To satisfy moment equilibrium, in-plane shear forces are present in the flange welds.

2. Either fillet, partial joint penetration (PJP), or a combination of PJP with reinforcing fillet welds can be used to connect the continuity plates to the column flanges if the following condition is satisfied:

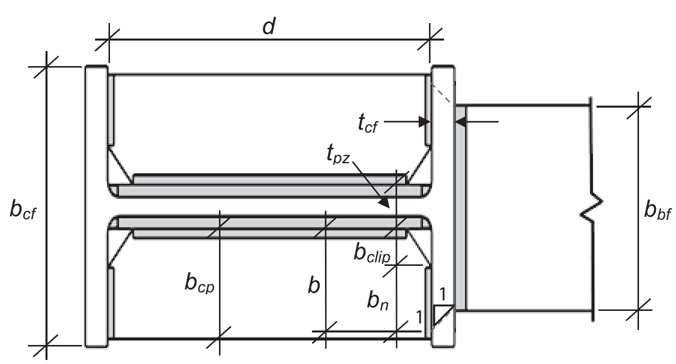


Fig. 21. Net bearing width of continuity plate.

$$\left(\frac{P_{cp}}{F_{ycp} A_{net}} \right)^2 + 3 \left(\frac{V_{cp}}{F_{ycp} A_{net}} \right)^2 < 1 \tag{45}$$

Otherwise, CJP welds are required.

Note: The continuity plates may yield, similar to the portion of the beam flanges that are CJP welded to the column. Unlike the beam flange, however, continuity plate is not subjected to shear through its thickness, which causes additional stress. It is proposed in this design procedure that CJP welds still be used if continuity plates are likely to experience significant yielding similar to the beam flanges. Otherwise, PJP, fillet welds, or a combination thereof can be used. Equation 45 is based on the von Mises yield criterion for plane stress and is used to check the net cross section strength of the continuity plate.

3. When either fillet or PJP welds are used, welds are to be designed to satisfy the following:

   a. Design the flange weld for the required resultant force, R_{cp}:

$$\phi_n R_n \geq R_{cp} \tag{46}$$

where

$$R_{cp} = \sqrt{P_{cp}^2 + V_{cp}^2} \tag{47}$$

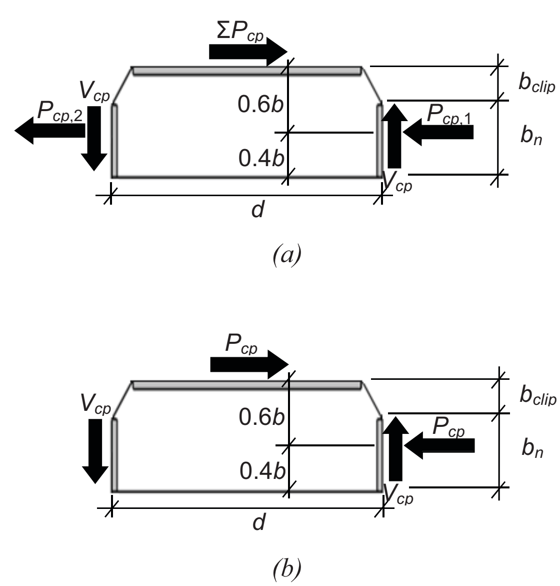


Fig. 22. Free-body diagram of a continuity plate: (a) interior connection; (b) exterior connection.

The design strength for two-sided flange fillet welds is

$$\phi_n R_n = 2(\phi_n)(0.6) t_e b_n F_{EXX} \left(1.0 + 0.5\sin^{1.5}\theta\right) \quad (48)$$

where t_e is the effective throat of one fillet weld, F_{EXX} is the minimum specified ultimate strength of the weld and ϕ_n is 0.9 per AISC 358. The angle of the resultant force, R_{cp}, measured from the weld longitudinal axis is

$$\theta = \tan^{-1} \frac{P_{cp}}{V_{cp}} \quad (49)$$

b. Check the flange weld at the location of maximum tensile stress, q_{max} (kips/in):

$$q_{max} = \frac{1.6 P_{cp}}{b_n} \quad (50)$$

Note: The maximum stress is based on the assumed trapezoidal normal stress distribution in Figure 13a. If two-sided fillet welds are used, the value of q_{max} cannot exceed the unit-length design strength, which can be computed by using Equation 48 and setting $b_n = 1.0$ in.

If fillet welds are used, the design weld strength can be based on $\theta = 90°$.

c. Check maximum shear stress in the flange weld, τ_{max} (kips/in):

$$\tau_{max} = \frac{2V_{cp}}{b_n} \quad (51)$$

Note: Use $\theta = 0°$ to compute the weld design strength.

Step 5. Design continuity plate web weld.

Design the web weld for a required shear force equal to the summation of force allocated to the continuity plate, ΣP_{cp}, as shown in Figure 22. For exterior moment connections (Figure 22b), the required shear force is simply P_{cp}.

$$\phi_n R_n \geq \Sigma P_{cp} \quad (52)$$

If two-sided fillet welds are used, the design strength is computed as

$$\phi_n R_n = 2(\phi)(0.6) t_e l_w F_{EXX} \quad (53)$$

where $\phi_n = 0.9$, t_e = effective throat of one fillet weld and l_w = length of the web weld.

The column panel zone base metal shear strength should also be checked to ensure it has the capacity to develop the force demand allocated to a pair of continuity plates (on each side of the column web).

Figures 23 and 24 show a comparison of two designs for an RBS moment connection with a W14 column. A similar comparison is presented in Figure 24 when a deep column (W33) is used. See Uang et. al (2011) for step-by-step calculations. Compared with the current seismic code requirements, these two design examples show that the proposed flexibility-based design approach often leads to thinner continuity plates and smaller welds. In addition, the option to use fillet, PJP, or PJP with reinforcing fillet flange welds versus CJP flange welds reduces the cost of fabrication and inspection. Also, the significantly reduced beam flange force demand for RBS connections will lead to cases where continuity plates that are required based on the current design code are not needed.

CONCLUSIONS

A historical review of code developments and past full-scale testing programs have suggested that conservatisms exist in the sizing and weld criteria of continuity plates in SMF moment connections. The two main areas of conservatism identified were (1) the beam flange force demand and (2) the force allocation into continuity plates.

Nonlinear FEA suggests that for moment connections with the beam web welded to the column flange—for example, RBS and WUF-W connections—the web transfers a noticeable portion of the moment, thus reducing the beam flange force to the column face. A revised beam flange force demand is introduced (see Equations 32, 33 and 34).

Results of parametric studies demonstrated a reduction of demand force to the continuity plates with increased column flange thickness. Likewise, an increased continuity plate thickness reduced the demand on the column flange. As a result, flexibility coefficients for the column flange and continuity plates were corroborated with FEA to introduce a means by which the beam flange force into the continuity plates can be apportioned.

A design procedure is proposed that provides a rational approach to (1) determine the need for continuity plates, (2) size the thickness of continuity plates and (3) size the flange and web welds to attach the continuity plates. Like other welded moment connection details, however, verification by full-scale testing is needed before the proposed procedure can be adopted for practical design.

ACKNOWLEDGMENTS

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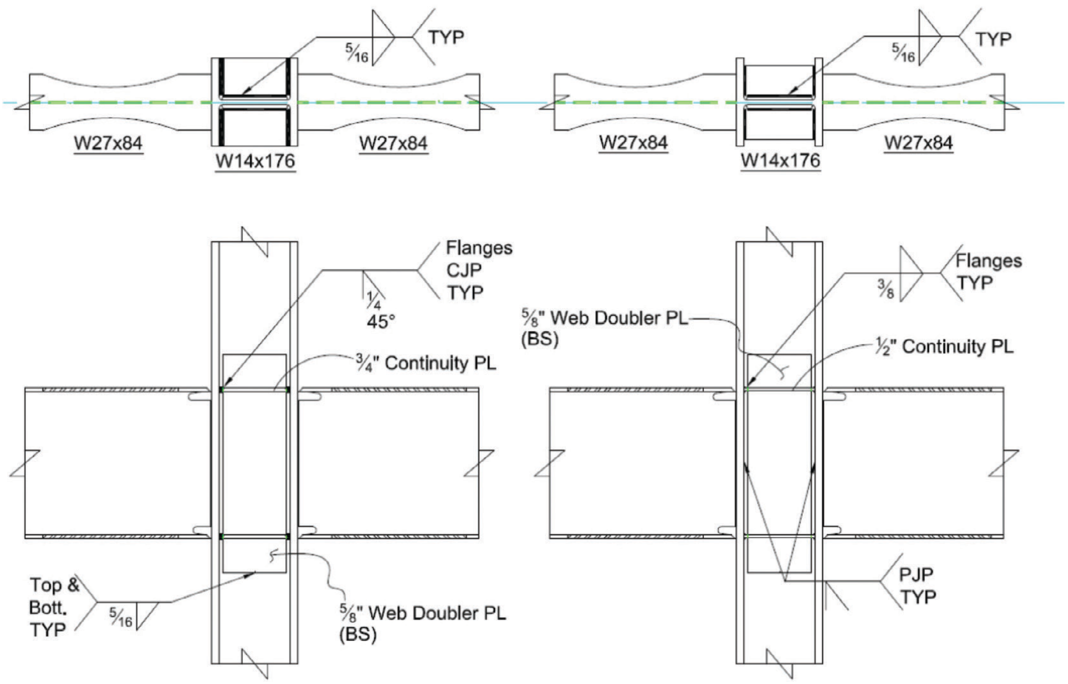
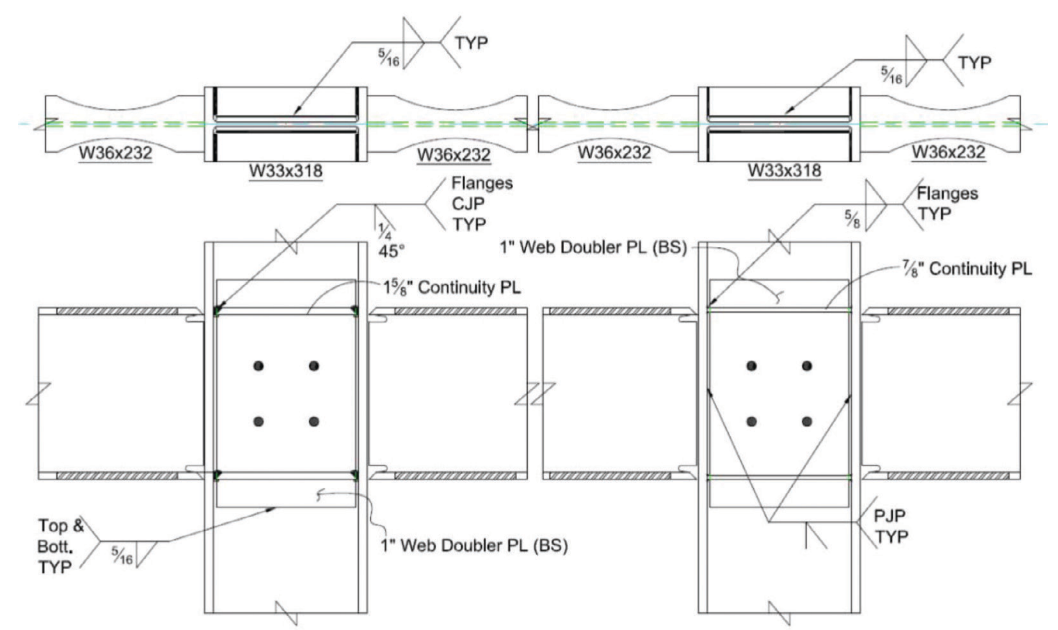


(a) current design practice *(b) proposed design procedure*

(a) *(b)*

Fig. 23. Comparison of continuity plate and weld design of an RBS moment connection with a W14 column: (a) current design practice; (b) proposed design procedure.



(a) current design practice *(b) proposed design procedure*

(a) *(b)*

Fig. 24. Comparison of continuity plate and weld design of an RBS moment connection with a deep column: (a) current design practice; (b) proposed design procedure.

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