Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications
Supplement No. 1

A supplement to ANSI/AISC 358-16

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PREFACE

(This Preface is not part of AISC 358-16s1, Supplement 1 to Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications, but is included for informational purposes only.)

This supplement was developed by the AISC Connection Prequalification Review Panel (CPRP) using a consensus process. This document is the first supplement to ANSI/AISC 358-16, Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications.

This supplement adds a new prequalified moment connection, the proprietary SlottedWeb Moment Connection, in a new Chapter 14. Chapter 11 covering the SidePlate Moment Connection has been expanded to include HSS columns and to permit bolted connections. Additionally, Chapter 10 covering the ConXtech CONXL Moment Connection has been revised to address a manufacturing safety issue.

A non-mandatory Commentary has been prepared to provide background for the provisions of the Standard and the user is encouraged to consult it. Additionally, non-mandatory User Notes are interspersed throughout the Standard to provide concise and practical guidance in the application of the provisions.

The reader is cautioned that professional judgment must be exercised when data or recommendations in this Standard are applied, as described more fully in the disclaimer notice preceding the Preface.

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SYMBOLS

This Standard uses the following symbols in addition to the terms defined in the Specification for Structural Steel Buildings (ANSI/AISC 360-16) and the Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341-16). Some definitions in the following list have been simplified in the interest of brevity. In all cases, the definitions given in the body of the Standard govern. Symbols without text definitions, used in only one location and defined at that location, are omitted in some cases. The section or table number on the right refers to where the symbol is first used.

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<th>Symbol</th>
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<td>( A_{\perp} )</td>
<td>Perpendicular amplified seismic drag or chord forces transferred through the SidePlate connection, resulting from applicable building code, kips (N)</td>
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<td>( A_{</td>
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<td>( H_{h} )</td>
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<td>( I_{total} )</td>
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<td>( M_{cant} )</td>
<td>Factored gravity moments from cantilever beams that are not in the plane of the moment frame but are connected to the exterior face of the side plates, resulting from code-applicable load combinations, kip-in. (N-mm)</td>
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<td>( M_f )</td>
<td>Probable maximum moment at face of the column, kip-in. (N-mm)</td>
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<td>( M_{group} )</td>
<td>Maximum probable moment demand at any connection element, kip-in. (N-mm)</td>
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<td>( M_{ay} )</td>
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<td>Moment resisted by the shear plate, kip-in. (N-mm)</td>
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<td>( R_y )</td>
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<td>( T )</td>
<td>Beam web height as given in the AISC Manual, in. (mm)</td>
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<td>( V_{beam} )</td>
<td>Shear at beam plastic hinge, kips (N)</td>
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$V_{cant}$  Factored gravity shear forces from cantilever beams that are not in the plane of the moment frame but are connected to the exterior face of the side plates, resulting from code-applicable load combinations, kips (N) ................................................................. 11.7

$V_{gravity}$  Beam shear force resulting from the load combination $1.2D + f_1L + 0.2S$, kips (N) .......... 14.8

$V_{weld}$  Shear resisted by the shear plate, kips (N) ................................................................. 14.8

$V_1, V_2$  Factored gravity shear forces from gravity beams that are not in the plane of the moment frame but are connected to the exterior surfaces of the side plate, resulting from the load combination of $1.2D + f_1L + 0.2S$ (where $f_1$ is the load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N) .................................................. 11.7

$Z_{beam}$  Plastic section modulus of the beam, in.³ (mm³) .................................................... 14.8

$Z_{cxc}$  Equivalent plastic section modulus of the column at a distance of $\frac{1}{2}$ the column depth from the top and bottom edge of the side plates, projected to the beam centerline, in.³ (mm³) .................................................... 11.4

$Z_{web}$  Plastic section modulus of the beam web, in.³ (mm³) .................................................. 14.8

$Z_{xb}$  Plastic modulus of beam about the $x$-axis, in.³ (mm³) .................................................. 11.7

$Z_{xc}$  Plastic modulus of column about the $x$-axis, in.³ (mm³) .................................................. 11.7

$b_f$  Flange width, in. (mm) ..................................................................................................... 14.8

$d$  Nominal beam depth, in. (mm) .......................................................................................... 14.8

$d_{col}$  Depth of the column, in. (mm) ..................................................................................... 14.4

$d_{c1}, d_{c2}$  Depth of column on each side of a bay in a moment frame, in. (mm) ....................... 11.3

$e_x$  Eccentricity of the shear plate weld, in. (mm) ................................................................. 14.8

$h$  Height of shear plate, in. (mm) .......................................................................................... 14.8

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$l_p$  Width of shear plate, in. (mm) .......................................................................................... 14.4

$l_s$  Beam slot length, in. (mm) ................................................................................................. 14.8

$t_{bf}$  Thickness of beam flange, in. (mm) .................................................................................. 14.8

$t_p$  Minimum required shear plate thickness, in. (mm) .......................................................... 14.8

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CHAPTER 11

SIDEPLATE MOMENT CONNECTION

The user's attention is called to the fact that compliance with this chapter of the standard requires use of an invention covered by multiple U.S. and foreign patent rights. By publication of this standard, no position is taken with respect to the validity of any claim(s) or of any patent rights in connection therewith. The patent holder has filed a statement of willingness to grant a license under these rights on reasonable and nondiscriminatory terms and conditions to applicants desiring to obtain such a license, and the statement may be obtained from the standard's developer.

11.1. GENERAL

The SidePlate® moment connection utilizes interconnecting plates to connect beams to columns. The connection features a physical separation, or gap, between the face of the column flange and the end of the beam. Both field-welded and field-bolted options are available. Beams may be either rolled or built-up wide-flange sections or hollow structural sections (HSS). Columns may be either rolled or built-up wide-flange sections, built-up box column sections or HSS for uniaxial configurations. Built-up flanged cruciform sections consisting of rolled shapes or built-up from plates may also be used as the columns for biaxial configurations. Figures 11.1, 11.2 and 11.3 show the various field-welded and field-bolted uniaxial connection configurations. The field bolted option is available in two configurations, referred to as Configuration A (standard) and Configuration B (narrow), as shown in Figure 11.3.

In the field-welded connection, top and bottom beam flange cover plates (rectangular or U-shaped) are used at the end(s) of the beam, as applicable, which also serve to bridge any difference between flange widths of the beam(s) and of the column. The connection of the beam to the column is accomplished with parallel full-depth side plates that sandwich and connect the beam(s) and the column together. In the field-bolted connection, beam flanges are connected to the side plates with either a cover plate or pair of angles and high strength pretensioned bolts as shown in Figures 11.2 and 11.3. Column horizontal shear plates and beam vertical shear elements (or shear plates as applicable) are attached to the wide-flange shape column and beam webs, respectively.

\[\text{Diagram of SidePlate® moment connection} \]

*The SidePlate® connection configurations and structures illustrated herein, including their described fabrication and erection methodologies, are protected by one or more of the following U.S. and foreign patents: U.S. Pat. Nos. 5,660,017; 6,138,427; 6,516,583; 6,591,573; 7,178,296; 8,122,671; 8,122,672; 8,146,322; 8,176,706; 8,205,408; Mexico Pat. No. 238,750; New Zealand Pat. No. 300,351; British Pat. No. 2497635; all held by MiTek Holdings LLC. Other U.S. and foreign patent protection are pending.*
Fig. 11.1. Assembled SidePlate uniaxial field-welded configurations: (a) one-sided wide-flange beam and column construction; (b) two-sided wide-flange beam and column construction; (c) wide-flange beam to either HSS or built-up box column; (d) HSS beam without cover plates to wide-flange column; (e) HSS beam with cover plates to wide-flange column; and (f) HSS beam with cover plates to either HSS or built-up box column.

Fig. 11.2. Assembled SidePlate uniaxial field-bolted standard configurations (Configuration A): (a) one-sided wide-flange beam and column construction; (b) two-sided wide-flange beam and column construction; (c) wide-flange beam to either HSS or built-up box column; (d) HSS beam to wide-flange column; (e) HSS beam with cover plate to wide-flange column; and (f) HSS beam with cover plates to either HSS or built-up box column.
Fig. 11.3. SidePlate field-welded and field bolted connection comparison: (a) typical field-welded connection; (b) typical field-bolted standard connection (Configuration A); (c) typical field-bolted narrow connection (Configuration B).

Figure 11.4 shows the connection geometry and major connection components for uniaxial field-welded configurations. Figure 11.5 shows the connection geometry and major connection components for biaxial field-welded configurations, which permits connecting up to four beams to a column. Field bolted connections are also permitted in biaxial configurations.
Fig. 11.4. SidePlate uniaxial configuration geometry and major components: (a) typical wide-flange beam to wide-flange column, detail, plan and elevation views; (b) HSS beam without cover plates to wide-flange column, plan view; (c) HSS beam with cover plates to wide-flange column, plan view; and (d) wide-flange beam to built-up box column, plan view.
Fig. 11.5. SidePlate biaxial dual-strong axis configurations in plan view: (a) full four-sided wide-flange column configuration; (b) corner two-sided wide-flange column configuration with single WT; (c) tee three-sided wide-flange column configuration with double WT (primary); and (d) tee three-sided wide-flange column configuration with single WT.

The SidePlate moment connection is proportioned to develop the probable maximum moment capacity of the connected beam. Plastic hinge formation is intended to occur primarily in the beam beyond the end of the side plates away from the column face, with limited yielding occurring in some of the connection elements.

**User Note:** Moment frames that utilize the SidePlate connection can be constructed using one of three methods. These are the full-length beam erection method (SidePlate FRAME configuration), the link-beam erection method (SidePlate Original configuration), and the fully shop prefabricated method. These methods are described in the commentary.

**11.2. SYSTEMS**

The SidePlate moment connection is prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limits of these provisions. The SidePlate moment connections are prequalified for use in planar moment-resisting frames and orthogonal intersecting moment-resisting frames (biaxial configurations, capable of connecting up to four beams at a column), as illustrated in Figure 11.5.
11.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

(1) Beams shall be rolled wide-flange, hollow structural section (HSS), or built-up I-shaped beams conforming to the requirements of Section 2.3. Beam flange thickness shall be limited to a maximum of 2.5 in. (63 mm).

(2) Rolled wide flange beam depths shall be limited to W40 (W1000) and W44 (W1100) for the field-welded and field-bolted connections, respectively. The depth of built-up wide flange beams shall not exceed the depth permitted for rolled wide flange beams.

(3) Beam depths shall be limited as follows for HSS shapes:

(a) For SMF systems, HSS14 (HSS 356) or smaller.

(b) For IMF systems, HSS16 (HSS 406) or smaller.

(4) Rolled and built-up wide-flange beam weight shall be limited to 302 lb/ft (449 kg/m) and 400 lb/ft (595 kg/m) for the field-welded and field-bolted connections, respectively. Beam flange area of the field-bolted connection shall be limited to a maximum of 36 in² (22900 mm²)

(5) The ratio of the hinge-to-hinge span of the beam, \( L_h \), to beam depth, \( d \), shall be limited as follows:

(a) For SMF systems, \( L_h/d \) is limited to:

- 6 or greater with rectangular shaped cover plates.
- 4.5 or greater with U-shaped cover plates for field-welded connections.
- 4.0 or greater with U-shaped cover plates for field-bolted connections.

(b) For IMF systems, \( L_h/d \) is limited to 3 or greater.

The hinge-to-hinge span of the beam, \( L_h \), is the distance between the locations of plastic hinge formation at each moment-connected end of that beam. The location of plastic hinge shall be taken as one-third of the beam depth, \( d/3 \) for the field-welded connection and one-sixth of the beam depth, \( d/6 \), for the field-bolted connection, away from the end of the side-plate extension, as shown in Figure 11.6. Thus,

\[
L_h = L - \frac{1}{3}(d_{c1} + d_{c2}) - 2(0.33)d - 2A \quad \text{(field-welded)} \tag{11.3-1a}
\]

\[
L_h = L - \frac{1}{6}(d_{c1} + d_{c2}) - 2(0.165)d - 2A \quad \text{(field-bolted)} \tag{11.3-1b}
\]

where

\[
L = \text{distance between column centerlines, in. (mm)}
\]

\[
d_{c1}, d_{c2} = \text{depth of column on each side of a bay in a moment frame, in. (mm)}
\]

User Note: The 0.33d and 0.165d constants represent the distance of the plastic hinge from the end of the side plate extension. A represents the typical extension of the side plates from the face of column flange.
(6) Width-to-thickness ratios for beam flanges and webs shall conform to the limits of the AISC Seismic Provisions.

(7) Lateral bracing of wide-flange beams shall be provided in conformance with the AISC Seismic Provisions. Lateral bracing of HSS beams shall be provided in conformance with Appendix 1, Section 1.3.2c of the AISC Specification, taking $M_1/M_2 = -1$ in AISC Specification Equation A-1-7. For either wide-flange or HSS beams, the segment of the beam connected to the side plates shall be considered to be braced. Supplemental top and bottom beam flange bracing at the expected hinge is not required.

(8) The protected zone in the beam for the field-welded and field-bolted connections shall consist of the portion of the beam as shown in Figure 11.7 and Figure 11.8, respectively.

2. **Column Limitations**

Columns shall satisfy the following limitations:

(1) Columns shall be any of the rolled shapes, hollow structural section (HSS), built-up I-shaped sections, flanged cruciform sections consisting of rolled shapes or built-up from plates or built-up box sections meeting the requirements of Section 2.3. Flange and web plates of built up box columns shall continuously be connected by fillet welds or PJP groove welds along the length of the column.

(2) HSS column shapes must conform to ASTM A1085.

(3) The beam shall be connected to the side plates that are connected to the flange tips of the wide-flange or corners of HSS or box columns.

(4) Rolled shape column depth shall be limited to W44 (W1100). The depth of built-up wide-flange columns shall not exceed that for rolled shapes. Flanged cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes. Built-up box columns shall not have a width exceeding 33 in. (840 mm).

(5) There is no limit on column weight per foot.
There are no additional requirements for column flange thickness.

Width-to-thickness ratios for the flanges and webs of columns shall conform to the requirements of the AISC Seismic Provisions.

Lateral bracing of columns shall conform to the requirements of the AISC Seismic Provisions.

Fig. 11.7. Location of beam and side plate protected zones for the field-welded connection: (a) one-sided connection; (b) two-sided connection
3. Connection Limitations

The connection shall satisfy the following limitations:

(1) All connection steel plates, which consist of side plates, cover plates, horizontal shear plates, and vertical shear elements, must be fabricated from structural steel that complies with ASTM A572/A572M Grade 50 (Grade 345).

Exception: The vertical shear element as defined in Section 11.6 may be fabricated using ASTM A36/A36M material.

(2) The extension of the side plates beyond the face of the column shall be within the range of $0.65d$ to $1.0d$ and $0.65d$ to $1.7d$, for the field-welded and field-bolted connections, respectively, where $d$ is the nominal depth of the beam.

(3) The protected zone of the connection in the side plates shall consist of a portion of each side plate that is 6-in. (150 mm) high and starts at the inside face of the flange of a wide-flange or HSS column and ends either at the end of the gap (field-welded connection) or the edge of the first bolt hole (field bolted connection) as shown in Figures 11.7 and 11.8.
11.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

Beam-to-column connections shall satisfy the following limitations:

(1) Beam flange width and thickness for rolled, built-up and HSS shapes shall satisfy the following equations for geometric compatibility (see Figure 11.9):

(a) Field-welded Connection

\[ b_{bf} + 1.1 t_{bf} + \frac{1}{2} \text{ in.} \leq b_{cf} \]  
\[ b_{bf} + 1.1 t_{bf} + 12 \text{ mm} \leq b_{cf} \]  

(b) Field-bolted Connection

\[ b_{bf} + 1.0 \text{ in.} \leq b_{cf} \]  
\[ b_{bf} + 25 \text{ mm} \leq b_{cf} \]

where

- \( b_{bf} \) = width of beam flange, in. (mm)
- \( b_{cf} \) = width of column flange, in. (mm)
- \( t_{bf} \) = thickness of beam flange, in. (mm)

(2) Panel zones shall conform to the applicable requirements of the AISC Seismic Provisions.

User Note: The column web panel zone strength shall be determined using AISC Specification Section J10.6b.

(3) Column-beam moment ratios shall be limited as follows:

*Fig. 11.9. Geometric compatibility (a) field-welded connection; (b) field-bolted standard connection (Configuration A), (c) field-bolted narrow connection (Configuration B)*
(a) For SMF systems, the column-beam moment ratio shall conform to the requirements of the AISC Seismic Provisions as follows:

(i) The value of \( \sum M'_{pb} \) shall be the sum of the projections of the expected flexural strengths of the beam(s) at the plastic hinge locations to the column centerline (Figure 11.10). The expected flexural strength of the beam shall be computed as:

\[
\sum M'_{pb} = \sum \left( 1.1 R_F y_b Z_b + M_b \right)
\]

where

\[ F_{yb} = \text{specified minimum yield stress of beam, ksi (MPa)} \]
\[ M_v = \text{additional moment due to shear amplification from the center of the plastic hinge to the centerline of the column.} \]
\[ M_e = \text{computed as the quantity } V_{hsh}; \]
\[ V_h = \frac{2M_{pc}}{L_h} + V_{gravity} \]

where

\[ L_h = \text{distance between plastic hinge locations, in. (mm)} \]
\[ M_{pc} = \text{probable maximum moment at plastic hinge, kip-in. (N-mm)} \]
\[ V_{gravity} = \text{beam shear force resulting from 1.2D + f_1L + 0.2S}\]
\[ (\text{where } f_1 \text{ is the load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N)} \]
\[ R_y = \text{ratio of expected yield stress to specified minimum yield stress} \]
\[ Z_{ho} = \text{nominal plastic section modulus of beam, in.}^3 (\text{mm}^3) \]

User Note: The load combination of 1.2D + f_1L + 0.2S is in conformance with ASCE/SEI 7-16. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 for S (snow) when the roof configuration is such that it does not shed snow off the structure.

(ii) The value of \( \sum M'_{pc} \) shall be the sum of the projections of the nominal flexural strengths \( (M_{pc}) \) of the column above and below the connection joint, at the location of theoretical hinge formation in the column (i.e., one quarter the column depth above and below the extreme fibers of the side plates), to the beam centerline, with a reduction for the axial force in the column (Figure 11.10). The nominal flexural strength of the column shall be computed as:

\[
\sum M'_{pc} = \sum Z_{hc} \left( F_{yc} - P_e / A_y \right)
\]

where

\[ F_{yc} = \text{the minimum specified yield strength of the column at the connection, ksi (MPa)} \]
\[ H = \text{story height, in. (mm)} \]
\( H_b \) = distance along column height from \( \frac{1}{4} \) of column depth above top edge of lower story side plates to \( \frac{1}{4} \) of column depth below bottom edge of upper story side plates, in. (mm)

\( P_{uc}/A_g \) = ratio of column axial compressive load, computed in accordance with load and resistance factor provisions, to gross area of the column, ksi (MPa)

\( Z_c \) = plastic section modulus of column, in.\(^3\) (mm\(^3\))

\( Z_{ec} \) = the equivalent plastic section modulus of column \((Z_c)\) at a distance of \( \frac{1}{4} \) column depth from top and bottom edge of side plates, projected to beam centerline, in.\(^3\) (mm\(^3\)), and computed as:

\[
Z_{ec} = \frac{Z_c (H/2)}{H_b/2} = \frac{Z_c H}{H_b}
\]

(b) For IMF systems, the column-beam moment ratio shall conform to the requirements of the AISC Seismic Provisions.

**Fig. 11.10.** Force and distance designations for computation of column-beam moment ratios.

### 11.5. CONNECTION WELDING LIMITATIONS

Filler metals for the welding of beams, columns and plates in the SidePlate connection shall meet the requirements for seismic force-resisting system welds in the AISC Seismic Provisions.

**User Note:** Mechanical properties for filler metals for seismic force-resisting system welds are detailed in AWS D1.8/D1.8M as referenced in the AISC Seismic Provisions.

The following welds are considered demand critical welds:

1. Shop fillet weld \{2\} that connects the inside face of the side plates to the wide-flange or HSS columns (see plan views in Figure 11.11, Figure 11.12 and Figure 11.13) and for biaxial dual-strong axis configurations connects the outside face of the secondary side plates to the outside face of primary side plates (see Figure 11.5).

2. Shop fillet weld \{5\} that connects the edge of the beam flange to the beam flange cover plate or angles (see Figures 11.14a and 11.14b).

3. Shop fillet weld \{5a\} that connects the outside face of the beam flange to the beam flange U-shaped cover plate or angles (see Figures 11.14a and 11.14b).
(4) Field fillet weld \{7\} that connects the beam flange cover plates to the side plates (see Figure 11.15a), or connects the HSS beam flange to the side plates.

(5) Fillet weld \{8\} that connects the top angles to the side plates in the field-bolted connection.

11.6. CONNECTION DETAILING

The following designations are used herein to identify plates and welds in the SidePlate connection shown in Figures 11.11 through 11.15:

1. Plates/Angles

\{A\} Side plate, located in a vertical plane parallel to the web(s) of the beam, connecting frame beam to column.

\{B\} Beam flange cover plate bridging between side plates \{A\}, as applicable.

\{C\} Vertical shear plate.

\{D\} Horizontal shear plate (HSP). This element transfers horizontal shear from the top and bottom edges of the side plates \{A\} to the web of a wide-flange column.

\{E\} Erection angle. One of the possible vertical shear elements \{F\}.

\{F\} Vertical shear elements (VSE). These elements, which may consist of angles and plates or bent plates, transfer shear from the beam web to the outboard edge of the side plates \{A\}.

\{G\} Longitudinal angles welded to the side plates \{A\} for connecting the beam flange cover plate (field-bolted connection).

\{H\} Longitudinal angles welded to the beam flange for connecting to the side plates \{A\} (field-bolted connection).

\{T\} Horizontal plates welded to the side plates \{A\} for connecting the beam flange cover plate as an alternative for Angle \{G\} (field-bolted connection).

2. Welds

\{1\} Shop fillet weld connecting exterior edge of side plate \{A\} to the horizontal shear plate \{D\} or to the face of a built-up box column or HSS section.

\{2\} Shop fillet weld connecting inside face of side plate \{A\} to the tip of the column flange, or to the corner of an HSS or built-up column section; and for biaxial dual-strong axis configurations connects outside face of secondary side plates to outside face of primary side plates.

\{3\} Shop fillet weld connecting horizontal shear plate \{D\} to wide-flange column web. Weld \{3\} is also used at the column flanges where required to resist orthogonal loads through the connection due to collectors, chords or cantilevers.

\{4\} Shop fillet weld connecting vertical shear elements \{F\} to the beam web, and where applicable, the vertical shear plate \{C\} to the erection angle \{E\}.

\{5\} Shop fillet weld connecting beam flange tip to cover plate \{B\}/angles \{H\}.

\{5a\} Shop weld connecting outside face of beam flange to cover plate \{B\} (or to the face of the beam flange with the angles \{H\}).
Fig. 11.11. One-sided SidePlate moment connection (A-type), column shop detail.
Fig. 11.12. Two-sided SidePlate moment connection (B-type), column shop detail.
Fig. 11.13. Two-sided SidePlate moment connection (C-type), column shop detail.

Fig. 11.14a. Beam shop detail (field-welded).
Fig. 11.14b. Beam shop detail, field-bolted standard (Configuration A)
Fig. 11.15. Beam-to-side plate field erection detail. (a) elevation and section B-B, field-welded; (b) elevation and section B-B, field-bolted standard (Configuration A).

3. Bolts

(1) Bolts shall be arranged symmetrically about the axis of the beam.

(2) Types of holes:

(a) Standard holes shall be used in the horizontal angles {G} and {H}.

(b) Either standard or oversized holes shall be used in the side plates and cover plates.

(c) Either standard or short-slotted holes (with the slot parallel to the beam axis) shall be used in the angle of the vertical shear element if applicable (VSE).

(3) Bolt holes in the side plates, cover plates and longitudinal angles shall be made by drilling, thermally cutting with grinding (with a surface roughness profile not exceeding 1000 micro-inches) or by sub-punching and reaming. Punched holes are not permitted.

(4) All bolts shall be installed as pretensioned high-strength bolts.
Bolts shall be pretensioned high-strength bolts conforming to ASTM F3125 grade A490 or A490M or F2280. Bolt diameter is limited to 1-1/2 in. (38 mm) maximum.

The use of shim plates between the side plates and the cover plate or angles is permitted at either or both locations, subject to the limitations of RCSC Specification.

Faying surfaces of side plates, cover plate and angles shall have a Class A slip coefficient or higher.

User Note: The use of oversized holes in the side plates and cover plates with pretensioned bolts that are not designed as slip critical is permitted, consistent with Section D2.2 of the Seismic Provisions. Although standard holes are permitted in the side plate and cover plate, their use may result in field modifications to accommodate erection tolerances.

11.7. DESIGN PROCEDURE

Step 1. Choose trial frame beam and column section combinations that satisfy geometric compatibility based on Equation 11.4-1 or 11.4-1M. For SMF systems, check that the section combinations satisfy the preliminary column-beam moment ratio given by:

$$\sum (F_{yc}Z_{xc}) > 1.7 \sum (F_{yb}Z_{xb})$$  \hspace{1cm} (11.7-1)

where

- $F_{yb}$ = specified minimum yield stress of beam, ksi (MPa)
- $F_{yc}$ = specified minimum yield stress of column, ksi (MPa)
- $Z_{xb}$ = plastic section modulus of beam, in.$^3$ (mm$^3$)
- $Z_{xc}$ = plastic section modulus of column, in.$^3$ (mm$^3$)

Step 2. Approximate the effects on global frame performance of the increase in lateral stiffness and strength of the SidePlate moment connection, due to beam hinge location and side plate stiffening, in the mathematical elastic steel frame computer model by using 100% rigid offset in the panel zone, and by increasing the moment of inertia, elastic section modulus and plastic section modulus of the beam to approximately three times that of the beam, for a distance of approximately 77% of the beam depth beyond the column face (approximately equal to the extension of the side plate beyond the face of the column), illustrated in Figure 11.16.

SMF beams that have a combination of shallow depth and heavy weight (i.e., beams with a relatively large flange area such as those found in the widest flange series of a particular nominal beam depth) require that the extension of the side plate $\{A\}$ be increased, up to the nominal depth of the beam, $d$ and $1.7d$, for the field-welded and field-bolted connections respectively.

User Note: This increase in extension of side plate $\{A\}$ of the field-welded connection, lengthens fillet weld $\{7\}$, thus limiting the extremes in the size of fillet weld $\{7\}$. Regardless of the extension of the side plate $\{A\}$, the plastic hinge occurs at a distance of $d/3$ and $d/6$ from the end of the side plates for the field-welded and field bolted connections, respectively.

Step 3. Confirm that the frame beams and columns satisfy all applicable building code requirements, including, but not limited to, stress or strength checks and design story drift checks.

Step 4. Confirm that the frame beam and column sizes comply with prequalification limitations per Section 11.3.
**Fig. 11.16. Modeling of component stiffness for linear-elastic analysis.**

**Step 5.** Upon completion of the preliminary and/or final selection of lateral load resisting frame beam and column member sizes using SidePlate connection technology, the engineer of record submits a computer model to SidePlate Systems, Inc. In addition, the engineer of record shall submit the following additional information, as applicable:

$$V_{gravity} = \text{factored gravity shear in moment frame beam resulting from the load combination of}$$

$$1.2D + f_1L + 0.2S \quad (\text{where} \ f_1 \ \text{is the load factor determined by the applicable building code for live loads, but not less than 0.5}, \ \text{kips (N)})$$

**User Note:** The load combination of $1.2D + f_1L + 0.2S$ is in conformance with ASCE/SEI 7-16. When using the 2015 International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 for $S$ (snow) when the roof configuration is such that it does not shed snow off of the structure.

(a) Factored gravity shear loads, $V_1$ and/or $V_2$, from gravity beams that are not in the plane of the moment frame, but connect to the exterior face of the side plate(s) where

$$V_1, V_2 = \text{beam shear force resulting from the load combination of} \ 1.2D + f_1L + 0.2S$$

$\ \text{(where} \ f_1 \ \text{is the load factor determined by the applicable building code for live loads, but not less than 0.5}, \ \text{kips (N)})$

(b) Factored gravity loads, $M_{cant}$ and $V_{cant}$, from cantilever gravity beams that are not in the plane of the moment frame, but connect to the exterior face of the side plate(s) where

$$M_{cant} = \text{cantilever beam moment resulting from code applicable load combinations, kip-in. (N-mm)}$$

$$V_{cant} = \text{cantilever beam shear force resulting from code applicable load combinations, kips (N)}$$

**User Note:** Code applicable load combinations may need to include the following when looking at cantilever beams: $1.2D + f_1L + 0.2S$ and $(1.2 + 0.2S_{ref})D + \rho Q_P + f_1L + 0.2S$, which are in conformance with ASCE/SEI 7-16. When using the 2015 International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 for $S$ (snow) when the roof configuration is such that it does not shed snow off of the structure.

(c) Perpendicular amplified seismic lateral drag or chord axial forces, $A_{\perp}$, transferred through the SidePlate connection.
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(d) In-plane factored lateral drag or chord axial forces, $A_{\parallel}$ transferred along the frame beam through the SidePlate connection.

$$A_{\parallel} = \text{amplified seismic drag or chord force resulting from applicable building code, kips (N)}$$

User Note: Where linear-elastic analysis is used to determine perpendicular collector or chord forces used to design the SidePlate connection, such forces should include the applicable load combinations specified by the building code, including considering the amplified seismic load ($\Omega_o$). Where nonlinear analysis or capacity design is used, collector or chord forces determined from the analysis are used directly, without consideration of additional amplified seismic load.

Step 6. Upon completion of the mathematical model review and after additional information has been supplied by the engineer of record, SidePlate engineers provide project-specific connection designs. Strength demands used for the design of critical load transfer elements (plates, welds and column) throughout the SidePlate beam-to-column connection and the column are determined by superimposing maximum probable moment, $M_{pr}$, at the known beam hinge location, then amplifying the moment demand to each critical design section, based on the span geometry, as shown in Figure 11.6, and including additional moment due to gravity loads. For each of the design elements of the connection, the moment demand is computed per Equation 11.7-2 and the associated shear demand is computed as:

$$M_{\text{group}} = M_{pr} + V_u x$$ \hspace{1cm} (11.7-2)

where

$$C_{pr} = \text{connection-specific factor to account for peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions}.$$

The equation used in the calculation of the $C_{pr}$ is provided by SidePlate as part of the connection design.

User Note: In practice, the value of $C_{pr}$ for SidePlate connections as determined from testing and nonlinear analysis ranges from 1.15 to 1.35.

Step 7. SidePlate designs all connection elements per the proprietary connection design procedures contained in SidePlate Connection Design Software (version 16 for field-welded and version 17 for field-bolted connections). The version is clearly indicated on each page of calculations. The final design includes structural notes and details for the connections.

User Note: The procedure uses an ultimate strength design approach to size plates and welds, incorporating strength, plasticity and fracture limits. For welds, an ultimate strength analysis
incorporating the instantaneous center of rotation may be used as described in AISC Steel Construction Manual Section J2.4b. For bolt design, eccentric bolt group design methodology incorporating ultimate strength of the bolts is used. Refer to the Commentary for an in-depth discussion of the process.

In addition to the column web panel zone strength requirements, the column web shear strength shall be sufficient to resist the shear loads transferred at the top and bottom of the side plates. The design shear strength of the column web shall be determined in accordance with AISC Specification Section G2.1.

**Step 8.** Engineer of record reviews SidePlate calculations and drawings to ensure that all project specific connection designs have been appropriately designed and detailed based on information provided in Step 5.
CHAPTER 14

SLOTTEDWEB™ (SW) MOMENT CONNECTION

The user’s attention is called to the fact that compliance with this chapter of the standard requires use of an invention covered by patent rights.† By publication of this standard, no position is taken with respect to the validity of any claim(s) or of any patent rights in connection therewith. The patent holder has filed a statement of willingness to grant a license under these rights on reasonable and nondiscriminatory terms and conditions to applicants desiring to obtain such a license, and the statement may be obtained from the standards developer.

14.1. GENERAL

The SlottedWeb™ moment connection features slots in the web of the beam that are parallel and adjacent to each flange, as shown in Figure 14.1. Inelastic behavior is expected to occur through yielding and buckling of the beam flanges in the region of the slot accompanied by yielding of the web in the region near the end of the shear plate.

Fig. 14.1. SW Beam-to-column moment connection.

14.2. SYSTEMS

The SlottedWeb™ (SW) connections are prequalified for the use in special moment frames (SMF) within the limits of these provisions.

14.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

†The SlottedWeb™ connection configuration illustrated herein is protected by one or more of the following U.S. patents: U.S. Pat. Nos. 5,680,738; 6,237,303; 7,047,695; all held by Seismic Structural Design Associates.
(1) Beams shall be rolled wide-flange or built-up I shaped members conforming to the requirements of Section 2.3.

(2) Beam depth shall be limited to a maximum of W36 (W920) for rolled shapes. The depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes.

(3) Beam weight shall be limited to a maximum 400 lb/ft (600 kg/m).

(4) Beam flange thickness shall be limited to a maximum of 2¼ in. (64 mm).

(5) The clear span-to-depth ratio of the beam shall be limited to 6.4 or greater

(6) Width-to-thickness ratios for the flanges and webs of the beam shall conform to the requirements of the AISC Seismic Provisions.

(7) Lateral bracing of the beams shall be provided in conformance with the AISC Seismic Provisions. No supplemental lateral bracing is required at the plastic hinges.

(8) The protected zones as shown in Figure 14.2 consist of:

(a) The portion of the beam web between the face of the column to the end of the slots plus one-half the depth of the beam, \( d_w \), beyond the slot end and

(b) The beam flange from the face of the column to the end of the slot plus one-half the beam flange width, \( b_f \)

Fig. 14.2. Protected zones.

2. Column Limitations

(1) Columns shall be of any of the rolled shapes or built-up sections permitted in Section 2.3.
The beam shall be connected to the flange of the column.

Rolled shape column depths shall be limited to W36 (W920). The depth of built-up wide-flange columns shall not exceed that allowed for rolled shapes. Flanged cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes. Built-up box columns shall not have a width or depth exceeding 24 in. (610 mm). Boxed wide flange columns shall not have a width or depth exceeding 24 in. (610 mm) if participating in orthogonal moment frames.

There is no limit on the weight per foot of columns.

There are no additional requirements for flange thickness.

Width-thickness ratios for the flanges and web of columns shall conform to the requirements of the AISC Seismic Provisions.

Lateral bracing of columns shall conform to the requirements of the AISC Seismic Provisions.

**14.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS**

Beam-to-Column connections shall satisfy the following limitations:

(1) Panel zones shall conform to the requirements of the AISC Seismic Provisions.

(2) Column-beam ratios shall be limited as follows:

The moment ratio shall conform to the AISC Seismic Provisions. The value of \( \sum M_{p} \) shall be taken equal to \( \sum (M_{w} + M_{w}) \), where \( M_{w} \) is the probable maximum moment of the beam, defined in Section 14.8, Step 3, and where \( M_{w} \) is the additional moment due to shear amplification from the plastic hinge, which is located at the end of the shear plate, to the centerline of the column.

\[
M_{w} = V_{beam} \left( l_{p} + d_{col} / 2 \right)
\]

where

- \( V_{beam} \) = shear at the beam plastic hinge, kips (N), computed according to step 3 in Section 14.8
- \( d_{col} \) = depth of the column, in. (mm)
- \( l_{p} \) = width of the shear plate, in. (mm)

**14.5. BEAM FLANGE-TO-COLUMN FLANGE WELD LIMITATIONS**

Beam flange to column flange connections shall satisfy the following limitations:

(1) Beam flanges shall be connected to the column flanges using complete joint penetration (CJP) groove welds. Beam flange welds shall conform to the requirements of demand critical welds in the AISC Seismic Provisions.

(2) Weld access hole geometry shall conform to the requirements of the AISC Specification.

**14.6. BEAM WEB AND SHEAR PLATE CONNECTION LIMITATIONS**

Bean web and shear plate connections shall satisfy the following limitations:

The shear plate shall be welded to the column flange using a CJP groove weld, a PJP groove weld, or a combination of PJP and fillet welds. The shear plate shall be bolted to the beam web and fillet welded to the beam web. The horizontal fillet welds at the top and bottom of the shear plate shall be
terminated at a distance not less than one fillet weld size from the end of the beam. The beam web shall be connected to the column flange using a CJP groove weld extending the full height of the shear plate. The shear plate connection shall be permitted to be used as backing for the CJP groove weld. The beam web-to-column flange CJP groove weld shall conform to the requirements for demand critical welds in the AISC Seismic Provisions.

(a) If weld tabs are used, they need not be removed.

(b) If weld tabs are not used, the CJP groove weld shall be terminated in a manner that minimizes notches and stress concentrations, such as with the use of cascaded welds. Cascaded welds shall be performed at a maximum angle of 45° relative to the axis of the weld. Nondestructive testing (NDT) of the cascaded weld ends need not be performed.

14.7. FABRICATION OF BEAM WEB SLOTS

The beam web slots shall be made using thermal cutting or milling of the slots and holes or by drilling the holes to produce surface roughness in the slots or holes not exceeding 1,000 micro-inches (25 microns). Gouges and notches that may occur in the cut slots shall be repaired by grinding. The beam web slots shall terminate at thermally cut or drilled 1 1/8-in. (27 mm) diameter holes for beams nominal 24 in. (610 mm) deep or greater or 13/16-in. (21 mm) holes for beams less than nominal 24 in. (610 mm) deep. Punched holes are not permitted. The slot widths and tolerances are shown in Figure 14.3. The length of the 1/8-in. slot shall be at least equal to the width of the shear plate, but need not exceed half the slot length, \( l_s \). The transition from the 1/8-in. (3 mm) slot to the 3/16-in. slot (6 mm) shall not have a slope greater than 1 vertical to 3 horizontal.

\[ l_s = 1.5b_f \]  

(14.8-1)
\[ I_s = 0.60 t_{bf} \sqrt{\frac{E}{F_{ye}}} \quad \text{(14.8-2)} \]

\[ I_s = \frac{d}{2} \quad \text{(14.8-3)} \]

\[ I_s = l_p + \frac{(l_b - l_p)}{10} \quad \text{(14.8-4)} \]

where

\[ E \] = steel elastic modulus, ksi (MPa)
\[ F_{ye} \] = expected yield strength of steel beam, ksi (MPa)
\[ R_y = \text{ratio of the expected yield stress to the minimum yield stress, } F_y \]
\[ b_f \] = beam flange width, in. (mm)
\[ d \] = nominal depth of the beam, in. (mm)
\[ l_b \] = half the clear span length of beam, in. (mm)
\[ l_p \] = width of the shear plate, in. (mm)
\[ t_{bf} \] = beam flange thickness, in. (mm)

Step 2. Design the shear plate. Steel with a specified minimum yield stress of 50 ksi (345 MPa) shall be used. The shear plate width shall not be greater than 1/2 the length of the beam web slot or 6 in. (152 mm), but not shorter than 1/3 the beam slot length. The height, \( h \), of the shear plate is determined as:

\[ h = T - 2 \text{ in.} \pm 1 \text{ in.} \quad \text{(14.8-5)} \]
\[ h = T - 50 \text{ mm} \pm 25 \text{ mm} \quad \text{(14.8-5M)} \]

where \( T \) is defined in the AISC Steel Construction Manual for wide-flange shapes. The minimum shear plate thickness shall be equal to at least 2/3 of the beam web thickness but not less than 3/8 in. (10 mm).

The minimum required shear plate thickness, \( t_p \), is based upon the additional moment due to shear amplification from the end of the shear plate to the face of the column. Use the plate elastic section modulus to conservatively compute the shear plate minimum thickness.

\[ t_p = C_p r \left( \frac{6}{h^2} \right) R_y \left( \frac{Z_{beam} t_p}{l_b - l_p} \right) \quad \text{(14.8-6)} \]

where

\[ Z_{beam} = \text{plastic modulus of the beam, in.}^3 (\text{mm}^3) \]

Step 3. Design the shear plate-to-beam web weld. The shear plate shall be welded to the beam web with an eccentrically loaded fillet weld group. The weld shall be designed to resist \( M_{weld} \) and \( V_{weld} \) and to account for the resulting eccentricity, \( e_x \). These values are determined as follows:

\[ M_{weld} = C_p r \left( \frac{t_p}{t_p + t_{bw}} \right) \left( \frac{h}{T} \right)^2 Z_{web} R_y F_y \quad \text{(14.8-7)} \]

\[ V_{weld} = V_{beam} \left( \frac{t_p}{t_p + t_w} \right) \quad \text{(14.8-8)} \]
\[ e_x = \frac{M_{\text{weld}}}{V_{\text{weld}}} \]  

where

\[ M_{\text{weld}} = \text{moment resisted by the shear plate, kip-in. (N-mm)} \]
\[ V_{\text{beam}} = \text{shear at the beam plastic hinge, kips (N)} \]

\[ V_{\text{weld}} = \frac{M_{\text{pr}}}{l_b} + V_{\text{gravity}} \]  

and where

\[ M_{\text{pr}} = C_{\text{pr}} R_y F_y Z_{\text{beam}} \]
\[ V_{\text{gravity}} = \text{beam shear force resulting from the load combination } 1.2D + f_i L + 0.2S \]

(where \( f_i \) is the load factor determined by the local building code for live loads, but not less than 0.5), kips (N)

User Note: The load combination of \( (1.2D + f_i L + 0.2S) \) is in conformance with ASCE/SEI-7. When using the International Building Code, a factor of 0.7 shall be used in lieu of the factor 0.2 for \( S \) (snow) when the roof configuration is such that it does not shed snow off the structure.

\[ V_{\text{weld}} = \text{shear resisted by the shear plate, kips (N)} \]
\[ Z_{\text{beam}} = \text{plastic modulus of the beam, in.}^3 \text{ (mm}^3) \]
\[ Z_{\text{web}} = \text{plastic section modulus of the beam web, in.}^3 \text{ (mm}^3) \]

\[ e_x = \frac{t_w T^2}{4} \]  

\[ e_x = \text{eccentricity of the shear plate weld, in. (mm)} \]
\[ t_{bw} = \text{thickness of the beam web, in. (mm)} \]

User Note: The AISC Manual design tables for “Eccentrically Loaded Weld Groups” may be used to design the shear plate-to-beam web fillet weld. Use the height and width of the shear plate and the shear eccentricity, \( e_x \), as shown in Figure 14.4, to determine the weld design table coefficients.

**Fig. 14.4. Eccentrically loaded weld group.**

**Step 4.** Design the shear plate to column flange weld.

The required strength of the weld connecting the shear plate to the column flange shall be equal to the nominal strength of the eccentrically loaded weld group as calculated according to Step 3.
Step 5. Select the high strength pretensioned bolts in standard holes for the shear plate-to-beam web connection to serve as both erection bolts and to stabilize the beam web from lateral buckling at the column flange. These bolts shall have a maximum bolt spacing of 6 in. (150 mm) on center over the full height of the plate. The diameter of the bolts shall be equal to or greater than the thickness of the beam web.

Step 6. Compute the probable maximum moment at the column face, $M_f$, for use in checking continuity plate and panel zone requirements.

$$M_f = M_{pr} + V_{beam} I_p$$  \hspace{1cm} (14.8-12)

Step 7. Check the shear strength of the beam according to AISC Specification Chapter G.

Step 8. Check continuity plate requirements according to Section 2.4.4

Step 9. Check column panel zone according to Section 14.4

Step 10. Check column-beam moment ratio according to Section 14.4
COMMENTARY
on Supplement No. 1
to AISC 358-16
Prequalified Connections for
Special and Intermediate
Steel Moment Frames for
Seismic Applications

Draft dated January 12, 2018

This Commentary is not part of ANSI/AISC 358-16s1, Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications. It is included for informational purposes only.

INTRODUCTION
The Commentary furnishes background information and references for the benefit of the design professional seeking further understanding of the basis, derivations and limits of the Standard.

The Standard and Commentary are intended for use by design professionals with demonstrated engineering competence.
CHAPTER 11

SIDEPLATE® MOMENT CONNECTION

11.1. GENERAL

The SidePlate® moment connection is a post-Northridge connection system that uses a configuration of redundant interconnecting structural plates, fillet weld groups and high strength pretensioned bolts (as applicable), which act as positive and discrete load transfer mechanisms to resist and transfer applied moment, shear and axial load from the connecting beam(s) to the column. This load transfer minimizes highly restrained conditions and triaxial strain concentrations that typically occur in flange-welded moment connection geometries. The connection system is used for both new and retrofit construction and for a multitude of design hazards such as earthquakes, extreme winds, and blast and progressive collapse mitigation.

The wide range of applications for SidePlate® connection technology, including the methodologies used in the fabrication and erection shown herein, are protected by one or more U.S. and foreign patents identified at the bottom of the first page of Chapter 11. Information on the SidePlate® moment connection can be found at www.sideplate.com. SidePlate® moment connections not specifically designed by SidePlate Systems, Inc. shall be considered unauthorized and not prequalified and shall not be manufactured.

The SidePlate® moment connections are designed and detailed in two types:

1. Field-welded connection
2. Field-bolted connection

Both types are fully restrained connections of beams to columns. Figures 11.1 and 11.2 show the field-welded and field-bolted connections’ various configurations, respectively. The field-bolted connection is available in two configurations, referred to as Configuration A (standard) and Configuration B (narrow).

Moment frames that utilize the SidePlate® connection system may be constructed using one of three methods. The most common construction method uses a full-length beam for erection, namely SidePlate FRAME® configuration, as shown in Figure C-11.1 (a) and (b). This method employs a full-length beam assembly consisting of the beam with shop-installed cover plates \{B\}/angles \{H\} (if required) and vertical shear elements (as applicable), which are either fillet-welded or bolted near the ends of the beam depending on the type of the connection.

Column assemblies are typically delivered to the job site with the horizontal shear plates \{D\} and side plates \{A\} shop welded to the column at the proper floor framing locations. Where built-up box columns are used, horizontal shear plates \{D\} are not required, nor applicable.

For the field-welded option: During frame erection, the full-length beam assemblies are lifted up in between the side plates \{A\} that are kept spread apart at the top edge of the side plates \{A\} with a temporary shop-installed spreader [Figure C-11.1 (a)]. A few bolts connecting the beam’s vertical shear plates \{C\} (shear elements as applicable) to adjacent free ends of the side plates \{A\} are initially inserted to provide temporary shoring of the full-length beam assembly, after which the temporary spreader is removed. The remaining erection bolts (as many as can be installed) are then inserted and installed to a snug tight condition. These erection bolts can also act as a clamp to effectively close or minimize potential root gaps that might have existed between the interior face of the side plates \{A\} and the longitudinal edges of the top cover plate \{B\} while bringing the top face of the wider bottom cover plate \{B\} into a snug fit with the bottom edges of the side plates \{A\}. To complete the field assembly, four horizontal fillet welds joining the side plates \{A\} to the cover plates \{B\} are then deposited in the horizontal welding position (Position 2F per AWS D1.1/D1.1M), and, when applicable, two vertical single-pass field fillet welds joining the side plates \{A\} to the vertical shear elements (VSE) are deposited in the vertical welding position (Position 3F per AWS D1.1/D1.1M). Alternately this can be...
configured such that the width of bottom cover plate \{B\} is equal to the width of the
top cover plate \{B\} (i.e., both cover plates \{B\} fit within the separation of the side
plates \{A\}, which would also be slightly deeper in their lengths to accommodate), in
lieu of the bottom cover plate \{B\} being wider than the distance between side plates
\{A\}. Note that when this option is selected by the engineer, the two bottom fillet
welds connecting the bottom cover plates \{B\} to the side plates \{A\} will be deposited
in the overhead welding position (Position 4F per AWS D1.1/D1.1M).

**For the field-bolted option:** During frame erection, the full-length beam assemblies
are dropped down in between the side plates \{A\} that are kept spread apart at the
bottom edge of the side plates \{A\} with a temporary shop-installed spreader (Figure
C-11.1b). A few bolts/fasteners assemblies connecting the beam’s top cover plate \{B\}
(or vertical shear plates \{C\} as applicable) to adjacent free ends of the longitudinal
angles on the side plates \{A\} (or the side plates \{A\} themselves) are initially inserted
to provide temporary shoring of the full-length beam assembly, after which the
temporary spreader is removed. Shim plates may be installed between the side plates
\{A\} and the cover plate \{B\} or longitudinal angles if required. The remaining
bolts/fastener assemblies are then inserted to a snug tight specification in a systematic
assembly within the joint, progressing from the most rigid part of the joint until the
connected plies are in as firm as contact as practicable. These bolts should clamp and
effectively minimize any gaps that might have existed between the interior face of the
side plates \{A\} and the longitudinal edges of the angles and that of the interface
between the bottom face of the top cover plate \{B\} and the top longitudinal angles
\{G\} on the exterior face of the side plates \{A\} (Configuration A only). Note the
standard configuration (Configuration A) has a pair of angles attached to the bottom
flange of the beam and the narrow configuration (Configuration B) consists of pairs
of angles attached to both the top and bottom flanges of the beam. To complete the
field assembly, the second step of the pretensioning methodology is the subsequent
systematic pretensioning of all bolt/fastener assemblies; they shall progress in a
similar manner as was done for the snug tight condition, from the most rigid part of
the joint that will minimize relation of previously pretensioned bolts.

Where the full-length beam erection method (SidePlate FRAME® configuration), is
not used, the original SidePlate® moment configuration may be used (2nd method).
The original SidePlate® moment configuration utilizes the link-beam erection method,
which connects a link beam assembly to the beam stubs of two opposite column tree
assemblies with field complete-joint-penetration (CJP) groove welds (Figures C-
11.1c and 11.1d). As a third method, in cases where moment frames can be shop
prefabricated and shipped to the site in one piece, no field bolting or welding is
required (Figure C-11.1e).

The SidePlate® moment connection is proportioned to develop the probable
maximum moment capacity of the connected beam. Beam flexural, axial and shear
forces are typically transferred to the top and bottom rectangular cover plates \{B\} via
four shop horizontal fillet welds that connect the edges of the beam flange tips to the
corresponding face of each cover plate \{B\} (two welds for each beam flange). When
the U-shaped cover plates \{B\} or angles \{H\} are used, the same load transfer occurs
via four shorter shop horizontal fillet welds that connect the edge of the beam flange
tips to the corresponding face of each cover plate \{B\}/angles \{H\} (two welds for each
beam flange), as well as two shop horizontal fillet welds that connect the outside
faces of the beams top and bottom flanges to the corresponding inside edge of each
U-shaped cover plate \{B\} (for the conditions with pairs of angles \{H\}, there are two
welds that will connect each angle to the corresponding beam flange face). These
same forces are then transferred from the cover plates \{B\} or pairs of angles \{H\} to
the side plates \{A\} via either four field horizontal fillet welds (in the field-welded
connection) or four lines of bolts (in the field-bolted connection) that connect the
cover plates \{B\} or pairs of angles \{H\} to the side plates \{A\}. The side plates \{A\}
transfer all of the forces from the beam (including that portion of shear in the beam
that is transferred from the beam’s web via vertical shear elements, or via the cover
plate \{B\} and pairs of angles \{H\}, as applicable), across the physical gap to the
column via shop fillet welding (or flare bevel welding, as required) of the side plates
\{A\} to the column flange tips (a total of four shop fillet welds; two for each side plate
\{A\}), and to complete the weld group there are two horizontally placed shop fillet
welds at the top and bottom of each side plates \{A\}. These welds may attach directly
to the face of a box or HSS column, or they may attach to horizontal shear plates \{D\}
as applicable (a total of four shop fillet welds two for each side plate \{A\}). The
horizontal shear plates \{D\} are in turn shop fillet welded to the column web and
under certain conditions, also to the inside face of column flanges.

Fig. C-11.1. SidePlate® moment connection construction methods: (a) full-length
beam erection method (SidePlate FRAME® configuration; field-welded); (b) full-
length beam erection method (SidePlate® moment standard configuration; field-
SidePlate Systems, Inc., developed, tested and validated the SidePlate® moment connection design methodology, design controls, critical design variables, and analysis procedures. The development of the SidePlate FRAME® configuration that employs the full-length beam erection method builds off of the research and testing history of its proven predecessor—the original configuration and its subsequent refinements. Moreover, from 2015 to 2017, the field-bolted connection was developed and successfully tested and validated. It resulted in further performance enhancements: optimizing the use of connection component materials with advanced analysis methods and maximizing the efficiency, simplicity and quality control of its fabrication and erection processes. Following the guidance of the AISC Seismic Provisions, the validation of the field-welded and field-bolted SidePlate FRAME® configuration consists of:

(a) Analytical testing conducted by SidePlate Systems, Inc. utilizing nonlinear finite element analysis (FEA) for built-up and rolled shapes, plates and welds and validated inelastic material properties by physical testing.

(b) In addition to the tests conducted between 1994 and 2006 utilizing the original configuration, SidePlate Systems, Inc., conducted physical validation testing with a full-length beam assembly (SidePlate FRAME® configuration) at the Lehigh University Center for Advanced Technology for Large Structural Systems (ATLSS) in 2010 (Hodgson et al., 2010a, 2010b, and 2010); a total of six cyclic tests, and at the University of California, San Diego (UCSD), Charles Lee Powell Laboratories in 2012 and 2013 a total of two cyclic tests was conducted (Minh Huynh and Uang, 2012), and a total of one biaxial cyclic test (Minh Huynh and Uang, 2013). The biaxial moment connection test subjected the framing in the orthogonal plane to a constant shear, creating a moment across the column-beam joint equivalent to that created by the probable maximum moment at the plastic hinge of the primary beam, while the framing in the primary plane was simultaneously subjected to the qualifying cycle loading specified by the AISC Seismic Provisions (AISC, 2016a). More recently, a physical testing program was conducted at the University of California, San-Diego (Mashayekh and Uang, 2016; Reynolds and Uang, 2017) to validate the performance of the field-bolted SidePlate® moment connection. A total of seven cyclic tests, two of which utilized HSS columns and one of which utilized built-up box columns, were conducted. The purpose of these tests was to confirm adequate global inelastic rotational behavior of either field-welded or field-bolted SidePlate® moment connections with parametrically selected member sizes, corroborated by analytical testing, and to identify, confirm and accurately quantify important limit state thresholds for critical connection components to objectively set critical design controls. The 2015-2017 testing program at UCSD additionally aimed to verify the satisfactory performance of HSS columns with width-to-thickness ratios of up to 21 in SidePlate® moment connections through the application of a significant axial load on the column in addition to the AISC Seismic Provisions loading protocol. The testing program also aimed to verify the satisfactory performance of SidePlate® moment connections with built-up box columns without any internal horizontal shear plates (D) or stiffener (continuity) plates, where flange and web plates of built-up box columns are continuously connected by either fillet welds or PJP groove welds along the length of the column. It implies that no CJP welds will be required within a zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange, flange and web plates of boxed wide-flange columns in SidePlate® moment connections.

(c) Tests on SidePlate® moment connections, both uniaxial and biaxial applications, show that yielding is generally concentrated within the beam
section just outside the ends of the two side plates {A}. Peak strength of
specimens is usually achieved at an interstory drift angle of approximately 0.03
to 0.05 rad. Specimen strength then gradually reduces due to local and lateral-
torsional buckling of the beam. Ultimate failure typically occurs at interstory
drift angles of approximately 0.04 to 0.06 rad for the field-welded and 0.06 to
0.08 for the field-bolted connection by low-cycle fatigue fracture from local
buckling of the beam flanges and web.

To ensure predictable, reliable and safe performance of the SidePlate FRAME®
configuration when subjected to severe load applications, the inelastic material
properties, finite element modeling (FEM) techniques and analysis methodologies
that were used in its analytical testing were initially developed, corroborated and
honed based on nonlinear analysis of prior full-scale physical testing of the original
SidePlate® configuration. The finite element techniques and design methodologies
have been further refined and polished as a result of the testing program with field-
bolted connections at UCSD from 2015 to 2017.

The earliest physical testing of SidePlate® connections consisted of a series of eight
uniaxial cyclic tests, one biaxial cyclic test conducted at UCSD and a separate series
of large-scale arena blast tests. The blast tests consisted of an explosion followed by
monotonic loading using the following configurations: two blast tests (one with and
one without a concrete slab present), two blast-damaged progressive collapse tests
and one non-blast damaged test, conducted by the Defense Threat Reduction Agency
(DTRA) of the U.S. Department of Defense (DoD), at Kirtland Air Force Base,
Albuquerque, NM.

These extensive testing efforts have resulted in the ability of SidePlate Systems, Inc.
to:

(a) Reliably replicate and predict the global behavior of the SidePlate FRAME®
configuration compared to actual tests.
(b) Explore, evaluate and determine the behavioral characteristics, redundancies
and critical limit state thresholds of its connection components.
(c) Establish and calibrate design controls and critical design variables of the
SidePlate FRAME® configuration, as validated by physical testing.

Connection prequalification is based on the completion of several carefully prescribed
validation testing programs, the development of a safe and reliable plastic capacity
design methodology that is derived from ample performance data from 31 full-scale
tests of which two were biaxial, and the judgment of the CPRP. The connection
prequalification objectives have been successfully completed; the rudiments are
summarized below:

(a) System-critical limit states have been identified and captured by physical full-
scale cyclic testing and corroborated through nonlinear FEA.
(b) The effectiveness of identified primary and secondary component redundancies
of the connection system has been demonstrated and validated through
parametric performance testing—both physical and analytical.
(c) Critical behavioral characteristics and performance nuances of the connection
system and its components have been identified, captured and validated.
(d) Material sub-models of inelastic stress/strain behavior and fracture thresholds
of weld consumables and base metals have been calibrated to simulate actual
behavior.
(e) Sufficient experimental and analytical data on the performance of the
connection system have been collected and assessed to establish the likely yield
mechanisms and failure modes.
(f) Rational nonlinear FEA models for predicting the resistance associated with
each mechanism and failure mode have been employed and validated through
physical testing.
Based on the technical merit of the above accomplishments, a rational ultimate strength design procedure has been developed based on physical testing, providing confidence that sufficient critical design controls have been established to preclude the initiation of undesirable mechanisms and failure modes and to secure expected safe levels of cyclic rotational behavior and deformation capacity of the connection system for a given design condition.

11.2. SYSTEMS

The SidePlate® moment connection meets the prequalification requirements for special and intermediate moment frames in both traditional in-plane frame applications (one or two beams framing into a column) as well as orthogonal intersecting moment-resisting frames (corner conditions with two beams orthogonal to one another, as well as three or four orthogonal beams framing into the same column).

The SidePlate® moment connection has been used in moment-resisting frames with skewed and/or sloped beams with or without skewed side plates {A}, although such usage is outside of the scope of this standard.

SidePlate® moment connection’s unique geometry allows its use in other design applications where in-plane diagonal braces or diagonal dampers are attached to the side plates {A} at the same beam-to-column joint as the moment-resisting frame while maintaining the intended SMF or IMF level of performance. When such dual systems are used, supplemental calculations must be provided to ensure that the connection elements (plates and welds) have not only been designed for the intended SMF or IMF connection in accordance with the prequalification limits set herein, but also for the additional axial, shear and moment demands due to the diagonal brace or damper.

11.3. PREQUALIFICATION LIMITS

1. Beam Limitations

A wide range of beam sizes, including both wide flange and HSS beams, has been tested with the SidePlate® moment connection. For the field-welded connection, the smallest beam size was a W18×35 (W460×52) and the heaviest a W40×297 (W1000×443). For the field-bolted connection, the smallest beam size was W21×73 (W530×109) and the largest beam size was W40×397 (W1000×591). The deepest beam tested was W44×290 (W1100×433) with the depth of 43.6 in. (1107 mm).

Beam compactness ratios have varied from that of a W18×35 (W460×52) with $b/2t_f = 7.06$ to a W40×294 (W1000×438) with $b/2t_f = 7.06$ to a W40×294 (W1000×438) with $b/2t_f = 3.11$. For HSS beam members, tests have focused on small members such as the HSS 7×4×1/2 (HSS177.8×101.6×12.7) having ratios of $b/t = 5.60$ and $h/t = 12.1$. As a result of all the SidePlate Systems, Inc. testing programs, critical ultimate strength design parameters for the design and detailing of the SidePlate® moment connection system have been developed for general project use. These requirements and design limits are the result of a detailed assessment of actual performance data coupled with independent physical validation testing and/or corroborative analytical testing of full-scale test specimens using nonlinear FEA. It was the judgment of the CPRP that the maximum beam depth and weight of the SidePlate® moment connection would be limited to the nominal beam depth and approximate weight of the sections tested, as has been the case for most other connections.

Since the behavior and overall ductility of the SidePlate® moment connection system is defined by the plastic rotational capacity of the beam, the limit state for the SidePlate® moment connection system is ultimately the failure of the beam flange, away from the connection. Therefore, the limit of the beam’s hinge-to-hinge span-to-depth ratio of the beam, $L_h/d$, is based on the demonstrated rotational capacity of the beam.

As an example, for test specimen 3 tested at Lehigh University (Hodgson et al., 2010c), the W40×294 (W1000×438) beam connected to the W36×395 (W920×588) column reached two full cycles at 0.06 rad of rotation (measured at the centerline of
the column), which is significantly higher than the performance threshold of one cycle at 0.04 rad of rotation required for successful qualification testing by the AISC Seismic Provisions. Most of the rotation at that amplitude came from the beam rotation at the plastic hinge. With the rotation of the column at 0.06 rad, the measured rotation at the beam hinge was between 0.085 and 0.09 rad (see Figure C-11.2a). The tested half-span was 14.5 ft (4.42 m), which represents a frame span of 29 ft (8.84 m) and an \( L_h/d \) ratio of 5.5. Assuming that 100% of the frame system’s rotation comes from the beam’s hinge rotation (a conservative assumption because it ignores the rotational contributions of the column and connection elements), it is possible to calculate a minimum span at which the frame drift requirement of one cycle at 0.04 rad is maintained, while the beam reaches a maximum of 0.085 rad of rotation. Making this calculation gives a minimum span of 20 ft (6.1 m) and an \( L_h/d \) ratio of 3. Making this same calculation for the tests of the W36×150 (W920×223) beam (Minh Huynh and Uang, 2012; Figure C-11.2b) using an average maximum beam rotation of 0.08 rad of rotation, gives a minimum span of 18 ft, 10 in. (5.74 m) and an \( L_h/d \) ratio of 3.2. Given that there will be variations in the performance of wide-flange beams due to local effects such as flange buckling, it is reasonable to set the lower bound \( L_h/d \) ratio for the SidePlate® field-welded moment connection system at 4.5 for SMF and 3.0 for IMF, regardless of beam compactness. It should be noted that the minimum \( L_h/d \) ratio of 4.5 (where \( L_h \) is measured from the centerline of the beam’s plastic hinges) typically equates to 6.7 as measured from the face of column to face of column when the typical side plate \{A\} extension (shown as “Side plate \{A\} extension” in Figure 11.6) from face of column is used. The 6.7 ratio, which is slightly less than the 7.0 for other SMF moment connections, allows the potential for a deeper beam to be used in a shorter bay than other SMF moment connections. The field-bolted testing program at UCSD (Mashayekh and Uang, 2016; Reynolds and Uang, 2017) showed that the field-bolted connections sustained approximately 2% more story drift so it is reasonable to set the lower bound \( L_h/d \) ratio for the SidePlate® field-bolted moment connection at 4.0 for SMF and 3 for IMF regardless of beam compactness (see Figure C-11.2c for the measured rotation of the field-bolted W40×211 beam and Figure C-11.2d for the measured rotation of the field-bolted W40×397 beam at the hinge location). All moment-connected beams are required to satisfy the width-to-thickness requirements of AISC Seismic Provisions Sections E2 and E3.

Required lateral bracing of the beam follows the AISC Seismic Provisions. However, due to the significant lateral and torsional restraint provided by the side plates \{A\} as observed in past full-scale tests, for calculation purposes, the unbraced length of the beam is taken as the distance between the respective ends of each side plate \{A\} extension (see Figures 11.11 through 11.15 for depictions of the alphabetical designations). As determined by the full-scale tests, no additional lateral bracing is required at or near the plastic beam hinge location.

The protected zone is defined as shown in Figures 11.7 and 11.8 and extends from the end of the side plate \{A\} to one-half the beam depth beyond the plastic hinge location, which is located at one-third the beam depth in the field-welded and one-sixth the beam depth in the field-bolted beyond the end of the side plate \{A\} due to the cover plate \{B\} or angle \{H\} extensions. This definition is based on test observations that indicate yielding typically does not extend past 83\% and 67\% of the depth of the beam from the end of the side plate \{A\} in the field-welded and field-bolted connections, respectively.
Fig. C-11.2. SidePlate® moment frame tests—backbone curves for (a) W40x294 (W1000x438) beam (field-welded); (b) W36x150 (W920x223) beam (field-welded); (c) W40x211 (W1000x314) beam (field-bolted); (d) W40x397 (W1000x591) beam (field-bolted) (measured at the beam hinge location).

2. Column Limitations

SidePlate® moment connections have been tested with W14 (W360), W16 (W410), W30 (W760), W33 (W840) built-up I-sections, W36 (W840), built-up box section of 30x30x2 (750x750x25) and hollow structural sections (HSS) including HSS14x14x7/8 and HSS18x18x3/4. Note, when using built-up box columns, the side plates {A} transfer the loads to the column in the same way as with wide-flange columns. The only difference is that the horizontal shear component at the top and bottom of the side plates {A} now transfer that horizontal shear directly into the faces of the built-up box column using a shop fillet weld, and thus an internal horizontal shear plate {D} or stiffener is not required. This was verified with the execution of
the test with a W40×397 beam and a 30×30×2 built-up box column without internal horizontal shear plates \{D\} or stiffeners (continuity plates). As such, built-up box columns are prequalified as long as they meet all applicable requirements of the AISC Seismic Provisions. There are no internal stiffener plates within the column, and there are no requirements that the columns be filled with concrete for either SMF or IMF applications. Also no CJP welds are required within a zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange, flange and web plates of boxed wide-flange columns in SidePlate® moment connections with a built-up box column. Note: in some blast or other extreme loading applications, there may be advantages to filling the HSS or built-up box columns with concrete to strengthen the column walls. The above statements have also been corroborated with the two tests conducted at UCSD in 2015 utilizing HSS columns.

In 2015, SidePlate Systems, Inc., conducted two tests with HSS columns as part of the testing program for expanding its prequalification to field-bolted connections (Mashayekh and Uang, 2016). The secondary purpose of these tests was the inclusion of HSS columns with a width-to-thickness ratio of up to 21 in SidePlate® moment connections. It was believed that the width-to-thickness ratio of the walls of HSS columns is a function of local buckling of the walls of the HSS shape in addition to the connection itself. Therefore, it was decided to apply a substantial axial load on the columns (40% nominal axial load capacity of the column) to test and relax the width-to-thickness limit for SidePlate® moment connections. The tests performed very well and there was no yielding/buckling on the face of HSS columns. As a result of two full-scale physical tests and numerous numerical studies, it was confirmed that the width-to-thickness limit of HSS columns in SidePlate® moment connections can be increased to 21, provided that the axial load in the column stays below 40% of the nominal axial load capacity of the column, i.e. 0.40$\frac{F_{y}}{A}$ for the tests performed very well and there were no issues regarding the performance of the column. However, it was decided to limit the HSS column to ASTM A1085 per CPRP’s recommendation.

The behavior of SidePlate® moment connections with cruciform columns is similar to uniaxial one- and two-sided moment connection configurations because the ultimate failure mechanism remains in the beam. Successful tests have been conducted on SidePlate® moment connections with cruciform columns using W36 (W920) shapes with rolled or built-up structural tees.

For SMF systems, the column bracing requirements of AISC Seismic Provisions Section E3.4c.1 are satisfied when a lateral brace is located at or near the intersection of the frame beams and the column. Note: Full-scale tests have demonstrated that without any additional lateral bracing that the full-depth side plates \{A\} provide the required indirect lateral bracing of the column flanges through the side plate \{A\}-to-column flange welds and the connection elements that connect the column web to the side plates \{A\}. Therefore, no additional direct lateral bracing of the column flanges is required.

3. **Connection Limitations**

All test specimens have used ASTM A572/A572M Grade 50 plate material. Nonlinear finite element parametric modeling of side plate \{A\} extensions in the range of 0.65$d$ to 1.0$d$ and 0.65$d$ to 1.7$d$, for the field-welded and field-bolted connections, respectively, have demonstrated similar overall connection and beam behavior when compared to the results of full-scale tests.

Because there is a controlled level of plasticity within the design of the two side plates \{A\}, the side plate \{A\} protected zones have been designated based upon test observations of the field-welded and field-bolted connections and indicated in Figures 11.7 and 11.8, respectively. It needs to be noted that a more conservative design methodology is used for the design of the side plates \{A\} of the field-bolted configuration which result in even less yielding in the critical section of the side plates \{A\}. However, it was decided for consistency to assign similar protected zones for both the field-welded and the field-bolted connections.
11.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

See Figures 11.11 through 11.15 for depictions of the alphabetical and numerical designations. The beams and columns selected must satisfy physical geometric compatibility requirements between the beam flange and column flange to allow sufficient lateral space for depositing fillet welds {5} along the longitudinal edges of the beam flanges that connect to the top and bottom cover plates {B}. Equations 11.4-1a/11.4-1aM and 11.4-1b/11.4-1bM assist designers in selecting appropriate final beam and column size combinations prior to the SidePlate® moment connection actually being designed for a specific project. Note: one of the field-bolted connection tests utilized PJP weld for weld {5} which allows for a tighter tolerance in the geometric compatibility checks. The test performed similar to others with fillet welds for weld {5}; thus weld {5} may be deposited as a PJP weld or fillet weld as needed.

Unlike more conventional moment frame designs that typically rely on the deformation of the column panel zone to achieve the required rotational capacity, SidePlate® moment connection technology instead stiffens and strengthens the column panel zone by providing a minimum of three panel zones (the column web plus the two full-depth side plates {A}). This configuration forces the vast majority of plastic deformation to occur through flange local buckling of the beam.

The column web must be capable of resisting the panel zone shear loads transferred from the horizontal shear plates {D} through the pair of shop fillet welds {3}. The strength of the column web is thereby calculated and compared to the ultimate strength of the welds {3} on both sides of the web. To be acceptable, the panel zone shear strength of the column must be greater than the strength of the two welds. This ensures that the limit state will be failure of the welds as opposed to failure of the column web. The two side plates {A} may be used as doubler plates to check the overall panel zone strength. The following calculation and check is built into the SidePlate® moment connection design software:

\[
\frac{R_u}{R_n} < 1.0
\]  

\[R_u = 0.60 F_d t_{cw} \left(1 + \frac{3 b_f t_f}{d_p t_{cw}}\right)\]  

(from Spec. Eq. J10-11)

where

- \(R_u\) = ultimate strength of fillet welds {3} from horizontal shear plates {D} to column web, kips (N)
- \(R_n\) = nominal strength of column web panel zone in accordance with AISC Specification Section J10.6b, kips (N)
- \(b_f\) = width of column flange, in. (mm)
- \(d_c\) = depth of column, in. (mm)
- \(d_p\) = depth of side plate {A}, in. (mm)
- \(t_{cw}\) = thickness of column web, in. (mm)
- \(t_f\) = thickness of column flange, in. (mm)

In determining the SMF column-beam moment ratio to satisfy strong column/weak beam design criteria, the beam-imposed moment, \(M'_{pb}\), is calculated at the column centerline using statics (i.e. accounting for the increase in moment due to shear amplification from the location of the plastic hinge to the center of the column, due to the development of the plastic moment capacity, \(M_{ps}\), of the beam at the plastic hinge location), and then linearly decreased to one-quarter the column depth above and below the extreme top and bottom fibers of the side plates {A}. This location is used for determination of the column strength as the column is unlikely to form a hinge within the panel zone due to the presence and strengthening effects of the two side plates {A}. 

\[AISC 358-16s1\]
1522 This requirement need not apply if any of the exceptions articulated in AISC Seismic
1523 Provisions Section E3.4a are satisfied. The calculation and check are included in the
1524 SidePlate® connection design software.

1525 11.5. CONNECTION WELDING LIMITATIONS

1526 Fillet welds joining the connection plates to the beam and column provided on all of
1527 the SidePlate® test specimens have been made by either the self-shielded flux cored
1528 arc welding process (FCAW-S or FCAW-G) with a few specimens using the
1529 submerged arc welding process (SAW) for certain shop fillet welds. Other than the
1530 original three prototype tests in 1994 and 1995 that used a non-notch-tough weld
1531 electrode, tested electrodes satisfy minimum Charpy V-notch toughness as required
1532 by the 2010 AISC Seismic Provisions. Also, it should be noted that typically the test
1533 specimens were fit and tacked together using an E7018 stick electrode and then
1534 welded with an FCAW process (implying that the intermixing of FCAW and E7018
1535 has been tested and is not of concern). Test specimens that included either a field
1536 complete-joint-penetration groove-welded beam-to-beam splice or field fillet welds
1537 specifically utilized E70T-6 for the horizontal position and E71T-8 for the vertical
1538 position.

1539 11.6. CONNECTION DETAILING

1540 Figures 11.11 through 11.13 show typical one and two-sided moment connection
1541 details used for shop fabrication of the column with fillet welds. Tests have shown
1542 that the horizontal shear plate \{D\} need not be welded to the column flanges for
1543 successful performance of the connection. However, if there are orthogonal forces
1544 being transferred through the connection from collector, chord or cantilever beams,
1545 then fillet welds connecting the horizontal shear plates \{D\} and the column flanges
1546 may be required.

1547 In the field-welded connection, tests have shown that the use of oversized bolt holes
1548 in the side plates \{A\}, located near their free end (see Figure C-11.3), do not affect
1549 the performance of the connection because beam moments and shears are transferred
1550 through fillet welds. Bolts from the side plate \{A\} to the vertical shear element are
1551 only required for erection of the full-length beam assembly prior to field welding of
1552 the connection and may be removed, at the contractors discretion, after the field fillet
1553 welds have been applied (also implying that if all these erection bolts cannot be
1554 placed it is acceptable, as it relates to the connections performance).

1555 Figure 11.14a and 11.14b show the typical full-length beam detail used for shop
1556 fabrication of the beam with fillet welds. Multiple options can be used to create the
1557 vertical shear element (if needed), such as a combination of angles and plates or
1558 simply bent plates.

1559 Figure 11.15a and 11.15b show the typical full-length beam-to-side plate \{A\} detail
1560 used for field erection of the beam with fillet welds and bolts, respectively. In the
1561 field-bolted connection, either longitudinal angles \{G\} (rolled or built-up) or
1562 horizontal plates \{T\} that are welded to the side plates \{A\}, may be used to transfer
1563 the load from the beam to the side plates \{A\} (Figure 11.15b).

1564 11.7. DESIGN PROCEDURE

1565 The design procedure for the SidePlate® moment connection system is based on
1566 results from both physical testing and detailed nonlinear finite element modeling. The
1567 procedure uses an ultimate strength design approach to size the plates and welds in
1568 the connection, incorporating strength, plasticity and fracture limits. For welds, an
1569 ultimate strength analysis incorporating the instantaneous center of rotation is used
1570 (as described in the AISC Steel Construction Manual Part 8). For bolts, an ultimate
1571 strength analysis incorporating eccentric bolt group design methodology and
1572 instantaneous center of rotation is used (as described in AISC Specification Section
1573 J2.4b). Overall, the design process is consistent with the expected seismic behavior of
1574 an SMF system: lateral drifts due to seismic loads induce moments and shear forces
1575 in the columns and beams. Where these moments exceed the yield capacity of a
beam, a plastic hinge will form. While the primary yield mechanism is plastic bending in the beam, in the field-welded connection, a balanced design approach allows for secondary plastic bending to occur within the side plates \{A\} (hence the reasoning for the protected zones on the side plates \{A\} for this option). In the field-bolted connection more conservative side plate \{A\} design methodology has been developed so secondary plastic hinging within the side plates \{A\} does not occur (hence, the protected zones on the side plates \{A\} in this option are not required).

Ultimately, the location of the hinge in the beam directly affects the amplification of load (i.e., moment and shear from both seismic and gravity) that is resisted by the components of the connection, the column panel zone and the column (as shown in Figure C-11.3). The capacity of each connection component can then be designed to resist its respective load demands induced by the seismic drift (including any increases due to shear amplification as measured from the beams plastic hinge location).

For the SidePlate\textsuperscript{®} moment connection, all of the connection details, including the sizing of connection plates, angles, fillet welds and bolts, are designed and provided by engineers at SidePlate Systems, Inc. The design of these details is based upon basic engineering principles, plastic capacities validated by full-scale testing, and nonlinear finite element analysis. A description of the design methods is presented in Step 7. The initial design procedure for the engineer of record in designing a project with SidePlate\textsuperscript{®} moment connections largely involves:

- Sizing the frame’s beams and columns, shown in Steps 1 and 2.
- Checking applicable building code requirements and performing a preliminary compliance check with all prequalification limitations, shown in Steps 3 and 4.
- Verifying that the SidePlate\textsuperscript{®} moment connections have been designed with the correct project data as outlined in Step 5 and are compliant with all prequalification limits, including final column-beam relationship limitations as shown in Steps 6, 7 and 8.

**Step 1.** Equations 11.4-1a/11.4-1aM and 11.4-1b/11.4-1bM should be used as a guide in selecting beam and column section combinations during design iterations.

![Fig. C-11.3. Amplification of maximum probable plastic hinge moment, $M_{pr}$, to the column face.](image)

Satisfying these equations minimizes the possibility of incompatible beam and column combinations that cannot be fabricated and erected or that may not ultimately satisfy column-beam moment ratio requirements.

**Step 2.** The SidePlate\textsuperscript{®} moment connection design forces a plastic hinge to form in the beam beyond the extension of the side plates \{A\} from the face of the column (side plate \{A\} extension in Figure 11.6). Because inelastic behavior is forced into
the beam at the hinge, the effective span of the beam is reduced, thus increasing the lateral stiffness and strength of the frame (see Figure C-11.4). This increase in stiffness and strength provided by the two parallel side plates \{A\} should be simulated when creating elastic models of the steel frame. Many commercial structural analysis software programs have a built-in feature for modeling the stiffness and strength of the SidePlate® moment connection.

**Step 5.** Some structural engineers design moment-frame buildings with a lateral-only computer analysis. The results are then superimposed with results from additional lateral and vertical load analysis to check beam and column stresses. Because these additional lateral and vertical loads can affect the design of the SidePlate® moment connection, they must also be submitted with the lateral-only model forces. Such additional lateral and vertical loads include drag and chord forces, factored shear loads at the plastic hinge location due to gravity loads on the moment frame beam itself, loads from gravity beams framing into the face of the side plates \{A\}, and gravity loads from cantilever beams (including vertical loads due to earthquakes) framing into the face of the side plates \{A\}.

There are instances where an in-plane lateral drag or chord axial force needs to transfer through the SidePlate® moment connection, as well as instances where it is necessary to transfer lateral drag or chord axial forces from the orthogonal direction through the SidePlate® moment connection. In such instances, these loads must be submitted in order to properly design the SidePlate® moment connection for these conditions.

**Step 6** of the procedure requires SidePlate Systems, Inc. to review the information received from the structural engineer, including the assumptions used in the generation of final beam and column sizes to ensure compliance with all applicable building code requirements and prequalification limitations contained herein. Upon reaching concurrence with the structural engineer of record that beam and column sizes are acceptable and final, SidePlate Systems, Inc. creates a load matrix of the entire structure with these member sizes, including all submitted applicable loads and forces, and designs and details all of the SidePlate® moment connections for a specific project in accordance with Step 7. Any changes in member sizes, loads or forces need to be coordinated with SidePlate Systems, Inc. as they will typically require this step to be repeated.

![Fig. C-11.4. Increased frame stiffness with reduction in effective span of the beam.](image-url)
The demands on the connection components are a function of the strain-hardened moment capacity of the beam, the gravity loads carried by the beam, and the relative locations of each component and the beam’s plastic hinge. Connection components closer to the column centerline are subjected to increased moment amplification compared to components located closer to the beam’s plastic hinge as illustrated in Figure C-11.3.

Step 7 of the process requires that SidePlate Systems, Inc. design and detail the connection components for the actions and loads determined in Step 6. The procedure uses an ultimate strength design approach to size plates, bolts and welds; incorporating strength, plasticity and fracture limits. For welds, an ultimate strength analysis incorporating the instantaneous center of rotation is used (as described in the AISC Steel Construction Manual Part 8). For bolts, an ultimate strength analysis incorporating eccentric bolt group design methodology and instantaneous center of rotation is used (as described in AISC Specification Section J2.4b). Overall, the design process is consistent with the expected seismic behavior of an SMF system as described previously.

The SidePlate® moment connection components are divided into four distinct design groups:

(a) load transfer out of the beam
(b) load transfer into the side plates \{A\}
(c) design of the side plates \{A\} at the column face
(d) load transfer into the column

The transfer of load out of the beam is achieved through welds \{4\} and \{5\}. The loads are in turn transferred through the vertical shear elements \{E\} and cover plates \{B\} into the side plates \{A\} by either welds \{6\} and \{7\} (field-welded) or bolt group (field-bolted). The load at the column face (gap region) is resisted solely by the side plates \{A\}, which transfers the load directly into the column through weld \{2\} and weld \{1\} in a box or HSS section. In a wide flange column, the load is transferred through weld \{2\} and indirectly through weld \{3\} through the combination of weld \{1\} and the horizontal shear plates \{D\}. At each of the four design locations, the elements are designed for the combination of moment, \(M_{\text{group}}\), and shear, \(V_{\text{w}}\).

Connection Design

Side Plate \{A\}, field-welded. To achieve the balanced design for the connection—the primary yield mechanism developing in the beam outside of the connection with secondary plastic behavior within the side plates \{A\}—the required minimum thickness of the side plate \{A\} is calculated using an effective side plate \{A\} plastic section modulus, \(Z_{\text{eff}}\), generated from actual side plate \{A\} behavior obtained from stress and strain profiles along the depth of the side plate \{A\}, as recorded in test data and nonlinear analysis (see Figure C-11.5). The moment capacity of the plates, \(M_{\text{np}}\), is then calculated using the simplified \(Z_{\text{eff}}\) and an effective plastic stress, \(F_{\text{y,eff}}\) of the plate. Allowing for yielding of the plate as observed in testing and analyses (Figure C-11.6) and comparing to the design demand \(M_{\text{group}}\), calculated at the face of column gives:

\[
\frac{M_{\text{group}}}{M_{\text{np}}} \leq 1.0
\]  

where

\[
M_{\text{np}} = F_{\text{y,eff}} Z_{\text{eff}}
\]

Side Plate \{A\}, field-bolted. The required minimum thickness of the side plate \{A\} is calculated based on the engineering principals of fully yielded section at either column face or at the location of the first bolt as shown in Figures C-11.7a and C-11.7b. The section of the side plate \{A\} at the column face has larger design demand in comparison with that of the net section at the location of the first bolt so the required minimum thickness will be the greater of the two design checks.
To ensure the proper behavior of the side plate \{A\} and to preclude undesirable limit states, such as buckling or rupture of the side plate \{A\}, the ratio of the gap distance between the end of the beam and the face of the column to the side plate \{A\} thickness is kept within a range for all connection designs. The optimum gap-to-thickness ratio has been derived based upon the results of full-scale testing and parametric nonlinear analysis.

Fig. C-11.5. Stress profile along depth of side plate \{A\} at the column face at maximum load cycle.

Fig. C-11.6. Idealized plastic stress distribution for computation of the effective plastic modulus, $Z_{eff}$, of the side plate.
Fig. C-11.7. (a) Side plate \( A \) elevation view and stress diagram at the net section; 
(b) side plate \( A \) elevation view and stress diagram at the column face 
[Configuration A (standard)].

**Cover Plate \( B \).** The thickness of the cover plates \( B \) is determined by calculating 
the resultant shear force demand, \( R_u \), from the beam moment couple as:

\[
R_u = \left( \frac{M_{\text{group}}}{d} \right) \tag{C-11.7-2}
\]

and by calculating the vertical shear loads, resisted through the critical shear plane of 
the cover plates \( B \).

The critical shear plane for the field-welded connection is defined as a section cut 
through the cover plate \( B \) adjacent to the boundary of weld \( \{7\} \), as shown in Figure 
C-11.8a. Hence, the thickness, \( t_{sp} \), of the cover plate \( B \) is:

\[
t_{sp} = \frac{R_u}{2(0.6)F_y L_{\text{crit}}} \tag{C-11.7-3}
\]

where \( L_{\text{crit}} = \text{length of critical shear plane through cover plate } B \) as shown in Figure C-11.8a, in. (mm)

The top cover plate \( B \) in the field-bolted connection (standard configuration) is 
designed based on the block shear check in the critical shear plane which is defined as 
a section cut through the cover plate \( B \) through the bolt holes, as shown in Figure 
C-11.8b.
Vertical Shear Element (VSE). The thickness of the VSE (which may include angles {E} and/or bent plates {C}; see Figures 11.11-11.15) is determined as the thickness required to transfer the vertical shear demand from the beam web into the side plates {A}. The vertical shear force demand, \( V_v \), at this load transfer comes from the combination of the capacities of the cover plates \( \{B\} \) and the VSE. The minimum thickness of the VSE, \( t_{vse} \), to resist the vertical shear force is computed as follows:

\[
V_v = \frac{V_v'}{2(0.6)F_yd_{pl}}
\]  
(C-11.7-4)

where

- \( V_v' \) = calculated vertical shear demand resisted by VSE, kips (N)
- \( d_{pl} \) = depth of vertical shear element, in. (mm)

Horizontal Shear Plate (HSP) \( \{D\} \). The thickness of the HSP \( \{D\} \) (see Figures 11.11-11.15) is determined as the thickness required to transfer the horizontal shear demand from the top (or bottom) of the side plates \( \{A\} \) into the column web. The shear demand on the HSP is calculated as the design load developed through the fillet weld connecting the top (or bottom) edge of the side plates \( \{A\} \) to the HSP (weld \( \{1\} \)). The demand force is determined using an ultimate strength analysis of the weld group at the column (weld \( \{1\} \) and weld \( \{2\} \)) as described in the following section.

\[
t_{hsp} = \frac{V_{hsp}'}{(0.6)F_yd_{pl}}
\]  
(C-11.7-5)

where

- \( V_{hsp}' \) = calculated horizontal shear demand delivered by weld \( \{1\} \) to the HSP, kips (N)
Welds. Welds are categorized into three weld groups and sized using an ultimate strength analysis.

The weld groups are categorized as follows (see Figures 11.11-11.115): fillet welds from the beam flange to the cover plate \{B\}/angles \{H\} (weld \{5\} and weld \{5a\}) and the fillet weld from the beam web to the VSE (weld \{4\}) constitute weld group 1. Fillet welds from the cover plate \{B\} to the side plate \{A\} (weld \{7\}) and fillet welds from the VSE to the side plate \{A\} (weld \{6\}) constitute weld group 2 (only field-welded connection). Fillet welds from the side plate \{A\} to the HSP \{D\} (weld \{1\})), fillet welds from the side plate \{A\} to the column flange tips (weld \{2\}) and fillet welds from the HSP \{D\} to the column web (weld \{3\}) make up weld group 3. Refer to Figure C-11.9.

\[
(i) \quad l_{pl} = \text{effective length of horizontal shear plate } \{D\}, \text{ in. (mm)}
\]

Fig. C-11.9. Location of design weld groups and associated moment demand (M_{Cw}).

The ultimate strength design approach for the welds incorporates an instantaneous center of rotation method as shown in Figure C-11.10 and described in the AISC Steel Construction Manual Part 8.

At each calculation iteration, the nominal shear strength, \(R_n\), of each weld group, for a determined eccentricity, \(e\), is compared to the demand from the amplified moment to the instantaneous center of the group, \(V_{pre}\). The process is continued until equilibrium is achieved. Since the process is iterative, SidePlate Systems, Inc. engineers use a design calculation software to compute the weld sizes required to achieve the moment and shear capacity needed for each weld group to resist the amplified moment and vertical shear demand, \(M_{group}\) and \(V_u\), respectively.

Bolts (field-bolted connection only). The ultimate strength analysis incorporating eccentric bolt group design methodology and instantaneous center of rotation as shown in Figure C-11.11 and described in AISC Specification Section J2.4b is used to design the number of required bolts. An iterative process is required to find the solution. At each calculation iteration, the nominal shear strength, \(R_n\), of the bolt group (comprising horizontal and vertical rows of bolts), for a determined eccentricity, \(e\), is compared to the demand from the amplified moment and shear to the instantaneous center of the group, \(V_{pre}\). The process is continued until equilibrium is achieved.

Step 8 requires that the engineer of record review calculations and drawings supplied by SidePlate Systems, Inc. engineers to ensure that all project-specific moment connection designs have been appropriately completed and that all applicable project-
specific design loads, building code requirements, building geometry, and beam-to-
column combinations have been satisfactorily addressed.

The Connection Prequalification Review Panel (CPRP) has prequalified the
SidePlate® moment connection after reviewing the proprietary connection design
procedure contained in the SidePlate® moment Connection Design Software (Version
16 for welded and Version 17 for bolted), as summarized here. In the event that
SidePlate® moment connection designs use a later software version to accommodate
minor format changes in the software’s user input summary and output summary, the
SidePlate® moment connection designs will be accompanied by a SidePlate® moment
connection validation report that demonstrates that the design dimensions, lengths
and sizes of all plates and welds generated using the CPRP-reviewed connection
design procedure remain unchanged from that obtained using the later version
connection design software. Representative beam sizes to be included in the
validation report are W36×150 (W920×223) and W40×294 (W1000×438) for the
field-welded and W36×150, W40×211 and W40×397 for the field-bolted connection.
CHAPTER 14
SlottedWeb™ (SW) Moment Connection

14.1. GENERAL
The SlottedWeb™ (SW) connection is a proprietary welded steel beam to steel column connection developed through private funding by Seismic Structural Design Associates, Inc. (SSDA). In the SW moment connection, slots in the beam web are made parallel and adjacent to the beam flanges. These slots, which start at the end of the beam and are typically one third to one half the nominal beam depth in length, are terminated at a round stress relief hole. The beam web is welded to the column flange and also to the shear plate to give the web both shear and moment capacity.

Analytical studies by Yu (1959) and finite element analyses (FEA) by Abel and Popov (1968) have shown that the shear distribution at the support of cantilever beams differs drastically from that predicted by classical Bernoulli-Euler beam theory that lead to the popular design concept wherein “the flanges carry the moment and the web carries the shear.” It was shown that in the case of a rigid support (beam web and flanges welded to a rigid column flange), the entire shear is resisted by the flanges. For typical “Flange-Welded, Web-Bolted” connections such as the so-called pre-Northridge connection, however, about 50% of the shear is resisted by the beam flanges. It is this 50% shear component in combination with the tension component that causes severe stress and strain gradients across and through the beam flanges of these connections.

By separating the beam flanges from the web in the region of the connection to the column, essentially all the beam shear is resisted by the beam web and, if the beam web is welded to the column, the web also resists a moment equal to the plastic moment capacity of the web, which is typically 30% of the beam plastic moment. Moreover, the elimination of the beam flange shear results in stress and strain gradients across and through the flanges to be nearly uniform.

Cyclic qualifying tests on the SW connection have been made using the single-cantilever type and bare steel specimens; see test results in Table C-14.1. This pseudo-static test with the loading protocol developed by the FEMA/SAC program (FEMA, 2000) has been adopted in Section K2 of the AISC Seismic Provisions (AISC, 2016a). These tests, along with the FEA of the SW connection, show that the yielding region is concentrated in the separated portion of the beam flanges and in the beam web at the end of the shear plate. Peak strengths of the test specimens are usually achieved at an interstory drift angle of approximately 0.03 and 0.04 rad. Reduction in strength, if any, is gradual and due to the out-of-plane buckling of both the beam flanges and web. Buckling of the flanges and web occurs concurrently but independently, which eliminates the lateral torsional mode of buckling. Review of the SSDA test data indicates that the SW connection, when designed and constructed in accordance with the limits and procedures presented herein, have developed interstory drift angles of a least 0.04 radian under cyclic loading on a consistent basis. Ultimate failure typically occurs at drift angles of 0.05 to 0.07 rad by low cycle fatigue fracture of the flange near the end of the slot or partial fracture of the beam web/shear plate weldment to the column flange (Richard, et al., 2001; Partridge, et al., 2002).

14.2. SYSTEMS
Review of the design rationale and the test results shown in Table C-14.1 indicates that the SW connection meets the prequalification requirements for special moment frames in Section K1 of the AISC Seismic Provisions.

14.3. PREQUALIFICATION LIMITS

1. Beam Limitations
A wide range of beam sizes have been tested by SSDA with the SW connection. The smallest beam tested was a W24×94 (W61×140M). The largest was a W36×393 (W920×585M). The AISC Seismic Provisions permit limited increases in beam weight and depth compared to the maximum sections tested and there is no evidence that modest
deviations from the maximum tested specimen would result in significantly different
performance.

Both beam depth and beam span-to-depth ratios are significant in the inelastic behavior
of beam-to-column connections. For the same induced curvature, deep beams will
experience greater strains than shallower beams. Similarly, beams with shorter span-to-
depth ratios will have a sharper moment gradient across the beam span, resulting in a
reduced length of the beam participating in the plastic hinging and increased strains under
inelastic rotational demands. The beam-to-column assemblies that were tested by SSDA
with the SW connection are given in Table C-14.1, which includes the test interstory drift
ratios.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Beam Column</th>
<th>Interstory Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>17</td>
<td>W33x141</td>
<td>4.2</td>
</tr>
<tr>
<td>18</td>
<td>W14x283</td>
<td>5.1</td>
</tr>
<tr>
<td>19</td>
<td>W27x94</td>
<td>4.3</td>
</tr>
<tr>
<td>20</td>
<td>W14x176</td>
<td>5.0</td>
</tr>
<tr>
<td>21</td>
<td>W36x300</td>
<td>4.5</td>
</tr>
<tr>
<td>22</td>
<td>W14x500</td>
<td>4.4</td>
</tr>
<tr>
<td>23</td>
<td>W24x94</td>
<td>4.1</td>
</tr>
<tr>
<td>24</td>
<td>W30x135</td>
<td>4.1</td>
</tr>
<tr>
<td>25</td>
<td>W36x170</td>
<td>4.0</td>
</tr>
<tr>
<td>26</td>
<td>W30x235</td>
<td>4.0</td>
</tr>
<tr>
<td>1a</td>
<td>W36x256</td>
<td>4.9</td>
</tr>
<tr>
<td>2a</td>
<td>W36x393</td>
<td>5.1</td>
</tr>
<tr>
<td>3a</td>
<td>W14x550 – (Gr. 65)</td>
<td>6.0</td>
</tr>
</tbody>
</table>

2. Column Limitations

All of the SW tests have been performed with the beam flange welded to the column
flange (i.e., strong-axis connections). The column sizes used in the tests ranged from
W14 columns to W30 columns.

The behavior of SW connections with cruciform columns and box columns is expected to
be similar to that of a rolled wide-flange column because the beam flanges frame into the
column flange and the column panel zone is oriented parallel to that of the beam. For
cruciform columns the web of the cut wide-flange column is welded with a CJP groove
weld to the continuous web one foot above and below the depth of the frame girder.
Given these similarities and the lack of evidence suggesting behavior limit states different
from those associated with rolled wide-flange shapes, cruciform and box column depths
are permitted equal to those for rolled wide flange column depths.

14.4. BEAM-COLUMN RELATIONSHIP LIMITATIONS

The column panel zone strengths of the SW test specimens varied over a wide range. This
includes specimens with strong panel zones wherein the yielding of the test specimen
came primarily from the beam only, i.e. the panel zone participation in interstory drift
was of the order of 12% to weak panel zones wherein the yielding of the test specimens
comprised panel zone participation of the order of 50%. The behavior of columns with
very weak panel zones can result in column flange “kinking” at the boundaries of the
panel zone. However, for the SW connection, because the beam web slots provide
flexibility to the beam flanges, the effects of this behavior are minimized.
14.5.    BEAM FLANGE-TO-COLUMN LIMITATIONS

CJP groove welds joining the beam flanges to the column flanges of the SW test connections were made using E70T-6-H16 electrodes with a minimum specified CVN toughness as specified in the AISC Seismic Provisions for demand critical welds. Further, the beam bottom flange backing was removed. The root weld pass was back-gouged out and replaced with new weld passes as required. A reinforcing fillet was then added to the bottom flange weld. At the top flange weld, the backing was fillet welded to the column flange. Weld tabs were removed at both the top and bottom flange welds.

14.6.    BEAM WEB AND SHEAR PLATE CONNECTION LIMITATIONS

In all SW test connections the shear plate was welded directly to the column flange using either a CJP or a PJP weld over the full height of the shear plate. The beam web was welded to the face of the column flange, and the shear plate served as the backing for this weld. Further, an eccentrically loaded weld group consisting of fillet welds was used to join the shear plate to the beam web. These welds were made using E71T-8-H16 electrodes with the minimum CVN toughness specified in the AISC Seismic Provisions. Additionally, the shear plate was joined to the beam web with high strength pretensioned bolts.

14.7.    FABRICATION OF THE BEAM WEB SLOTS

The beam web slots in the SW test specimens were flame cut along the “k-line” of the beam to a termination hole which was either drilled or thermally cut. The narrow slot width over the shear plate is designed to inhibit beam flange buckling near the face of the column (to protect the beam flange-to-column flange weld) and force the major beam flange buckling to occur over the wider part of the slot.

14.8.    DESIGN PROCEDURE

The design rationale for the SW connection is based upon:

(a) The IBC (ICC, 2015) and the AISC Specification (AISC, 2016b) and the principles of plastic design
(b) Results of cyclic qualification tests using beams ranging from W24×94 to W36×393 and columns ranging from W14×176 to W14×550 and W27×307 to W30×235
(c) Inelastic finite element analyses to evaluate the stress and strain distributions and buckling modes

In Step 1 the beam slots are designed to:

(1) Force the beam shear at the connection to be carried predominately by the beam web.
(2) Provide a nearly uniform stress and strain distribution horizontally across and vertically through the beam flanges from the column face to the end of the beam web slot.
(3) Allow plastic beam flange and beam web buckling to occur independently in the region of the beam web slot. This eliminates the lateral-torsional mode of buckling found in beams where the beam web is not slotted.
(4) Ensure plastic beam flange buckling so that the full plastic moment capacity of the beam is developed:

\[
\frac{I_x}{I_y} \leq 0.60 \sqrt{\frac{E}{F_y}} \quad \text{(C-14.8-1)}
\]
In Step 2(a) for SMF systems a maximum nominal height of the shear plate is used that can accommodate the slot and the weld across the top and bottom of the shear plate. The minimum thickness of the shear plate is based upon the moment increase in the connection from the plastic hinge at the end of the shear plate to the face of the column. Observations from the SW tests have shown that a shear plate equal to or greater than two-thirds the beam web thickness should be used to stabilize the beam web and shear plate from out-of-plane bending to protect the web and plate welds at the column flange. To stabilize the beam web at the column flange use a minimum shear plate thickness of 2/3 of the beam web thickness but not less than 3/8 in. (10 mm).

In Step 3 AISC Specification tables may be used to determine the weld size of an eccentrically loaded weld group made from fillet welds for the shear plate based upon the shear plate moment and shear forces as shown in Figure C-14.1.

![Fig. C-14.1. Beam web—shear plate force distribution.](image)

* The centroid of the SW connection plastic moment is located at the end of the shear plate.
* At Section A-A the beam web moment and shear are resisted by the beam web and the shear plate.
* Distribute these forces between the shear plate and the beam web based upon the strength distribution.
* \( V_{plate} = \left( \frac{t_b}{b} \right) \times V_{beam} \), \( M_{plate} = \left( \frac{t_b}{b} \right) \times \left[ \left( \frac{b}{h} \right) \times M_{beam} \right] \).
* Use \( V_{beam} \) and \( M_{beam} \) to design the shear plate to the beam web weld.

In Step 4 the shear plate to column flange weld must exceed the fillet weld strength of the shear plate eccentrically loaded fillet weld group that resists the increase in the connection moment from the plastic hinge at the end of the shear plate to the column flange.

In Step 5(a) the bolts are designed for erection purposes and also to clamp the shear plate to the beam web. The effect of this clamping action minimizes the out of plane buckling of the plate and beam web near the column flange weldment.

In Step 7 a resistance factor of 1.0 is used and a \( C_f \) of 1.0 in accordance with Equation G2-2 based upon the 13 cyclic tests (as shown in Table C-14.1) and finite element analyses.
REFERENCES

CHAPTER 11
SIDEPLATE MOMENT CONNECTION


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CHAPTER 14
SLOTTEDWEB MOMENT CONNECTION


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