Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications

Including Supplement No. 1 and Supplement No. 2

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Approved by the AISC Connection Prequalification Review Panel
and issued by the AISC Board of Directors
PREFACE

(This Preface is not part of ANSI/AISC 358-10, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*, or its supplements, but is included for informational purposes only.)


For ease of use, this printing integrates Supplement No. 1. (ANSI/AISC 358s1-11) and Supplement No. 2 (ANSI/AISC 358s2-14). Supplement No. 1 consists of the material in Chapter 10. Supplement No. 2 consists of minor updates to Chapters 2 and 10 along with the material in Chapter 11. Minor editorial revisions are incorporated in other chapters. Symbols and glossary terms from the supplements are integrated into the Symbols and Glossary section.

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 and its supplements use the following symbols in addition to the terms defined in the *Specification for Structural Steel Buildings* (ANSI/AISC 360-10) and the *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341-10). Some definitions in the list below 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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<td>$A_c$</td>
<td>Contact areas between the continuity plate and the column flanges that have attached beam flanges, in.$^2$ (mm$^2$)</td>
<td>6.7</td>
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<tr>
<td>$A_c$</td>
<td>Area of concrete in the column, in.$^2$ (mm$^2$)</td>
<td>10.8</td>
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<tr>
<td>$A_s$</td>
<td>Area of steel in the column, in.$^2$ (mm$^2$)</td>
<td>10.8</td>
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<td>$A_L$</td>
<td>Perpendicular amplified seismic drag or chord forces transferred through the SidePlate connection, resulting from applicable building code, kips (N)</td>
<td>11.7</td>
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<tr>
<td>$A_{</td>
<td></td>
<td>}$</td>
</tr>
<tr>
<td>$C_{pr}$</td>
<td>Factor to account for peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions</td>
<td>2.4.3</td>
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<tr>
<td>$C_t$</td>
<td>Factor used in Equation 6.10-17</td>
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<td>$F_{EXX}$</td>
<td>Filler metal classification strength, ksi (MPa)</td>
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<td>Factored beam flange force, kips (N)</td>
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<td>Nominal tensile strength of bolt from the AISC Specification, ksi (MPa)</td>
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<td>$F_{nv}$</td>
<td>Nominal shear strength of bolt from the AISC Specification, ksi (MPa)</td>
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<td>Specified minimum tensile strength of beam material, ksi (MPa)</td>
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<td>$F_{uf}$</td>
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<td>Nominal weld design strength per the AISC Specification, ksi (MPa)</td>
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<td>Specified minimum yield stress of the yielding element, ksi (MPa)</td>
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<td>Specified minimum yield stress of column flange material, ksi (MPa)</td>
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<td>Specified minimum yield stress of flange material, ksi (MPa)</td>
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<td>$F_{yp}$</td>
<td>Specified minimum yield stress of end-plate material, ksi (MPa)</td>
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<td>$F_{ys}$</td>
<td>Specified minimum yield stress of stiffener material, ksi (MPa)</td>
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<td>$H_h$</td>
<td>Distance along column height from $\frac{1}{4}$ of the column depth above the top edge of the lower-story side plates to $\frac{1}{4}$ of the column depth below the bottom edge of the upper-story side plates, in. (mm)</td>
<td>11.4</td>
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<td>$H_l$</td>
<td>Height of the story below the node, in. (mm)</td>
<td>10.8</td>
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<tr>
<td>$H_u$</td>
<td>Height of the story above the node, in. (mm)</td>
<td>10.8</td>
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<td>$I_{beam}$</td>
<td>Moment of inertia of the beam in the plane of bending, in.$^4$ (mm$^4$)</td>
<td>Figure 11.16</td>
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<td>$I_{total}$</td>
<td>Approximation of moment of inertia due to beam hinge location and side plate stiffness, in.$^4$ (mm$^4$)</td>
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<td>Length of bracket, in. (mm)</td>
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<td>Distance between plastic hinge locations, in. (mm)</td>
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<td>$L_{st}$</td>
<td>Length of end plate stiffener, in. (mm)</td>
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GLOSSARY

This Standard and its supplements use the following terms in addition to the terms defined in the 2010 AISC Specification for Structural Steel Buildings (ANSI/AISC 360-10) and the 2010 AISC Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341-10). Terms defined in the Glossary are italicized in the Glossary and where they first appear within a section throughout the Standard.

Air carbon arc cutting. Process of cutting steel by the heat from an electric arc applied simultaneously with an air jet.

Backing. Piece of metal or other material, placed at the weld root to facilitate placement of the root pass.

Backgouge. Process of removing by grinding or air carbon arc cutting all or a portion of the root pass of a complete-joint-penetration groove weld, from the reverse side of a joint from which a root was originally placed.

Cascaded weld ends. Method of terminating a weld in which subsequent weld beads are stopped short of the previous bead, producing a cascade effect.

Concrete structural slab. Reinforced concrete slab or concrete fill on steel deck with a total thickness of 3 in. (75 mm) or greater and a concrete compressive strength in excess of 2,000 psi (14 MPa).

Full-length beam erection method. A method of erecting a SidePlate steel frame that employs a full-length beam assembly consisting of the beam with shop-installed cover plates (if required) and vertical shear elements (except for HSS beams) which are fillet-welded near the ends of the beam. In the field the full-length beams are lifted up in between pre-installed side plates and are joined to the plates with fillet welds.

Horizontal shear plate (HSP). Plates that transfer a portion of the moment in the side plates to the web of a wide flange column in a SidePlate moment connection.

Link-beam erection method. A method of erecting a SidePlate steel frame that utilizes column tree assemblies with shop-installed beam stubs which are then connected in the field to a link beam using complete-joint-penetration (CJP) groove welds.

Nonfusible backing. Backing material that will not fuse with the base metals during the welding process.

Plastic hinge location. Location in a column-beam assembly where inelastic energy dissipation is assumed to occur through the development of plastic flexural straining.

Probable maximum moment at the plastic hinge. Expected moment developed at a plastic hinge location along a member, considering the probable (mean) value of the material strength for the specified steel and effects of strain hardening.

Reinforcing fillet. Fillet weld applied to a groove welded T-joint to obtain a contour to reduce stress concentrations associated with joint geometry.

Root. Portion of a multi-pass weld deposited in the first pass of welding.
9.2-xx GLOSSARY

**Thermal cutting.** Group of cutting processes that severs or removes metal by localized melting, burning or vaporizing of the workpiece.

**Vertical shear elements (VSE).** Structural elements that transfer shear from a wide flange beam web to the outboard edge of the side plates in a SidePlate moment connection.

**Weld tab.** Piece of metal affixed to the end of a welded joint to facilitate the initiation and termination of weld passes outside the structural joint.
CHAPTER 1
GENERAL

1.1. SCOPE
This Standard specifies design, detailing, fabrication and quality criteria for connections that are prequalified in accordance with the AISC Seismic Provisions for Structural Steel Buildings (herein referred to as the AISC Seismic Provisions) for use with special moment frames (SMF) and intermediate moment frames (IMF). The connections contained in this Standard are prequalified to meet the requirements in the AISC Seismic Provisions only when designed and constructed in accordance with the requirements of this Standard. Nothing in this Standard shall preclude the use of connection types contained herein outside the indicated limitations, nor the use of other connection types, when satisfactory evidence of qualification in accordance with the AISC Seismic Provisions is presented to the authority having jurisdiction.

1.2. REFERENCES
The following standards form a part of this Standard to the extent that they are referenced and applicable:

American Institute of Steel Construction (AISC)
ANSI/AISC 341-10 Seismic Provisions for Structural Steel Buildings (herein referred to as the AISC Seismic Provisions)
ANSI/AISC 360-10 Specification for Structural Steel Buildings (herein referred to as the AISC Specification)

ASTM International (ASTM)
A354-07a Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners
A370-09 Standard Test Methods and Definitions for Mechanical Testing of Steel Products
A488/A488M-10 Standard Practice for Steel Castings, Welding, Qualifications of Procedures and Personnel
A490-08b Standard Specification for Heat-Treated Steel Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength
A572/A572M-07 Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel
A574-11 Standard Specification for Alloy Steel Socket Head Cap Screws
A781/A781M-11 Standard Specification for Castings, Steel and Alloy, Common Requirements, for General Industrial Use
REFERENCES

A958/A958M-10 Standard Specification for Steel Castings, Carbon and Alloy, with Tensile Requirements, Chemical Requirements Similar to Standard Wrought Grades
B19-10 Standard Specification for Cartridge Brass Sheet, Strip, Plate, Bar, and Disks
B36/B36M Standard Specification for Brass Plate, Sheet, Strip, And Rolled Bar
E186-10 Standard Reference Radiographs for Heavy Walled (2 to 4½ in. (50.8 to 114 mm)) Steel Castings
E446-10 Standard Reference Radiographs for Steel Castings Up to 2 in. (50.8 mm) in Thickness
E709-08 Standard Guide for Magnetic Particle Examination

American Welding Society (AWS)
AWS D1.1/D1.1M-2010 Structural Welding Code—Steel
AWS D1.8/D1.8M-2009 Structural Welding Code—Seismic Supplement

Manufacturers Standardization Society (MSS)

Research Council on Structural Connections (RCSC)
Specification for Structural Joints using High-Strength Bolts, 2009 (herein referred to as the RCSC Specification)

1.3. GENERAL

All design, materials and workmanship shall conform to the requirements of the AISC Seismic Provisions and this Standard. The connections contained in this Standard shall be designed according to the Load and Resistance Factor Design (LRFD) provisions. Connections designed according to this Standard are permitted to be used in structures designed according to the LRFD or Allowable Strength Design (ASD) provisions of the AISC Seismic Provisions.
CHAPTER 2
DESIGN REQUIREMENTS

2.1. SPECIAL AND INTERMEDIATE MOMENT FRAME
CONNECTION TYPES

The connection types listed in Table 2.1 are prequalified for use in connecting beams
to column flanges in special moment frames (SMF) and intermediate moment frames
(IMF) within the limitations specified in this Standard.

TABLE 2.1.
Prequalified Moment Connections

<table>
<thead>
<tr>
<th>Connection Type</th>
<th>Chapter</th>
<th>Systems</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reduced beam section (RBS)</td>
<td>5</td>
<td>SMF, IMF</td>
</tr>
<tr>
<td>Bolted unstiffened extended end plate (BUEEP)</td>
<td>6</td>
<td>SMF, IMF</td>
</tr>
<tr>
<td>Bolted stiffened extended end plate (BSEEP)</td>
<td>6</td>
<td>SMF, IMF</td>
</tr>
<tr>
<td>Bolted flange plate (BFP)</td>
<td>7</td>
<td>SMF, IMF</td>
</tr>
<tr>
<td>Welded unreinforced flange-welded web (WUF-W)</td>
<td>8</td>
<td>SMF, IMF</td>
</tr>
<tr>
<td>Kaiser bolted bracket (KBB)</td>
<td>9</td>
<td>SMF, IMF</td>
</tr>
<tr>
<td>ConXtech ConXL moment connection (ConXL)</td>
<td>10</td>
<td>SMF, IMF</td>
</tr>
<tr>
<td>SidePlate moment connection (SidePlate)</td>
<td>11</td>
<td>SMF, IMF</td>
</tr>
</tbody>
</table>

2.2. CONNECTION STIFFNESS

All connections contained in this Standard shall be considered fully restrained (Type
FR) for the purpose of seismic analysis.

2.3. MEMBERS

The connections contained in this Standard are prequalified in accordance with the
requirements of the AISC Seismic Provisions when used to connect members meeting
the limitations of Sections 2.3.1, or 2.3.2 or 2.3.3, as applicable.

1. Rolled Wide-Flange Members

Rolled wide-flange members shall conform to the cross section profile limitations
applicable to the specific connection in this Standard.

2. Built-up Members

Built-up members having a doubly symmetric, I-shaped cross section shall meet the
following requirements:
(1) Flanges and webs shall have width, depth and thickness profiles similar to rolled wide-flange sections meeting the profile limitations for wide-flange sections applicable to the specific connection in this Standard, and

(2) Webs shall be continuously connected to flanges in accordance with the requirements of Sections 2.3.2a or 2.3.2b, as applicable.

2a. Built-up Beams

The web and flanges shall be connected using complete-joint-penetration (CJP) groove welds with a pair of reinforcing fillet welds within a zone extending from the beam end to a distance not less than one beam depth beyond the plastic hinge location, $S_h$, unless specifically indicated in this Standard. The minimum size of these fillet welds shall be the lesser of $\frac{3}{16}$ in. (8 mm) and the thickness of the beam web.

Exception: This provision shall not apply where individual connection prequalifications specify other requirements.

2b. Built-up Columns

Built-up columns shall conform to the provisions of subsections (1) through (4), as applicable. Built-up columns shall satisfy the requirements of the AISC Specification except as modified in this Section. Transfer of all internal forces and stresses between elements of the built-up column shall be through welds.

(1) I-Shaped Columns

The elements of built-up I-shaped columns shall conform to the requirements of the AISC Seismic Provisions.

Within a zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange, unless specifically indicated in this Standard, the column webs and flanges shall be connected using CJP groove welds with a pair of reinforcing fillet welds. The minimum size of the fillet welds shall be the lesser of $\frac{3}{16}$ in. (8 mm) and the thickness of the column web.

Exception: For SidePlate moment connections, each column flange may be connected to the column web using a pair of continuous fillet welds. The required shear strength of the fillet welds, $\phi R_n$, shall equal the shear developed at the column flange to web connection where the shear force in the column is the smaller of:

(a) The nominal shear strength of the column per AISC Specification Equation G2-1.

(b) The maximum shear force that can be developed in the column when plastic hinge(s) form in the connected beam(s).

(2) Boxed Wide-Flange Columns

The wide-flange shape of a boxed wide-flange column shall conform to the requirements of the AISC Seismic Provisions.

The width-to-thickness ratio, $b/t$, of plates used as flanges shall not exceed $0.6\sqrt{E/F_y}$, where $b$ shall be taken as not less than the clear distance between plates.
The width-to-thickness ratio, \( h/t_w \), of plates used only as webs shall conform to the requirements of the AISC Seismic Provisions.

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 shall be joined by CJP groove welds. Outside this zone, plate elements shall be continuously connected by fillet or groove welds.

(3) **Built-up Box Columns**

The width-to-thickness ratio, \( b/t \), of plates used as flanges shall not exceed

\[ 0.6\sqrt{E/F_y} \]

where \( b \) shall be taken as not less than the clear distance between web plates.

The width-to-thickness ratio, \( h/t_w \), of plates used only as webs shall conform to the requirements of the AISC Seismic Provisions.

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 box columns shall be joined by CJP groove welds. Outside this zone, box column web and flange plates shall be continuously connected by fillet welds or groove welds.

(4) **Flanged Cruciform Columns**

The elements of flanged cruciform columns, whether fabricated from rolled shapes or built up from plates, shall meet the requirements of the AISC Seismic Provisions.

Within a zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange, the web of the tee-shaped sections shall be welded to the web of the continuous I-shaped section with CJP groove welds with a pair of reinforcing fillet welds. The minimum size of fillet welds shall be the lesser of \( \frac{5}{16} \) in. (8 mm) or the thickness of the column web. Continuity plates shall conform to the requirements for wide-flange columns.

**Exception:** For SidePlate moment connections, the web of the tee-shaped section(s) may be welded to the web of the continuous I-shaped section with a pair of continuous fillet welds. The required strength of the fillet welds, \( \phi R_n \), shall equal the shear developed at the column web to tee-shaped section connection where the shear force in the column is the smaller of:

(a) The shear strength of the column section per AISC Specification Equation G2-1.

(b) The maximum shear that can be developed in the column when plastic hinge(s) form in the connected beam(s).

3. **Hollow Structural Sections (HSS)**

The width-to-thickness ratio, \( h/t_w \), of HSS members shall conform to the requirements of the AISC Seismic Provisions and shall conform to additional cross-section profile limitations applicable to the individual connection as specified in the applicable chapter.
2.4. CONNECTION DESIGN PARAMETERS

1. Resistance Factors

Where available strengths are calculated in accordance with the AISC Specification, the resistance factors specified therein shall apply. When available strengths are calculated in accordance with this Standard, the resistance factors $\phi_d$ and $\phi_n$ shall be used as specified in the applicable section of this Standard. The values of $\phi_d$ and $\phi_n$ shall be taken as follows:

(a) For ductile limit states:
   
   $\phi_d = 1.00$

(b) For nonductile limit states:
   
   $\phi_n = 0.90$

2. Plastic Hinge Location

The distance of the plastic hinge from the face of the column, $S_h$, shall be taken in accordance with the requirements for the individual connection as specified herein.

3. Probable Maximum Moment at Plastic Hinge

The probable maximum moment at the plastic hinge shall be:

$$M_{pr} = C_{pr} R_y F_y Z_e$$  \hspace{1cm} (2.4.3-1)

where

- $M_{pr}$ = probable maximum moment at plastic hinge, kip-in. (N-mm)
- $R_y$ = ratio of the expected yield stress to the specified minimum yield stress, $F_y$, as specified in the AISC Seismic Provisions
- $Z_e$ = effective plastic section modulus of the section (or connection) at the location of the plastic hinge, in.$^3$ (mm$^3$)
- $C_{pr}$ = factor to account for the peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions. Unless otherwise specifically indicated in this Standard, the value of $C_{pr}$ shall be:

$$C_{pr} = \frac{F_y + F_u}{2F_y} \leq 1.2$$  \hspace{1cm} (2.4.3-2)

where

- $F_y$ = specified minimum yield stress of the yielding element, ksi (MPa)
- $F_u$ = specified minimum tensile strength of the yielding element, ksi (MPa)
4. **Continuity Plates**

Beam flange continuity plates shall be provided in accordance with the AISC Seismic Provisions.

**Exceptions:**
1. For bolted end-plate connections, continuity plates shall be provided in accordance with Section 6.7.
2. For the Kaiser bolted bracket connection, the provisions of Chapter 9 shall apply. When continuity plates are required by Chapter 9, thickness and detailing shall be in accordance with the AISC Seismic Provisions.
3. For the SidePlate connection, beam flange continuity plates are not required. Horizontal shear plates as defined in Chapter 11 may be required.

2.5. **PANEL ZONES**

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

**Exception:** For the SidePlate moment connection, the contribution of the side plates to the overall panel zone strength shall be considered as described in Section 11.4(2).

2.6. **PROTECTED ZONE**

The protected zone shall be as defined for each prequalified connection. Unless otherwise specifically indicated in this Standard, the protected zone of the beam shall be defined as the area from the face of the column flange to one-half of the beam depth beyond the plastic hinge. The protected zone shall meet the requirements of the AISC Seismic Provisions, except as indicated in this Standard. Bolt holes in beam webs, when detailed in accordance with the individual connection provisions of this Standard, shall be permitted.
CHAPTER 3

WELDING REQUIREMENTS

3.1. FILLER METALS

Filler metals shall conform to the requirements of the AISC Seismic Provisions.

3.2. WELDING PROCEDURES

Welding procedures shall be in accordance with the AISC Seismic Provisions.

3.3. BACKING AT BEAM-TO-COLUMN AND CONTINUITY PLATE-TO-COLUMN JOINTS

1. Steel Backing at Continuity Plates

Steel backing used at continuity plate-to-column welds need not be removed. At column flanges, steel backing left in place shall be attached to the column flange using a continuous 5/16-in. (8-mm) fillet weld on the edge below the CJP groove weld.

When backing is removed, the root pass shall be backgouged to sound weld metal and backwelded with a reinforcing fillet. The reinforcing fillet shall be continuous with a minimum size of 5/16 in. (8 mm).

2. Steel Backing at Beam Bottom Flange

Where steel backing is used with CJP groove welds between the bottom beam flange and the column, the backing shall be removed.

Following the removal of steel backing, the root pass shall be backgouged to sound weld metal and backwelded with a reinforcing fillet. The size of the reinforcing fillet leg adjacent to the column flange shall be a minimum of 5/16 in. (8 mm), and the reinforcing fillet leg adjacent to the beam flange shall be such that the fillet toe is located on the beam flange base metal.

Exception: If the base metal and weld root are ground smooth after removal of the backing, the reinforcing fillet adjacent to the beam flange need not extend to base metal.

3. Steel Backing at Beam Top Flange

Where steel backing is used with CJP groove welds between the top beam flange and the column, and the steel backing is not removed, the steel backing shall be attached to the column by a continuous 5/16-in. (8-mm) fillet weld on the edge below the CJP groove weld.
4. Prohibited Welds at Steel Backing

Backing at beam flange-to-column flange joints shall not be welded to the underside of the beam flange, nor shall tack welds be permitted at this location. If fillet welds or tack welds are placed between the backing and the beam flange in error, they shall be repaired as follows:

(1) The weld shall be removed such that the fillet weld or tack weld no longer attaches the backing to the beam flange.

(2) The surface of the beam flange shall be ground flush and shall be free of defects.

(3) Any gouges or notches shall be repaired. Repair welding shall be done with E7018 SMAW electrodes or other filler metals meeting the requirements of Section 3.1 for demand critical welds. A special welding procedure specification (WPS) is required for this repair. Following welding, the repair weld shall be ground smooth.

5. Nonfusible Backing at Beam Flange-to-Column Joints

Where nonfusible backing is used with CJP groove welds between the beam flanges and the column, the backing shall be removed and the root backgouged to sound weld metal and backwelded with a reinforcing fillet. The size of the reinforcing fillet leg adjacent to the column shall be a minimum of 3/16 in. (8 mm) and the reinforcing fillet leg adjacent to the beam flange shall be such that the fillet toe is located on the beam flange base metal.

Exception: If the base metal and weld root are ground smooth after removal of the backing, the reinforcing fillet adjacent to the beam flange need not extend to base metal.

3.4. DETAILS AND TREATMENT OF WELD TABS

Where used, weld tabs shall be removed to within 1/8 in. (3 mm) of the base metal surface and the end of the weld finished, except at continuity plates where removal to within 1/4 in. (6 mm) of the plate edge shall be permitted. Removal shall be by air carbon arc cutting (CAC-A), grinding, chipping or thermal cutting. The process shall be controlled to minimize errant gouging. The edges where weld tabs have been removed shall be finished to a surface roughness of 500 μ-in. (13 microns) or better. The contour of the weld end shall provide a smooth transition to adjacent surfaces, free of notches, gouges and sharp corners. Weld defects greater than 1/16 in. (1.5 mm) deep shall be excavated and repaired by welding in accordance with an applicable WPS. Other weld defects shall be removed by grinding, faired to a slope not greater than 1:5.

3.5. TACK WELDS

In the protected zone, tack welds attaching backing and weld tabs shall be placed where they will be incorporated into a final weld.
3.6. CONTINUITY PLATES

Along the web, the corner clip shall be detailed so that the clip extends a distance of at least 1 1/2 in. (38 mm) beyond the published $k_{det}$ dimension for the rolled shape. Along the flange, the plate shall be clipped to avoid interference with the fillet radius of the rolled shape and shall be detailed so that the clip does not exceed a distance of 1/2 in. (12 mm) beyond the published $k_1$ dimension. The clip shall be detailed to facilitate suitable weld terminations for both the flange weld and the web weld. When a curved corner clip is used, it shall have a minimum radius of 1/2 in. (12 mm).

At the end of the weld adjacent to the column web/flange juncture, weld tabs for continuity plates shall not be used, except when permitted by the engineer of record. Unless specified to be removed by the engineer of record, weld tabs shall not be removed when used in this location.

Where continuity plate welds are made without weld tabs near the column fillet radius, weld layers shall be permitted to be transitioned at an angle of 0° to 45° measured from the vertical plane. The effective length of the weld shall be defined as that portion of the weld having full size. Nondestructive testing (NDT) shall not be required on the tapered or transition portion of the weld not having full size.

3.7. QUALITY CONTROL AND QUALITY ASSURANCE

Quality control and quality assurance shall be in accordance with the AISC Seismic Provisions.
CHAPTER 4
BOLTING REQUIREMENTS

4.1. FASTENER ASSEMBLIES
Bolts shall be pretensioned high-strength bolts conforming to ASTM A325/A325M, A490/A490M, F1852 or F2280, unless other fasteners are permitted by a specific connection.

4.2. INSTALLATION REQUIREMENTS
Installation requirements shall be in accordance with AISC Seismic Provisions and the RCSC Specification, except as otherwise specifically indicated in this Standard.

4.3. QUALITY CONTROL AND QUALITY ASSURANCE
Quality control and quality assurance shall be in accordance with the AISC Seismic Provisions.
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 is a fully restrained connection of beams (comprising either rolled or built-up wide flange sections or hollow structural sections) to columns (comprising either rolled or built-up wide flange sections, built-up biaxial sections of wide-flange and/or tee section(s), or built-up box column sections) using fillet welds and interconnecting plates to connect the moment-resisting beam to its corresponding column as shown in Figure 11.1.

The connection system is typically constructed exclusively of fillet welds (except for flare bevel groove welds at rounded edges of HSS sections as applicable) for both shop fabrication and field erection. The connection features a physical separation, or gap, between the face of the column flange and the end of the beam. 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. 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. Column horizontal shear plates and beam vertical shear elements (or shear plates as applicable) are attached to the column and beam webs, respectively.

‡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. 208,750; 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.
Figure 11.2 shows the connection geometry and major connection components for uniaxial configurations. Figure 11.3 shows the connection geometry and major connection components for biaxial configurations, capable of connecting up to four beams to a column.

Moment frames that utilize the SidePlate connection system can be constructed using one of three methods. Most commonly, construction is with the SidePlate FRAME® configuration that utilizes the full-length beam erection method, as shown in Figure 11.4(a). This method employs a full-length beam assembly consisting of the beam with shop-installed cover plates (if required) and vertical shear elements (except for HSS beams) which are fillet-welded near the ends of the beam.

Column assemblies are typically delivered to the job site with the horizontal shear plates and side plates shop fillet welded to the column at the proper floor framing locations. Where built-up box columns are used, horizontal shear plates are not required, nor applicable.
Fig. 11.2. 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.
During frame erection, the full-length beams are lifted up in between the side plates that are kept spread apart at the top edge of the side plates with a temporary shop-installed spreader (Figure 11.4a). A few bolts connecting the beam’s vertical shear plates (shear elements as applicable) to adjacent free ends of the side plates are initially inserted to provide temporary shoring of the full-length beam assembly, after which the temporary spreader is removed. The remaining erection bolts are then inserted and all bolts are installed snug tight. These erection bolts also act as a clamp to effectively close any root gap that might have existed between the interior face of the side plates and the longitudinal edges of the top cover plate, while bringing the top face of the wider bottom cover plate into a snug fit with the bottom edges of the side plates. To complete the field assembly, four horizontal fillet welds joining the side plates to the cover plates 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 to the vertical shear elements are deposited in the vertical welding position (Position 3F per AWS D1.1/D1.1M).
Where the full-length beam erection method using the SidePlate FRAME configuration is not used, the original SidePlate configuration may be used. The original SidePlate 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 11.4b and 11.4c). 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 11.4d). As depicted in Figure 11.4, the full-length beam erection method can alternately be configured such that the width of bottom flange cover plate is equal to the width of the top cover plate (i.e., both cover plates fit within the separation of the side plates), in lieu of the bottom cover plate being wider than the distance between side plates. Note that when this option is selected by the engineer, the two bottom fillet welds connecting the cover plates to the side plates will be deposited in the overhead welding position (Position 4F per AWS D1.1/D1.1M).

The SidePlate moment connection is proportioned to develop the probable maximum moment capacity of the connected beam. Beam flexural, axial and shear forces are mainly transferred to the top and bottom rectangular cover plates via four shop horizontal fillet welds that connect the edges of the beam flange tips to the corresponding face of each cover plate (two welds for each beam flange). When the U-shaped cover plates 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 (two welds for each beam flange), as well as four shop horizontal fillet welds that connect the top face of the beam top flange and the bottom face of the bottom beam flange to the corresponding inside edge of each U-shaped cover plate (two welds for each beam flange face). These same forces are then transferred from the cover plates to the side plates via four field horizontal fillet welds that connect the cover plates to the side plates. The side plates 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), across the physical gap to the column via shop fillet welding of the side plates to the column flange tips (a total of four shop fillet welds; two for each column flange), and to the horizontal shear plates (a total of four shop fillet welds; one for each horizontal shear plate). The horizontal shear plates are in turn shop fillet welded to the column web and under certain conditions, also to the inside face of column flanges.

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. The side plates, in particular, are designed with the expectation of developing moment capacity larger than the plastic moment capacity of the side plate, and this results in yielding and strain hardening in the vicinity of the side plate protected zones.
Fig. 11.4. SidePlate construction methods. (a) full-length beam erection method (SidePlate FRAME configuration); (b) link-beam erection method (original SidePlate configuration); (c) link beam-to-beam stub splice detail; and (d) all shop-prefabricated single-story moment frame (no field welding); multi-story frames dependent on transportation capabilities.
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 connection is prequalified for use in planar moment-resisting frames and orthogonal intersecting moment-resisting frames (baxial configurations, capable of connecting up to four beams at a column) as illustrated in Figure 11.3.

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) Beam depths shall be limited to W40 (W1000) for rolled shapes. Depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes.

(3) Beam depths shall be limited as follows for HSS shapes:
   (a) For SMF systems, HSS10 (HSS 254) or smaller.
   (b) For IMF systems, HSS12 (HSS 304.8) or smaller.

(4) Beam weight is limited to 302 lb/ft (449 kg/m).

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

The hinge-to-hinge span of the beam, \(L_h\), is defined as 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, \(d/3\), away from the end of the side plate extension, as shown in Figure 11.5. Thus,

\[
L_h = L - \frac{1}{4}z(d_{col,1} + d_{col,2}) - 2(0.33 + 0.77)d
\]  

(11.3-1)

User Note: The 0.33\(d\) constant represents the distance of the plastic hinge from the end of the side plate extension. The 0.77\(d\) constant 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 1.2.3(b) 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 shall consist of the portion of the beam as shown in Figure 11.6 and Figure 11.7.

2. Column Limitations

Columns shall satisfy the following limitations:

(1) Columns shall be any of the rolled wide-flange shapes, 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.

(2) The beam shall be connected to the side plates that are connected to the flange tips of the column.
(3) 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 24 in. (610 mm).

(4) There is no limit on column weight per foot.

(5) There are no additional requirements for column flange thickness.

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

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

3. Connection Limitations

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, where d is the nominal depth of the beam.

(3) The protected zone in the side plates shall consist of a portion of each side plate that is 6 in. (152 mm) high by a length of the gap distance plus 4 in. (102 mm) long, centered at the gap region along the top and bottom edges of each side plate (Figures 11.6 and 11.7).

11.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

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

(1) Beam flange width and thickness for rolled shapes shall satisfy the following equations for geometric compatibility (see Figure 11.8):

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

\[ b_{bf} + 1.1t_{bf} + 12 \text{ mm} \leq b_{cf} \]  

(S.I.)

where

\[ b_{bf} = \text{width of beam flange, in. (mm)} \]

\[ b_{cf} = \text{width of column flange, in. (mm)} \]

\[ t_{bf} = \text{thickness of beam flange, in. (mm)} \]

(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 by Section J10.6a of the AISC Specification.

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

(a) For SMF systems, the column-beam moment ratio shall conform to the requirements of the AISC Seismic Provisions as follows:

(1) 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 11.9). The expected flexural strength of the beam shall be computed as:

$$\sum M'_{pb} = \sum (1.1R_yF_{yb}Z_b + M_v) \quad (11.4-2)$$

where

- $R_y = \text{ratio of the expected yield stress to the specified minimum yield stress } F_y$ as specified in the AISC Seismic Provisions
- $F_{yb} = \text{specified minimum yield stress of the beam, ksi (MPa)}$
- $Z_b = \text{nominal plastic section modulus of the beam, in.}^3 (\text{mm}^3)$
- $M_v = \text{additional moment due to shear amplification from the center of the plastic hinge to the centerline of the column. } M_v \text{ shall be computed as the quantity } V_hsh; \text{ where } V_h \text{ is the shear at the point of theoretical plastic hinging, computed in accordance with Equation 11.4-3, and } s_h = \text{the distance of the assumed point of plastic hinging to the column centerline, which is equal to half the depth of the column plus the extension of the side plates beyond the face of column plus the distance from the end of the side plates to the plastic hinge, } d/3.$

$$V_h = \frac{2M_{pc}}{L_h} + V_{\text{gravity}} \quad (11.4-3)$$

where

- $L_h = \text{distance between plastic hinge locations, in. (mm)}$
- $V_{\text{gravity}} = \text{beam shear force resulting from } 1.2D + f_1L + 0.2S \text{ (where } f_1 \text{ is the load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N)}$

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

(2) 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 11.9). The nominal flexural strength of the column shall be computed as:

\[ \sum M_{pc}^* = \sum Z_{ec}(F_{yc} - P_{uc}/A_g) \]  

(11.4-4)

where

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

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

(11.4-5)

- \( F_{yc} \) = the minimum specified yield strength of the column at the connection, ksi (MPa)
- \( P_{uc}/A_g \) = 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 the column, in.\(^3\) (mm\(^3\))
- \( H \) = story height, in. (mm)
- \( H_h \) = distance along column height from \( \frac{1}{4} \) of the column depth above the top edge of the lower story side plates to \( \frac{1}{4} \) of the column depth below the bottom edge of the upper story side plates, in. (mm)

---

Fig. 11.9. Force and distance designations for computation of column-beam moment ratios.
(b) For IMF systems, the column-beam moment ratio shall conform to the requirements of the AISC Seismic Provisions.

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 column (see plan views in Figure 11.10, Figure 11.11 and Figure 11.12) 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 11.3).

2. Shop fillet weld (5) that connects the edge of the beam flange to the beam flange cover plate (see Figure 11.13).

3. Shop fillet weld (5a) that connects the outside face of the beam flange to the beam flange U-shaped cover plate (see Figure 11.13).

4. Field fillet weld (7) that connects the beam flange cover plates to the side plates (see Figure 11.14), or connects the HSS flange to the side plates.

11.6. CONNECTION DETAILING

The following designations are used herein to identify plates and welds in the SidePlate connection shown in Figures 11.10 through 11.15:

1. Plates

   {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}.
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}.

2. Welds

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

2. Shop fillet weld connecting inside face of side plate {A} to the tip of the column flange, 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}.

5a. Shop fillet weld connecting outside face of beam flange to cover plate {B} U-shaped slot.

6. Field vertical fillet weld connecting vertical shear element (angle or bent plate) (F) to end of side plate {A}.

7. Field horizontal fillet weld connecting the cover plate {B} to the side plate {A}, or connects HSS flange to side plates.

Figure 11.10 shows the connection detailing for a one-sided moment connection configuration in which one beam frames into a column (A-type). Figure 11.11 shows the connection detailing for a two-sided moment connection configuration in which the beams are identical (B-type). Figure 11.12 shows the connection detailing for a two-sided moment connection configuration in which the beams differ in depth (C-type). Figure 11.13 shows the full-length beam assembly shop detail. Figure 11.14 shows the full-length beam-to-side-plate field erection detail. If two beams frame into a column to form a corner, the connection detailing is referred to as a D-type (not shown). The connection detailing for a three-sided and four-sided moment connection configuration is referred to as an E-type and F-series, respectively (not shown). Figure 11.15 shows the link beam-to-beam stub splice detail used with the original SidePlate configuration.
Fig. 11.10. One-sided SidePlate moment connection (A-type), column shop detail.

Fig. 11.11. Two-sided SidePlate moment connection (B-type), column shop detail.
Fig. 11.12. Two-sided SidePlate moment connection (C-type), column shop detail.

Fig. 11.13. Full-length beam shop detail.
Fig. 11.14. Full-length beam-to-side plate field erection detail.

Fig. 11.15. Link-beam erection method detail.
11.7. DESIGN PROCEDURE

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

\[ \sum (F_{yc}Z_{xc}) > 1.7 \sum (F_{yb}Z_{xb}) \]  

where

- \( F_{yb} \) = specified minimum yield stress of the beam, ksi (MPa)
- \( F_{yc} \) = specified minimum yield stress of the 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, \( d \).

**User Note:** This increase in extension of side plate \( \{A\} \) 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 \( d/3 \) from the end of the side plates.

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

**Step 4.** Confirm that the frame beam and column sizes comply with prequalification limitations per Section 11.3.

**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 their computer model to SidePlate Systems, Inc. In addition, the engineer of record shall submit the following additional information, as applicable:
(a) \( V_{\text{gravity}} \) = factored gravity shear in moment frame beam 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)

User Note: The load combination of \( 1.2D + f_1L + 0.2S \) is in conformance with ASCE 7. 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 of the structure.

(b) 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

\[
V_1, V_2 = \text{beam shear force resulting from the load combination of } 1.2D + f_1L + 0.2S \text{ (where } f_1 \text{ is the load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N)}
\]

(c) Factored gravity loads, \( M_{\text{cant}} \) and \( V_{\text{cant}} \), 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

\[
M_{\text{cant}}, V_{\text{cant}} = \text{cantilever beam moment and shear force resulting from code applicable load combinations, kip-in. (N-mm) and kips (N), respectively}
\]
User Note: Code applicable load combinations may need to include the following when looking at cantilever beams: 

\[ 1.2D + f_1L + 0.2S \text{ and } (1.2 + 0.2S_{DS})D + pQ_E + f_1L + 0.2S, \]
which are in conformance with ASCE 7. 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 of the structure.

(d) Perpendicular amplified seismic lateral drag or chord axial forces, \( A_\perp \), transferred through the SidePlate connection.

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

User Note: Where linear-elastic analysis is used to determine perpendicular collector or chord forces used to design the SidePlate connection, such forces shall include applicable load combinations specified by the building code, including considering the amplified seismic load (\( \Omega_0 \)). 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.

(e) In-plane factored lateral drag or chord axial forces, \( A_{\parallel} \), transferred through the SidePlate connection.

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

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 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 11.5, 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:

\[ M_{group} = M_{pr} + V_t x \quad (11.7-2) \]

where

\[ M_{group} \text{ = maximum probable moment demand at any connection element, kip-in. (N-mm)} \]

\[ M_{pr} \text{ = maximum probable moment at the plastic hinge per Section 2.4.3, kip-in. (N-mm), computed as:} \]
\[ M_{pr} = C_{pr}R_yF_yZ_x \]  
\hline
\( C_{pr} \) = 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.

\( R_y \) = ratio of the expected yield stress to the specified minimum yield stress, \( F_y \)

\( F_y \) = specified minimum yield stress of the yielding element, ksi (MPa)

\( Z_x \) = plastic section modulus of beam about the x-axis, in.\(^3\) (mm\(^3\))

\( V_u \) = maximum shear demand from probable maximum moment and factored gravity loads, kips (N), computed as:

\[ V_u = \frac{2M_{pr}}{L_h} + V_{gravity} \]  

\( L_h \) = distance between plastic hinge locations, in. (mm)

\( V_{gravity} = \) gravity beam shear resulting from \( 1.2D + f_1L + 0.2S \) (where \( f_1 \) is the load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N)

\( x \) = distance from plastic hinge location to centroid of connection element, in. (mm)

**Step 7.** SidePlate designs all connection elements per the proprietary connection design procedures contained in SidePlate FRAME Connection Design Software (version 5.2, revised January 2013). 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 is used (as described in AISC Specification Section J2.4b). Refer to the Commentary of this standard 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.
APPENDIX A
CASTING REQUIREMENTS

A1. CAST STEEL GRADE
Cast steel grade shall be in accordance with ASTM A958/A958M Grade SC8620 class 80/50.

A2. QUALITY CONTROL (QC)
1. Inspection and Nondestructive Testing Personnel
Visual inspection and nondestructive testing shall be conducted by the manufacturer in accordance with a written practice by qualified inspectors. The procedure and qualification of inspectors is the responsibility of the manufacturer. Qualification of inspectors shall be in accordance with ASNT-TC-1a or an equivalent standard. The written practice shall include provisions specifically intended to evaluate defects found in cast steel products. Qualification shall demonstrate familiarity with inspection and acceptance criteria used in evaluation of cast steel products.

2. First Article Inspection (FAI) of Castings
The first article is defined as the first production casting made from a permanently mounted and rigged pattern. FAI shall be performed on the first casting produced from each pattern. The first article casting dimensions shall be measured and recorded. FAI includes visual inspection in accordance with Section A2.3, nondestructive testing in accordance with Section A2.4, tensile testing in accordance with Section A2.6, and Charpy V-notch testing in accordance with Section A2.7.

3. Visual Inspection of Castings
Visual inspection of all casting surfaces shall be performed to confirm compliance with ASTM A802/A802M and MSS SP-55 with a surface acceptance Level I.

4. Nondestructive Testing (NDT) of Castings
4a. Procedures
Radiographic testing (RT) shall be performed by quality assurance (QA) according to the procedures prescribed in ASTM E446 and ASTM E186 with an acceptance Level III or better.

Ultrasonic testing (UT) shall be performed by QA according to the procedures prescribed by ASTM A609/A609M Procedure A with an acceptance Level 3, or better.
Magnetic particle testing (MT) shall be performed by QA according to the procedures prescribed by ASTM E709 with an acceptance Level V, or better, in accordance with ASTM A903/A903M.

4b. Required NDT

(1) First Article
   RT and MT shall be performed on the first article casting.

(2) Production Castings
   UT shall be performed on 100% of the castings.
   MT shall be performed on 50% of the castings.

(3) Reduction of Percentage of UT
   The UT rate is permitted to be reduced if approved by the engineer of record and the authority having jurisdiction. The UT rate may be reduced to 25%, provided the number of castings not conforming to Section A2.4a is demonstrated to be 5% or less. A sampling of at least 40 castings shall be made for such reduction evaluation. This reduction is not permitted for castings with weld repairs.

(4) Reduction of Percentage of MT
   The MT rate is permitted to be reduced if approved by the engineer of record and the authority having jurisdiction. The MT rate may be reduced to 10%, provided the number of castings not conforming to Section A2.4a is demonstrated to be 5% or less. A sampling of at least 20 castings shall be made for such reduction evaluation. This reduction is not permitted for castings with weld repairs.

5. Weld Repair Procedures

   Castings with discontinuities that exceed the requirements of Section A2.4a shall be weld repaired. Weld repair of castings shall be performed in accordance with ASTM A488/A488M. The same testing method that discovered the discontinuities shall be repeated on repaired castings to confirm the removal of all discontinuities that exceed the requirements of Section A2.4a.

6. Tensile Requirements

   Tensile tests shall be performed for each heat in accordance with ASTM A370 and ASTM 781/A781M.

7. Charpy V-Notch (CVN) Requirements

   CVN testing shall be performed in accordance with ASTM A370 and ASTM 781/A781M. Three notched specimens shall be tested with the first heat, and with each subsequent 20th ton (18,100 kg) of finished material. The specimens shall have a minimum CVN toughness of 20 ft-lb (27 J) at 70 °F (21 °C).

8. Casting Identification

   The castings shall be clearly marked with the pattern number and a unique serial number for each individual casting providing traceability to heat and production records.
A3. MANUFACTURER DOCUMENTS

1. **Submittal to Patent Holder**

   The following documents shall be submitted to the patent holder, prior to the initiation of production as applicable:

   (1) Material chemical composition report
   (2) First article inspection report

2. **Submittal to Engineer of Record and Authority Having Jurisdiction**

   The following documents shall be submitted to the engineer of record and the authority having jurisdiction, prior to, or with shipment as applicable:

   (1) Production inspection and NDT reports
   (2) Tensile and CVN test reports
   (3) Weld repair reports
   (4) Letter of approval by the patent holder of the manufacturer’s FAI report
APPENDIX B
FORGING REQUIREMENTS

B1. FORGED STEEL GRADE
Raw material shall conform to the requirements of ASTM A572/A572M, Gr. 50 (345). Forging process shall conform to the requirements of ASTM A788 and ASTM A668. Mechanical properties shall conform to the requirements of Table B1.1.

B2. BAR STOCK
Bar stock shall be cut to billets appropriate to the part being forged. All billets shall be marked with heat number.

B3. FORGING TEMPERATURE
Billets shall be forged at a minimum temperature of 2150 °F (1180 °C) and a maximum temperature of 2250 °F (1230 °C).

B4. HEAT TREATMENT
Immediately following impression forging, part the part being forged shall be normalized for one hour at 1650 °F (900 °C) then air cooled.

B5. FINISH
Finished forgings shall have shot blast finish, clean of mill scale.

B6. QUALITY ASSURANCE
One sample of bar stock from each heat shall be cut to a length of 6 in. (152 mm) and forged to a 5 in. by 2 in.-thick bar (127 mm by 50 mm). Samples shall be marked with longitudinal and transverse directions. Chemistry and physical properties per Table B1.1 shall be verified to ASTM A572/A572M Gr. 50 (345) for both longitudinal and transverse direction on each sample.
Magnetic particle testing shall be conducted on the initial 12 pieces from each run to verify tooling and forging procedures. Cracks shall not be permitted. If cracks are found, the tooling or forging procedure shall be modified and an additional 12 initial pieces shall be tested. This process shall be repeated until 12 crack-free samples are obtained prior to production.

**B7. DOCUMENTATION**

Laboratory test data documenting chemistry, strength, elongation, reduction of area, and Charpy requirements for the samples tested in accordance with Section B6 shall be submitted.

Inspection reports documenting satisfactory performance of magnetic particle tests per Section B6 shall be submitted.

Certification of conformance with the requirements of this Appendix shall be submitted to the purchaser.
COMMENTARY
on Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications

Including Supplement No. 1 and Supplement No. 2

This Commentary is not part of ANSI/AISC 358-10, Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications, ANSI/AISC 358s1-11, Supplement No. 1, or ANSI/AISC 358s2-14, Supplement No. 2. It is included for informational purposes only.

INTRODUCTION
The Standard is intended to be complete for normal design usage.

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 1
GENERAL

1.1. SCOPE

Design of special moment frames (SMF) and intermediate moment frames (IMF) in accordance with the AISC Seismic Provisions and applicable building codes includes an implicit expectation that they will experience substantial inelastic deformations when subjected to design-level earthquake ground shaking, generally concentrated at the moment-resisting beam-to-column connections. In the 1994 Northridge earthquake, a number of steel moment frame buildings were found to have experienced brittle fractures that initiated at the welded beam flange-to-column flange joints of moment connections. These brittle fractures were unexpected and were quite different from the anticipated behavior of ductile beam flexural yielding in plastic hinge zones. Where they occurred, these brittle fractures prevented the formation of ductile plastic hinge zones and resulted in frame behavior substantially different from that upon which the design requirements for these systems were based.

Following this discovery, the Federal Emergency Management Agency (FEMA) provided funding to a coalition of universities and professional associations, known as the SAC Joint Venture. Over a period of six years, the SAC Joint Venture, with participation from AISC, AISI, AWS and other industry groups, conducted extensive research into the causes of the damage that had occurred in the Northridge earthquake and effective means of reducing the possibility of such damage in future earthquakes.

Numerous issues were identified in the SAC studies as contributing causes of these brittle fractures. This Standard specifically addresses the following four causes that were identified in the SAC study:

(1) Connection geometries that resulted in large stress concentrations in regions of high triaxiality and limited ability to yield;

(2) Use of weld filler metals with low inherent notch toughness and limited ductility;

(3) High variability in the yield strengths of beams and columns resulting in unexpected zones of weakness in connection assemblies; and

(4) Welding practice and workmanship that fell outside the acceptable parameters of the AWS D1.1/D1.1M Structural Welding Code at that time.

A more complete listing of the causes of damage sustained in the Northridge earthquake may be found in a series of publications (FEMA 350, FEMA 351, FEMA 352, FEMA 353, FEMA 355C, and FEMA 355D) published in 2000 by the SAC Joint Venture that presented recommendations for design and construction of moment resisting frames designed to experience substantial inelastic deformation.
during design ground shaking. These recommendations included changes to material specifications for base metals and welding filler metals, improved quality assurance procedures during construction and the use of connection geometries that had been demonstrated by testing and analysis to be capable of resisting appropriate levels of inelastic deformation without fracture. Most of these recommendations have been incorporated into the AISC Seismic Provisions for Structural Steel Buildings (AISC, 2010) as well as into AWS D1.8/D1.8M Structural Welding Code—Seismic Supplement (AWS, 2009).

Following the SAC Joint Venture recommendations, the AISC Seismic Provisions require that moment connections used in special or intermediate steel moment frames be demonstrated by testing to be capable of providing the necessary ductility. Two means of demonstration are acceptable. One means consists of project-specific testing in which a limited number of full-scale specimens, representing the connections to be used in a structure, are constructed and tested in accordance with a protocol prescribed in Chapter K of the AISC Seismic Provisions. Recognizing that it is costly and time consuming to perform such tests, the AISC Seismic Provisions also provide for prequalification of connections consisting of a rigorous program of testing, analytical evaluation and review by an independent body, the Connection Prequalification Review Panel (CPRP). Connections contained in this Standard have met the criteria for prequalification when applied to framing that complies with the limitations contained herein and when designed and detailed in accordance with this Standard.

1.2. REFERENCES

References for this Standard are listed at the end of the Commentary.

1.3. GENERAL

Connections prequalified under this Standard are intended to withstand inelastic deformation primarily through controlled yielding in specific behavioral modes. To obtain connections that will behave in the indicated manner, proper determination of the strength of the connection in various limit states is necessary. The strength formulations contained in the LRFD method are consistent with this approach.
CHAPTER 2
DESIGN REQUIREMENTS

2.1. SPECIAL AND INTERMEDIATE MOMENT FRAME CONNECTION TYPES

Limitations included in this Standard for various prequalified connections include specification of permissible materials for base metals, mechanical properties for weld filler metals, member shape and profile, and connection geometry, detailing and workmanship. These limitations are based on conditions, demonstrated by testing and analytical evaluation, for which reliable connection behavior can be attained. It is possible that these connections can provide reliable behavior outside these limitations; however, this has not been demonstrated. When any condition of base metal, mechanical properties, weld filler metals, member shape and profile, connection geometry, detailing or workmanship falls outside the limitations specified herein, project-specific qualification testing should be performed to demonstrate the acceptability of connection behavior under these conditions.

Limited testing of connections of wide-flange beams to the webs of I-shaped columns had been conducted prior to the Northridge earthquake by Tsai and Popov (1986, 1988). This testing demonstrated that these “minor-axis” connections were incapable of developing reliable inelastic behavior even at a time when major axis connections were thought capable of developing acceptable behavior. No significant testing of such minor axis connections following the Northridge earthquake has been conducted. Consequently, such connections are not currently prequalified under this Standard.

Similarly, although there has been only limited testing of connections in assemblies subjected to biaxial bending of the column, the judgment of the CPRP was that as long as columns are designed to remain essentially elastic and inelastic behavior is concentrated within the beams, it would be possible to obtain acceptable behavior of beam-column connection assemblies subjected to biaxial loading. Flanged cruciform section columns, built-up box columns, and boxed wide-flange columns are permitted to be used in assemblies subjected to bi-axial loading for those connections types where inelastic behavior is concentrated in the beam, rather than in the column. It should be noted that the strong column—weak beam criteria contained in AISC 341 are valid only for planar frames. When both axes of a column participate in a moment frame, columns should be evaluated for the ability to remain essentially elastic while beams framing to both column axes undergo flexural hinging.

2.3. MEMBERS

2. Built-up Members

The behavior of built-up I-shaped members has been extensively tested in bolted endplate connections and has been demonstrated to be capable of developing the...
necessary inelastic deformations. These members have not generally been tested in other prequalified connections; however, the conditions of inelastic deformation imposed on the built-up shapes in these other connection types are similar to those tested for the bolted end-plate connections.

2b. Built-up Columns

Four built-up column cross section shapes are covered by this Standard. These are illustrated in Figure C-2.1 and include:

(1) I-shaped welded columns that resemble standard rolled wide-flange shapes in cross section shape and profile.

(2) Cruciform W-shape columns, fabricated by splitting a wide-flange section in half and welding the webs on either side of the web of an unsplit wide-flange section at its mid-depth to form a cruciform shape, each outstanding leg of which terminates in a rectangular flange.

(3) Box columns, fabricated by welding four plates together to form a closed box-shaped cross section.

(4) Boxed W-shape columns constructed by adding side plates to the sides of an I-shaped cross section.

The preponderance of connection tests reviewed as the basis for prequalifications contained in this Standard consisted of rolled wide-flange beams connected to the flanges of rolled wide-flange columns. A limited number of tests of connections of wide-flange beams to built-up box section columns were also reviewed.

![Diagram of column shapes](image)

(a) I-shaped section  
(b) Flanged cruciform section  
(c) Box section  
(d) Boxed W-shape section

Figure C-2.1. Column shapes. Plate preparation and welds are not shown.


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The flanged cruciform column and boxed wide-flange columns have not specifically been tested. However, it was the judgment of the CPRP that as long as such column sections met the limitations for I-shaped sections and box-shaped sections, respectively, and connection assemblies are designed to ensure that most inelastic behavior occurred within the beam as opposed to the column, the behavior of assemblies employing these sections would be acceptable. Therefore, prequalification has been extended to these cross sections for connection types where the predominant inelastic behavior is in the beam rather than the column.

2.4. CONNECTION DESIGN PARAMETERS

1. Resistance Factors

A significant factor considered in the formulation of resistance factors is the occurrence of various limit states. Limit states that are considered brittle (non-ductile) and subject to sudden catastrophic failure are typically assigned lower resistance factors than those that exhibit yielding (ductile) failure. Because, for the prequalified connections, design demand is determined based on conservative estimates of the material strength of weak elements of the connection assembly, and materials, workmanship and quality assurance are more rigorously controlled than for other structural elements, resistance factors have been set somewhat higher than those traditionally used. It is believed that these resistance factors, when used in combination with the design, fabrication, erection and quality-assurance requirements contained in the Standard, will provide reliable service in the prequalified connections.

2. Plastic Hinge Location

This Standard specifies the presumed location of the plastic hinge for each prequalified connection type. In reality, inelastic deformation of connection assemblies is generally distributed to some extent throughout the connection assembly. The plastic hinge locations specified herein are based on observed behavior during connection assembly tests and indicate the locations of most anticipated inelastic deformation in connection assemblies conforming to the particular prequalified type.

3. Probable Maximum Moment at Plastic Hinge

The probable plastic moment at the plastic hinge is intended to be a conservative estimate of the maximum moment likely to be developed by the connection under cyclic inelastic response. It includes consideration of likely material overstrength and strain hardening.

4. Continuity Plates

Beam flange continuity plates serve several purposes in moment connections. They help to distribute beam flange forces to the column web, they stiffen the column web to prevent local crippling under the concentrated beam-flange forces and they minimize stress concentrations that can occur in the joint between the beam flange and column due to nonuniform stiffness of the attached elements.
Almost all connection assembly testing has been conducted on specimens that include a significant length (typically one half story height) of column above and below the beam or beams framing into the column. Thus, the condition that typically exists in a structure’s top story, where the column terminates at the level of the beam top flange has not specifically been tested to demonstrate acceptable detailing. A cap plate detail similar to that illustrated in Figure C-2.2 is believed to be capable of providing reliable performance when connection elements do not extend above the beam top flange. In some connections, e.g. extended end plate and Kaiser bolted bracket connections, portions of the connection assembly extend above the column top flange. In such cases, the column should be extended to a sufficient height above the beam flange to accommodate attachment and landing of those connection elements. In such cases, stiffener plates should be placed in the column web, opposite the beam top flange, as is done at intermediate framing levels.

The attachment of continuity plates to column webs is designed to be capable of transmitting the maximum shear forces that can be delivered to the continuity plate. This may be limited by the beam-flange force, the shear strength of the continuity plate itself, or the welded joint between continuity plate and column flange.

The AISC Seismic Provisions require that continuity plates be attached to column flanges with CJP groove welds so the strength of the beam flange can be properly developed into the continuity plate. For single-sided connections in which a moment-connected beam attaches to only one of the column flanges, it is generally not necessary to use CJP groove welds to attach the continuity plate to the column flange.

Figure C-2.2. Example cap plate detail at column top for RBS connection.
that does not have a beam attached. In such cases, acceptable performance can often be obtained by attaching the continuity plate to the column with a pair of minimum-size fillet welds.

When beams are moment connected to the side plates of boxed wide-flange column sections, continuity plates or cap plates should always be provided opposite the beam flanges, as is required for box section columns.
CHAPTER 3
WELDING REQUIREMENTS

3.3. BACKING AT BEAM-TO-COLUMN AND CONTINUITY PLATE-TO-COLUMN JOINTS

At the root of groove welds between beam flanges or continuity plates and column flanges, the inherent lack of a fusion plane between the left-in-place steel backing and the column flange creates a stress concentration and notch effect, even when the weld has uniform and sound fusion at the root. Further, when ultrasonic testing is performed, this left-in-place backing may mask significant flaws that may exist at the weld root. These flaws may create a more severe notch condition than that caused by the backing itself (Chi et al., 1997).

1. Steel Backing at Continuity Plates

The stress and strain level at the groove weld between a continuity plate and column flange is considerably different than that at the beam flange-to-column flange connection; therefore it is not necessary to remove the backing. The addition of the fillet weld beneath the backing makes the inherent notch at the interface an internal notch, rather than an external notch, reducing the notch effect. When backing is removed, the required reinforcing fillet weld reduces the stress concentration at the right-angle intersection of the continuity plate and the column flange.

2. Steel Backing at Beam Bottom Flange

The removal of backing, whether fusible or nonfusible, followed by backgouging to sound weld metal, is required so that potential root defects within the welded joint are detected and eliminated, and the stress concentration at the weld root is eliminated.

The influence of left-in-place steel backing is more severe on the bottom flange, as compared to the top flange, because at the bottom flange, the stress concentration from the backing occurs at the point of maximum applied and secondary tensile stresses in the groove weld, at the weld root, and at the outer fiber of the beam flange.

A reinforcing fillet weld with a 5/16-in. (8-mm) leg on the column flange helps to reduce the stress concentration at the right-angle intersection of the beam flange and column flange, and is placed at the location of maximum stress. The fillet weld’s horizontal leg may need to be larger than 5/16 in. (8 mm) to completely cover the weld root area, eliminating the potential for multiple weld toes at the root that serve as small stress concentrations and potential fracture initiation points. When grinding the weld root and base metal area, previously deposited weld toe regions and their associated fracture initiation sites are removed, and the horizontal leg of the fillet weld need not be extended to base metal.
3. **Steel Backing at Beam Top Flange**

Because of differences in the stress and strain conditions at the top and bottom flange connections, the stress/strain concentration and notch effect created by the backing/column interface at the top flange is at a lower level, compared to that at the bottom flange. Therefore, backing removal is not required. The addition of the reinforcing fillet weld makes the inherent notch at the interface an internal notch, rather than an external notch, further reducing the effect. Because backing removal, backgouging and backwelding would be performed through an access hole beneath the top flange, these operations should be avoided whenever possible.

4. **Prohibited Welds at Steel Backing**

Tack welds for beam flange-to-column connections should be made within the weld groove. Tack welds or fillet welds to the underside of beam at the backing would direct stress into the backing itself, increasing the notch effect at the backing/column flange interface. In addition, the weld toe of the tack weld or fillet weld on the beam flange would act as a stress concentration and a potential fracture initiation site.

Proper removal of these welds is necessary to remove the stress concentration and potential fracture initiation site. Any repair of gouges and notches by filling with weld metal must be made using filler metals with the required notch toughness.

5. **Nonfusible Backing at Beam Flange-to-Column Joints**

After nonfusible backing is removed, backgouging to sound metal removes potential root flaws within the welded joint. A reinforcing fillet weld with a \( \frac{5}{16} \)-in. (8-mm) leg on the column flange helps reduce the stress concentration at the right-angle intersection of the beam flange and column flange.

The fillet weld’s horizontal leg may need to be larger than \( \frac{5}{16} \) in. (8 mm) to completely cover the weld root area, eliminating the potential for small stress concentrations and potential fracture initiation points. When grinding the weld root and base metal area, previously deposited weld toe regions and their associated fracture initiation sites are removed, therefore the horizontal leg of the fillet weld need not be extended to base metal.

3.4. **DETAILS AND TREATMENT OF WELD TABS**

Weld tabs are used to provide a location for initiation and termination of welds outside the final weld location, improving the quality of the final weld. The removal of weld tabs is performed to remove the weld discontinuities and defects that may be present at these start and stop locations. Because weld tabs are located at the ends of welds, any remaining weld defects at the weld-end removal areas may act as external notches and fracture initiation sites and are therefore removed. A smooth transition is needed between base metal and weld to minimize stress concentrations.
3.5. **TACK WELDS**

Tack welds outside weld joints may create unintended load paths and may create stress concentrations that become crack initiation sites when highly strained. By placing tack welds within the joint, the potential for surface notches and hard heat affected zones (HAZs) is minimized. When placed within the joint, the HAZ of a tack weld is tempered by the subsequent passes for the final weld.

Tack welds for beam flange-to-column connections are preferably made in the weld groove. Tack welds of backing to the underside of beam flanges would be unacceptable, and any tack welds between weld backing and beam flanges are to be removed in accordance with Section 3.3.4. Steel backing may be welded to the column under the beam flange, where a reinforcing fillet is typically placed.

When tack welds for the attachment of weld tabs are placed within the weld joint, they become part of the final weld.

3.6. **CONTINUITY PLATES**

The rotary straightening process used by steel rolling mills to straighten rolled sections cold works the webs of these shapes in and near the $k$-area. This cold working can result in an increase in hardness, yield strength, ultimate tensile strength, and yield-to-tensile ratio; and a decrease in notch toughness. In some instances, Charpy V-notch toughness has been recorded to be less than 2 ft-lb at 70 °F [3 J at 20 °C] (Barsom and Korvink, 1998). These changes do not negatively influence the in-service behavior of uncracked shapes. However, the potential for post-fabrication $k$-area base metal cracking exists in highly restrained joints at the weld terminations for column continuity plates, web doublers, and thermal cut coped beams.

When the minimum clip dimensions are used along the member web, the available continuity plate length must be considered in the design and detailing of the welds to the web. For fillet welds, the fillet weld should be held back one to two weld sizes from each clip. For groove welds, weld tabs should not be used in the $k$-area since they could cause base metal fracture from the combination of weld shrinkage, the stress concentration/notch effect at the weld end, and the low notch-toughness web material.

When the maximum clip dimensions are used along the member flange, the width, hence the capacity, of the continuity plate is not reduced substantially. Care must be used in making quality weld terminations near the member radius, as the use of common weld tabs is difficult. If used, their removal in this region may damage the base metal, necessitating difficult repairs. The use of cascaded ends within the weld groove may be used within the dimensional limits stated. Because of the incomplete filling of the groove, the unusual configuration of the weld, and the relatively low level of demand placed upon the weld at this location, nondestructive testing of cascaded weld ends in groove welds at this location are not required.
3.7. QUALITY CONTROL AND QUALITY ASSURANCE

Chapter J of the AISC Seismic Provisions specifies the minimum requirements for a quality assurance plan for the seismic load resisting system. It may be appropriate to supplement the Chapter J provisions with additional requirements for a particular project based on the qualifications of the contractor(s) involved and their demonstrated ability to produce quality work. Contract documents are to define the quality control (QC) and quality assurance (QA) requirements for the project.

QC includes those tasks to be performed by the contractor to ensure that their materials and workmanship meet the project’s quality requirements. Routine welding QC items include personnel control, material control, preheat measurement, monitoring of welding procedures, and visual inspection.

QA includes those tasks to be performed by an agency or firm other than the contractor. QA includes monitoring of the performance of the contractor in implementing the contractor’s QC program, ensuring that designated QC functions are performed properly by the contractor on a routine basis. QA may also include specific inspection tasks that are included in the contractor’s QC plan, and may include nondestructive testing of completed joints.
CHAPTER 4
BOLTING REQUIREMENTS

4.1. FASTENER ASSEMBLIES
ASTM F1852 twist-off type tension-control fastener assemblies are appropriate equivalents for ASTM A325 bolts. ASTM F2280 twist-off type tension control fastener assemblies are appropriate substitutes for ASTM A490. Such assemblies are commonly produced and used, and are addressed by the RCSC Specification (RCSC, 2009).

4.2. INSTALLATION REQUIREMENTS
Section D2 of the AISC Seismic Provisions designates all bolted joints to be pretensioned joints, with the additional requirement that the joint’s faying surfaces meet Class A conditions for slip-critical joints. Some connection types designate the bolted joint to be designed as slip-critical, and others waive the faying surface requirements of the AISC Seismic Provisions.

4.3. QUALITY CONTROL AND QUALITY ASSURANCE
See Commentary Section 3.7.
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 plate and fillet weld groups, 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. The connection is not prequalified when side plates of an unlicensed design and/or manufacturer are used.

SidePlate Systems, Inc., developed, tested and validated SidePlate 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 (which uses a full-length beam assembly fillet-welded in the field to a column assembly to achieve maximum shop fabrication and field erection efficiencies) builds off the research and testing history of its proven predecessor—the original SidePlate steel frame connection system that employs the link-beam erection method (which used column tree assemblies with shop-installed beam stubs, which were then connected in the field to a link beam using CJP welds) and its subsequent refinements. It represents the culmination of an ongoing research and development program (since 1994), which has 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 SidePlate FRAME configuration consists of:

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

(b) Physical validation testing conducted at the Lehigh University Center for Advanced Technology for Large Structural Systems (ATLSS) (Hodgson et al.,...
2010a, 2010b, and 2010c; a total of six cyclic tests) and at University of California, San Diego (UCSD), Charles Lee Powell Laboratories (Minh Huynh and Uang, 2012; a total of three cyclic tests). The purpose of these tests was to confirm global inelastic rotational behavior of 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 third cyclic test at UCSD was a biaxial moment connection test which 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, 2010a). 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. 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 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 connection. The prior physical testing 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 and subsequent monotonic progressive collapse tests: 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 the Kirtland Air Force Base, Albuquerque, NM. This extensive effort has resulted in the ability of SidePlate Systems 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 testings.

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 which is derived from ample performance data from 24
full-scale tests of which two were biaxial, and the judgment of the CPRP. The connection prequalification objectives have been successfully completed; the rudiments of which 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.

(g) 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, although such usage is outside of the scope of this standard.

SidePlate’s unique geometry allows its use in other design applications where in-plane diagonal braces or diagonal dampers are attached to the side plates 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 wide-flange sections, the smallest beam size was a W18×35 (W460×52) and the heaviest a W40×297 (W1000×443). Beam compactness ratios have varied from that of a W18×35 (W460×52) with \(\frac{b_f}{2t_f} = 7.06\) to a W40×294 (W1000×438) with \(\frac{b_f}{2t_f} = 3.11\). For HSS beam members, tests have focused on small members such as the HSS7×3h/2 (HSS177.8×101.6×12.7) having ratios of \(\frac{b}{t} = 5.60\) and \(\frac{h}{t} = 12.1\). As a result of the SidePlate 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 all 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, \(\frac{L_h}{d}\), 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-11.1a). The tested half-span was 14.5 ft (4.42 m), which represents a frame span of 29 ft (8.84 m) and an \(\frac{L_h}{d}\) ratio of 5.5. Assuming that 100% of the frame system’s rotation comes from the beam’s hinge rotation (note: this is 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-11.1b) using an average maximum beam rotation of 8% 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 moment connection system at 4.5 for SMF using the U-shaped cover plate and 3.0 for IMF using the U-shaped cover plate, 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 extension of 0.77\( d \) (shown as dimension A 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.

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 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 extension (see Figures 11.10 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 11.6 and 11.7 and extends from the end of the side plate to one-half the beam depth beyond the plastic hinge location which is located at one-third the beam depth beyond the end of the side plate. This definition is based on test observations that indicate yielding typically does not extend past 83% of the depth of the beam from the end of the side plate.
Fig. C-11.1. SidePlate tests—backbone curves for (a) W40×294 (W1000×438) beam; and (b) W36×150 (W920×223) beam (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-section and a rolled W36 (W920). Although no tested data is available for test specimens using built-up box columns, the side plates 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 face of the built-up box column web using a shop fillet weld, and thus an internal horizontal shear plate or stiffener is not required. 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. However, in some blast applications, there may be advantages to filling the HSS or built-box columns with concrete to strengthen the column walls in such extreme loading applications.

The behavior of SidePlate 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 connections with cruciform columns using W36 (W920) shapes with rolled or built-up structural tees.

For SMF systems, the column bracing requirements of Section E3.4c(1) of the AISC Seismic Provisions are satisfied when a lateral brace is located at or near the intersection of the frame beams and the column. Full-scale tests have demonstrated that the full-depth side plates provide the required indirect lateral bracing of the column flanges through the side plate-to-column flange welds and the connection elements that connect the column web to the side plates. 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. Non-linear finite element parametric modeling of side plate extensions in the range of \( 0.65d \) to \( 1.0d \) has 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, the side plate protected zones have been designated based upon test observations. Protected zones are indicated in Figures 11.6 and 11.7.

11.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

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\}. See Figures 11.10 through 11.15 for depictions of the alphabetical and numerical designations. Equations 11.4.1/11.4.1M assist designers in selecting appropriate final beam and column size combinations prior to the SidePlate connection actually being designed for a specific project.

Unlike more conventional moment frame designs that typically rely on the deformation of the column panel zone to achieve the required rotational capacity, SidePlate 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). 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 following calculation and check is built into the SidePlate connection design software:

\[
\frac{R_u}{R_n} < 1.0 \quad (C-11.4-1)
\]

where

\( R_u \) = ultimate strength of fillet welds \{3\} from horizontal shear plates to column web

\( R_n \) = nominal strength of column web panel zone per AISC Specification Section J10.6b

\[
R_n = 0.60F_{yd}w_{cw}d_{cw} \left(1 + \frac{3b_f \gamma^2}{d_{yd}d_{w}d_{cw}}\right) \quad (C-11.4-2)
\]

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, \( M_{pb} \), 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. 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.

This requirement need not apply if any of the exceptions articulated in AISC Seismic Provisions Section E3.4a are satisfied. The calculation and check is included in the SidePlate connection design software.
11.5. CONNECTION WELDING LIMITATIONS

Fillet welds joining the connection plates to the beam and column provided on all of the SidePlate test specimens have been made by either the self-shielded flux cored arc welding process (FCAW-S or FCAW-G) with a few specimens using the submerged arc welding process (SAW) for certain shop fillet welds. Other than the original three prototype tests in 1994 and 1995 that used a non-notch-tough weld electrode, tested electrodes satisfy minimum Charpy V-notch toughness as required by the 2010 AISC Seismic Provisions. Test specimens that included either a field complete-joint-penetration groove-welded beam-to-beam splice or field fillet welds specifically utilized E70T-6 for the horizontal position and E71T-8 for the vertical position.

11.6. CONNECTION DETAILING

Figures 11.10 through 11.12 show typical one- and two-sided moment connection details used for shop fabrication of the column with fillet welds. Tests have shown that the horizontal shear plate (D) need not be welded to the column flanges for successful performance of the connection. However, if there are orthogonal forces being transferred through the connection from collector, chord or cantilever beams, then fillet welds connecting the horizontal shear plates and the column flanges are required.

Tests have shown that the use of oversized bolt holes in the side plates, located near their free end (see Figure C-11.2), do not affect the performance of the connection because beam moments and shears are transferred through fillet welds. Bolts from the side plate to the vertical shear element are only required for erection of the full-length beam assembly prior to field welding of the connection.

Figure 11.13 shows the typical full-length beam detail used for shop fabrication of the beam with fillet welds. Multiple options can be used to create the vertical shear element, such as a combination of angles and plates or simply bent plates. Figure 11.14 shows the typical full-length beam-to-side plate detail used for field erection of the beam with fillet welds.

11.7. DESIGN PROCEDURE

The design procedure for the SidePlate connection system is based on results from both physical testing and detailed nonlinear finite element modeling. The procedure uses an ultimate strength design approach to size the plates and welds in the connection, incorporating strength, plasticity and fracture limits. For welds, an ultimate strength analysis incorporating the 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: lateral drifts due to seismic loads induce moments and shear forces 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, a balanced design approach allows for secondary plastic bending to occur within the side plates.
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-11.2). The capacity of each connection component can then be designed to resist its respective load demands induced by the seismic drift (including any increase due to shear amplification as measured from the beam plastic hinge).

For the SidePlate moment connection, all of the connection details, including the sizing of connection plates and fillet welds, 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 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 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 11.4-111.4-1M should be used as a guide in selecting beam and column section combinations during design iterations.

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 connection design forces a plastic hinge to form in the beam beyond the extension of the side plates from the face of the column (dimension A in Figure 11.5). 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-11.3). This increase in stiffness and strength provided by the two parallel side plates 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 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, and gravity loads from cantilever beams (including vertical loads due to earthquakes) framing into the face of the side plates.

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 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 designs and details all of the SidePlate moment connections for a specific project per Step 7.

The SidePlate moment connection design procedure is based on the idealized primary behavior of an SMF system: the formation of a plastic hinge in the beam, outside of the connection. Although the primary yield mechanism is development of a plastic hinge in the beam near the end of the side plate, secondary plastic behavior (plastic moment capacity) is developed within the side plates themselves, at the face of the column. Overall, a balanced design is used for the connection components to ensure that the plastic hinge will form at the predetermined location. 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-11.2.
Step 7 of the process requires that SidePlate Systems 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 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 Section J2.4b of the AISC Specification). Overall, the design process is consistent with the expected seismic behavior of an SMF system as described above.

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 \{F\} and cover plates \{B\} into the side plates \{A\} by welds \{6\} and \{7\}, respectively. 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 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, $V_u$.

Connection Design

Side Plate \{A\}. 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—the required minimum thickness of the side plate is calculated using an effective side plate plastic modulus, $Z_{\text{eff}}$, generated from
actual side plate behavior obtained from stress and strain profiles along the depth of the side plate, as recorded in test data and nonlinear analysis (see Figure C-11.4). The moment capacity of the plates, $M_{n,sp}$, is then calculated using the simplified $Z_{eff}$ and an effective plastic stress, $F_{ye}$, of the plate. Allowing for yielding of the plate as observed in testing and analyses (Figure C-11.5) and comparing to the design demand $M_{group}$ calculated at the face of column gives:

$$\frac{M_{group}}{M_{n,sp}} \leq 1.0 \quad (C-11.7-1)$$

where

$$M_{n,sp} = F_{ye}Z_{eff}$$

To ensure the proper behavior of the side plates and to preclude undesirable limit states such as buckling or fracture of the side plates, the ratio of the gap distance between the end of the beam and the face of the column to the side plate thickness is

---

Fig. C-11.4. Stress profile along depth of side plate at the column face at maximum load cycle.

Fig. C-11.5. Idealized plastic stress distribution for computation of the effective plastic modulus, $Z_{eff}$, of the side plate.
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.

**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:

$$R_u = \frac{M_{\text{group}}}{d}$$

(C-11.7-2)

and by calculating the vertical shear loads, $R_{uv}$, resisted through the critical shear plane of the cover plates {B}.

The critical shear plane is defined as a section cut through the cover plate {B} adjacent to the boundary of weld {7}, as shown in Figure C-11.6. Hence, the thickness, $t_{cp}$, of the cover plates is:

$$t_{cp} = \frac{R_u}{2(0.6)F_{ye}L_{\text{crit}}}$$

(C-11.7-3)

where

$L_{\text{crit}} = \text{length of critical shear plane through cover plate as shown in Figure C-11.6, in. (mm)}$

**Vertical Shear Element (VSE) {F}.** The thickness of the VSE {F} (which may include angles and/or bent plates) 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, $V_u$, at this load transfer comes from the combination of the capacities of the cover plates and the VSE. The minimum thickness of the VSE, $t_{vse}$, to resist the vertical shear force is computed as follows:

$$t_{vse} = \frac{V'_u}{2(0.6)F_ey^l_{pl}}$$

(C-11.7-4)

where

$V'_u = \text{calculated vertical shear demand resisted by VSE}$

$d_{pl} = \text{depth of vertical shear element}$

Fig. C-11.6. Critical shear plane of cover plate {B}.
Horizontal Shear Plate (HSP) \( \{D\} \). The thickness of the HSP \( \{D\} \) is determined as the thickness required to transfer the horizontal shear demand from the top (or bottom) of the side plate 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 plate 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.

\[
t_{hsp} = \frac{V''_u}{(0.6) F_d l_{pl}}
\]

where

\( V''_u \) = calculated horizontal shear demand delivered by weld \( \{1\} \) to the HSP, kips (N)

\( l_{pl} \) = effective length of horizontal shear plate, in. (mm)

Welds. Welds are categorized into three weld groups and sized using an ultimate strength analysis.

The weld groups are categorized as follows: fillet welds from the beam flange to the cover plate (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 to the side plate (weld \( \{7\} \)), and fillet welds from the VSE to the side plate (weld \( \{6\} \)) constitute weld group 2. Fillet welds from the side plate to the HSP (weld \( \{1\} \)), fillet welds from the side plate to the column flange tips (weld \( \{2\} \)), and fillet welds from the HSP to the column web (weld \( \{3\} \)) make up weld group 3. Refer to Figure C-11.7.

The ultimate strength design approach for the welds incorporates an instantaneous center of rotation method as shown in Figure C-11.8 and described in Section J2.4b of the AISC Specification.
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, $V_{pre}e$. The process is continued until equilibrium is achieved. Since the process is iterative, SidePlate System engineers use a design spreadsheet 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.

**Step 8** requires that the engineer of record review calculations and drawings supplied by SidePlate 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 has prequalified the SidePlate moment connection after reviewing the proprietary connection design procedure contained in the SidePlate FRAME Connection Design Software (version 5.2, revised January 2013), as summarized here. In the event that SidePlate FRAME connection designs use a later software version to accommodate minor format changes in the software’s user input summary and output summary, the SidePlate connection designs will be accompanied by a SidePlate 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).

![Fig. C-11.8. Instantaneous center of rotation of a sample weld group](Fig. C-J2.12 from AISC 360-10, Specification for Structural Steel Buildings).
APPENDIX A
CASTING REQUIREMENTS

A1. CAST STEEL GRADE

The cast steel grade is selected for its ability to provide ductility similar to that of
casted steel. The material has a specified yield and tensile strength of 50 ksi (354 MPa)
and 80 ksi (566 MPa), respectively. The ASTM specification requires the castings be
produced in conjunction with a heat treatment process that includes normalizing and
stress relieving. It also requires each heat of steel meet strict mechanical properties.
These properties include the specified tensile and yield strengths, as well as elongation
and area reduction limitations.

A2. QUALITY CONTROL (QC)

See Commentary Section 3.7.

2. First Article Inspection (FAI) of Castings

The intent of this section is that at least one casting of each pattern undergo FAI.
When a casting pattern is replaced or when the rigging is modified, FAI is to be
repeated.

3. Visual Inspection of Castings

All casting surfaces shall be free of adhering sand, scales, cracks, hot tears, poros-
ity, cold laps, and chaplets. All cored holes in castings shall be free of flash and
raised surfaces. The ASTM specification includes acceptance criteria for the four
levels of surface inspection. Level I is the most stringent criteria. The Manufacturers
Standardization Society (MSS) specification includes a set of reference comparators
for the visual determination of surface texture, surface roughness and surface dis-
continuities.

4. Nondestructive Testing (NDT) of Castings

These provisions require the use of nondestructive testing to verify the castings do
not contain indications that exceed the specified requirements.

Radiographic testing (RT) is capable of detecting internal discontinuities and is spec-
ified only for the FAI. The ASTM specifications contain referenced radiographs and
five levels of RT acceptance. The lower acceptance levels are more stringent and are
typically required on high-performance aerospace parts such as jet engine turbine
blades or on parts that may leak such as valves or pumps. Level III is considered the
industry standard for structurally critical components.

Ultrasonic testing (UT) is also capable of detecting internal discontinuities and is
specified for production castings. The ASTM specification includes seven levels of
UT acceptance. The lower acceptance levels are more stringent and are typically reserved for machined surfaces subject to contact stresses such as gear teeth. Level 3 is considered the industry standard for structurally critical components.

The areas to be covered by RT or UT are those adjacent to the junctions of:

1. The vertical flange and the horizontal flange
2. The vertical flange and the vertical stiffener
3. The horizontal flange and the vertical stiffener

Magnetic particle testing (MT) is required to detect other forms of discontinuities on or near the surface of the casting. The ASTM specification includes five levels of MT acceptance. The lower levels are more stringent and are typically reserved for pressure vessels. Level V is considered the industry standard for structurally critical components.

Shrinkage is one of the more commonly occurring internal discontinuities and is a result of metal contraction in the mold during solidification. Shrinkage is avoided using reservoirs of molten metal known as risers that compensate for the volumetric contraction during solidification. Numerical modeling of solidification and prediction of shrinkage have been the focus of a number of investigations performed in conjunction with the Steel Founders’ Society of America (SFSA). Niyama et al. (1982) developed a criterion that relates the casting temperature gradient and cooling rate. Based on the Niyama criterion, Hardin et al. (1999) developed a correlation between casting simulation and radiographic testing. Subsequently, Carlson et al. (2003) determined that variation in internal porosity (shrinkage) was related to the pattern and rigging of the casting mold.

Based on these conclusions, the provisions require RT and MT on the first article casting to verify that the pattern and rigging are capable of producing a satisfactory casting. Subsequent castings manufactured with the same pattern and rigging require UT and MT to verify production consistency.

Research performed by Briggs (1967) on the effect of discontinuities found that castings perform satisfactorily under loads in excess of service requirements even with discontinuities of considerable magnitude. Testing demonstrated fatigue and static failures occurred at the location of maximum stress regardless of the presence of discontinuities in other sections.

6. Tensile Requirements

Coupons or keel blocks for tensile testing shall be cast and treated from the same batch of representative castings. Each test specimen shall have complete documentation and traceability. If the specimens fail to meet required specifications, then all the castings they represent shall be rejected.
A3. MANUFACTURER DOCUMENTS

Submittal documents allow a thorough review on the part of the patent holder, engineer of record, the authority having jurisdiction and outside consultants, if required.
APPENDIX B
FORGING REQUIREMENTS

There is no Commentary for this Appendix.
REFERENCES

The following references have been reviewed as a basis for the prequalification of the connections described in this Standard. Although some references are not specifically cited in this Standard, they have been reviewed by the AISC Connection Prequalification Review Panel and are listed here to provide an archival record of the basis for this Standard in accordance with the requirements of Chapter K of the AISC Seismic Provisions.

ALL CONNECTIONS


AISC (2010a), Seismic Provisions for Structural Steel Buildings, AISC/ANSI 341-10, American Institute of Steel Construction, Chicago, IL.

AISC (2010b), Specification for Structural Steel Buildings, AISC/ANSI 360-10, American Institute of Steel Construction, Chicago, IL.

ATC (1992), Guidelines for Cyclic Seismic Testing of Components of Steel Structures, ATC-24, Applied Technology Council, Redwood City, CA.


REFERENCES


CHAPTER 5
REDUCED BEAM SECTION (RBS) MOMENT CONNECTION


REFERENCES


REFERENCES


CHAPTER 6
BOLTED UNSTIFFENED AND STIFFENED EXTENDED END-PLATE MOMENT CONNECTIONS


Abel, M.S. and Murray, T.M. (1992b), “Analytical and Experimental Investigation of the Extended Unstiffened Moment End-Plate Connection with Four Bolts at the Beam Tension Flange,” Research Report CE/VPI-ST-93/08, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA.

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Murray, T.M. (1990), Extended End-Plate Moment Connections, Design Guide No. 4, American Institute of Steel Construction, Chicago, IL.


SAC/BD00/21, submitted to the SAC Joint Venture, Virginia Polytechnic Institute and State University, Blacksburg, VA.


CHAPTER 7
BOLTED FLANGE PLATE (BFP) MOMENT CONNECTION


CHAPTER 8
WELDED UNREINFORCED FLANGE–WELDED WEB (WUF-W) MOMENT CONNECTION


CHAPTER 9
KAISER BOLTED BRACKET (KBB) MOMENT CONNECTION


REFERENCES


CHAPTER 10
CONXTECH CONXL MOMENT CONNECTION


AISC (2010a), Seismic Provisions for Structural Steel Buildings, AISC/ANSI 341-10, American Institute of Steel Construction, Chicago, IL.

AISC (2010b), Specification for Structural Steel Buildings, AISC/ANSI 360-10, American Institute of Steel Construction, Chicago, IL.

Seek, M.W. and Murray, T.M. (2005), “Cyclic Test of 8-Bolt Extended Stiffened Steel Moment End Plate Connection with Concrete Structural Slab,” report submitted to the American Institute of Steel Construction, AISC, Virginia Polytechnic Institute and State University, Blacksburg, VA.


CHAPTER 11
SIDEPLATE MOMENT CONNECTION


Hodgson, I.C., Tahmasebi, E. and Ricles, J.M. (2010c), ATLSS Report No. 10-14, “Cyclic Testing of Beam-to-Column Assembly Connected with SidePlate FRAME Special Moment Frame Connections—Test Specimens 1B and 3,” December, Center for Advanced Technology for Large Structural Systems (ATLSS), Lehigh University, Bethlehem, PA.


REFERENCES


APPENDIX A
CASTING REQUIREMENTS


