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An American National Standard

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

December 13, 2005

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

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SYMBOLS

The Standard uses the following symbols in addition to the terms defined in the 2005 AISC *Specification for Structural Steel Buildings* and the 2005 AISC *Seismic Provisions for Structural Steel Buildings.*

A_c Contact areas between the continuity plate and the column flanges that have attached beam flanges, in.^{2} (mm^2)

C_{pr} Factor to account for peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions, as given in Equation 2.4.3-2

C_t Factor used in Equation 6.9-23

F_{fu} Ultimate flange force, kips (N)

F_{su} Ultimate stiffener force, kips (N)

F_{up} Specified minimum tensile strength of end-plate, ksi (MPa)

F_v Nominal shear strength of bolts, ksi (MPa)

L' Distance between plastic hinges, in. (mm)

L_{st} Length of end plate stiffener, in. (mm)

M_{cf} Column flange flexural strength, kip-in. (N-mm)

M_f Maximum moment expected at face of column, kip-in. (N-mm)

M_{np} Moment without prying action in the bolts, kip-in. (N-mm)

M_{pe} Plastic moment of beam based on expected yield stress, kip-in. (N-mm)

M_{pr} Probable maximum moment at plastic hinge, kip-in. (N-mm)

N Thickness of beam flange plus 2 times the *reinforcing fillet* weld size, in. (mm)

P_t Minimum specified tensile strength of the bolt, kips (N)

R_n Required force for stiffener design, kips (N)

R_y Ratio of expected yield stress to specified minimum yield stress F_y, as specified in the AISC *Seismic Provisions*.

R_{yb} Ratio of expected yield stress to specified minimum yield stress F_y, for a beam

R_{yc} Ratio of expected yield stress to specified minimum yield stress F_y, for a column

S_h Distance from the face of a column to a plastic hinge, in. (mm)

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

V_{RBS} Larger of the two values of shear force at the center of the reduced beam section at each end of a beam, kips (N)

V'_{RBS} Smaller of the two values of shear force at the center of the reduced beam section at each end of a beam, kips (N)

Y_c Column flange yield line mechanism parameter, in. (mm)

Y_p End-plate yield line mechanism parameter, in. (mm)

Z_x Plastic section modulus of a member, in.^{3} (mm^3)

Z_e Effective plastic modulus of a section (or connection) at the location of a plastic hinge, in.^{3} (mm^3) Z_{RBS}

Plastic section modulus at the center of the reduced beam section, in.^{3} (mm^3)

a Horizontal distance between a column flange and the start of an RBS cut, in. (mm)

b Width of compression element as defined in the AISC *Specification*, in. (mm)

b	Length of an RBS cut, in. (mm)
b_{bf}	Width of beam flange, in. (mm)
b_p	Width of plate, in. (mm)
c	Depth of cut at the center of the reduced beam section, in. (mm)
$d_{b\ Req'd}$	Required bolt diameter, in. (mm)
f_1	Load factor determined by the applicable building code for live loads but not less than 0.5
g	Horizontal gage between bolts, in. (mm)
h_1	Distance from the centerline of a compression flange to the tension side inner bolt rows in 4E and 4ES end-plate moment connections, in. (mm)
h_i	Distance from centerline of a compression flange to the centerline of the i^{th} tension bolt row, in. (mm)
h_o	Distance from the centerline of a compression flange to the tension side outer bolt row in 4E and 4ES connections, in. (mm)
h_{st}	Height of stiffener, in. (mm)
k_c	Distance from outer face of a column flange to web toe of fillet (design value) or fillet weld, in. (mm)
n_b	Number of bolts at a compression flange
n_i	Number of inner bolts
n_o	Number of outer bolts
p_b	Pitch between the inner and outer row of bolts in an 8ES end-plate moment connection, in. (mm)
p_f	Vertical distance between beam flange and the nearest row of bolts, in. (mm)
p_{fi}	Distance from the inside of a beam tension flange to the nearest inside bolt row, in. (mm)
p_{fo}	Distance from the outside of a beam tension flange to the nearest outside bolt row, in. (mm)
p_{si}	Distance from the inside face of a column stiffener to the nearest inside bolt row, in. (mm)
p_{so}	Distance from the outside face of column stiffener to the nearest outside bolt row, in. (mm)
s	Distance from the centerline of the most inside or most outside tension bolt row to the edge of a yield line pattern, in. (mm)
t_{bw}	Thickness of beam web, in. (mm)
t_{cw}	Thickness of column web, in. (mm)
t_p	Thickness of plate or panel zone including doubler plates, in. (mm)
w	Uniform beam gravity load, kips per linear ft (N per linear mm)
ϕ_d	Resistance factor for ductile limit states
ϕ_n	Resistance factor for non-ductile limit states

GLOSSARY

The Standard uses the following terms in addition to the terms defined in the 2005 AISC *Specification for Structural Steel Buildings* and the 2005 AISC *Seismic Provisions for Structural Steel Buildings*. Glossary terms are italicized where they appear in the text.

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.

Complete joint penetration (CJP) groove weld. Groove weld in which weld metal extends through the joint thickness.

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 2000 psi (14 MPa).

Non-fusible backing. Backing material that will not fuse with the base metals during the welding process.

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

Probable maximum moment at 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 "tee-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.

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

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, or the use of other connection types, when satisfactory evidence of qualification in accordance with Appendix S of 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:

2005 AISC *Seismic Provisions for Structural Steel Buildings*
2004 AWS *D1.1 Structural Welding Code – Steel* (herein referred to as AWS D1.1)
2004 RCSC *Specification for Structural Joints using ASTM A325 or A490 Bolts* (herein referred to as the RCSC Specification)
2005 AISC *Specification for Structural Steel Buildings* (herein referred to as the AISC *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

Connection Type	Reference Section	Systems
Reduced beam section (RBS)	Chapter 5	SMF, IMF
Bolted unstiffened extended end plate (BUEEP)	Chapter 6	SMF*, IMF
Bolted stiffened extended end plate (BSEEP)	Chapter 6	SMF*, IMF

*Not prequalified for special moment frames (SMFs) with concrete structural slabs in direct contact with the steel

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, as applicable.

2.3.1 Rolled Wide-Flange Members

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

2.3.2 Built-up Members

Built-up members having a doubly symmetric, I-shaped cross-section shall be permitted when:

(1) Flanges and webs 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 are continuously connected to flanges in accordance with the requirements of Sections 2.3.2a or 2.3.2b, as applicable.**

2.3.2a Beams

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 web and flanges shall be connected using *complete joint penetration (CJP) groove welds* with a pair of *reinforcing fillet* welds. The minimum size of these fillet welds shall be the lesser of 5/16 in. (8 mm) or the thickness of the beam web.

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

2.3.2b Columns

Built-up columns shall conform to the provisions of subsections (1) through (4), as applicable. Built-up columns shall satisfy the requirements of AISC *Specification* Section E6 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 Welded 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 fillet welds shall be the lesser of 5/16 in. (8 mm) or the thickness of the column web.

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 $0.6\sqrt{E_s/F_y}$, 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 provisions of Table I–8–1 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 $0.6\sqrt{E_s/F_y}$, 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*.

User Note: For flanged cruciform columns, the provisions of AISC *Specification* Section E6 must be considered.

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 $^5/_{16}$ in. (300 mm) or the thickness of the column web. Continuity plates shall conform to the requirements for wide-flange columns.

2.4 Connection Design Parameters

2.4.1 Load Combinations and 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 ϕ_d and ϕ_n shall be used as specified in the applicable section of this Standard. The values of ϕ_d and ϕ_n shall be taken as:

(a) For ductile limit states:

$$\phi_d = 1.00$$

(b) For non-ductile limit states:

$$\phi_n = 0.90$$

2.4.2 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.

2.4.3 Probable Maximum Moment at Plastic Hinge

The probable maximum moment at the location of the plastic hinge shall be taken as:

$$M_{pr} = C_{pr} R_y F_y Z_e \tag{2.4.3-1}$$

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; see AISC *Seismic Provisions*

Z_e = effective plastic 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:

$$C_{pr} = \frac{F_y + F_u}{2F_y} \leq 1.2 \tag{2.4.3-2}$$

where

F_y = specified minimum yield stress of the type of steel to be used in the yielding element, ksi (MPa)

F_u = specified minimum tensile strength of the type of steel to be used in the yielding element, ksi (MPa)

2.4.4 Beam Flange Continuity Plates

Continuity plates shall be provided.

Exceptions:

(1) For bolted end-plate connections, the provisions of Section 6 shall apply.

(2) When the beam flange connects to the flange of a wide-flange or built-up I-shaped column having a thickness that satisfies Equations 2.4.4-1 and 2.4.4-2, continuity plates need not be provided:

$$t_{cf} \geq 0.4\sqrt{1.8 b_{bf} t_{bf} \frac{F_{yb} R_{yb}}{F_{yc} R_{yc}}} \tag{2.4.4-1}$$

$$t_{cf} \geq \frac{b_{bf}}{6} \tag{2.4.4-2}$$

where

t_{cf} = minimum required thickness of column flange when no continuity plates are provided, in. (mm)

b_{bf} = beam flange width, in. (mm)
t_{bf} = beam flange thickness, in. (mm)
F_{yb} = specified minimum yield stress of the beam flange, ksi (MPa)
F_{yc} = specified minimum yield stress of the column flange, ksi (MPa)
R_{yb} = ratio of the expected yield stress of the beam material to the specified minimum yield stress, per the AISC *Seismic Provisions*
R_{yc} = ratio of the expected yield stress of the column material to the specified minimum yield stress, per the AISC *Seismic Provisions*

(3) When the beam flange connects to the flange of the I-shape in a boxed wide-flange column having a thickness that satisfies Equations 2.4.4-3 and 2.4.4-4, continuity plates need not be provided.

$$t_{cf} \geq 0.4\sqrt{\left[1-\frac{b_{bf}}{b_{cf2}}\left(b_{cf}-\frac{b_{bf}}{4}\right)\right]1.8b_{bf}t_{bf}\frac{F_{yb}R_{yb}}{F_{yc}R_{yc}}} \tag{2.4.4-3}$$

$$t_{cf} \geq \frac{b_f}{12} \tag{2.4.4-4}$$

2.4.4a Continuity Plate Thickness

Where continuity plates are required, the thickness of the plates shall be determined as follows:

(a) For one-sided (exterior) connections, continuity plate thickness shall be at least one-half of the thickness of the beam flange.

(b) For two-sided (interior) connections, the continuity plate thickness shall be at least equal to the thicker of the two beam flanges on either side of the column.

Continuity plates shall also conform to the requirements of Section J10 of the AISC *Specification*.

2.4.4b Continuity Plate to Column Attachment

Continuity plates, if provided, shall be welded to column flanges using CJP groove welds.

Continuity plates shall be welded to column webs using groove welds or fillet welds. The required strength of the sum of the welded joints of the continuity plates to the column web shall be the smallest of the following:

(a) The sum of the design strengths in tension of the contact areas of the continuity plates to the column flanges that have attached beam flanges.

(b) The design strength in shear of the contact area of the plate with the column web.

(c) The design strength in shear of the column panel zone.

(d) The sum of the expected yield strengths of the beam flanges transmitting force to the continuity plates.

2.5 Panel Zones

Panel zones shall conform to the minimum requirements for SMF or IMF, as applicable, in Section 9.3 or Section 10.3 of the AISC *Seismic Provisions*. References to matching of tested connections shall not apply.

2.6 Protected Zone

The protected zone shall be as defined for each prequalified connection. The protected zone shall meet the requirements of Section 7.4 of the AISC *Seismic Provisions*, except as indicated in this Standard. 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 theoretical hinge point. 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 Section 7.3 and Appendix W of the AISC *Seismic Provisions*.

3.2 Welding Procedures

Welding procedures shall be in accordance with Section 7.3 and Appendix W of the AISC *Seismic Provisions*.

3.3 Backing at Beam to Column and Continuity Plate to Column Joints

3.3.1 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, following the removal of *backing*, 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).

3.3.2 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 *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 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 *backing*, the *reinforcing fillet* adjacent to the beam flange need not extend to base metal.

3.3.3 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 *backing* is not removed, the *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.

3.3.4 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.

3.3.5 Non-Fusible Backing at Beam Flange-to-Column Joints

Where *non-fusible backing* is used with CJP groove welds between the beam flanges and the column, the *backing* shall be removed, 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 $^5/_{16}$ 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* is ground smooth after removal of *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 microinches (13 micron) 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. (1.6-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" detail dimension for the rolled shape. Along the flange, the plate shall be clipped to avoid interference with the 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" detail dimension. The clip shall be detailed to facilitate suitable weld terminations for both the flange weld and the web weld. When a curved 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. Non-destructive 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 Appendix Q of the AISC *Seismic Provisions*.

CHAPTER 4

BOLTING REQUIREMENTS

4.1 Fastener Assemblies

Bolts shall be pretensioned high-strength bolts conforming to ASTM A325 or A490. Twist-off type tension control bolt assemblies of equivalent mechanical properties and chemical composition may be substituted for A325 and A490 fastener assemblies.

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 Appendix Q of the AISC *Seismic Provisions*.

CHAPTER 5

REDUCED BEAM SECTION (RBS) MOMENT CONNECTION

5.1 General

In a reduced beam section (RBS) moment connection (Figure 5.1), portions of the beam flanges are selectively trimmed in the region adjacent to the beam-to-column connection. Yielding and hinge formation are intended to occur primarily within the reduced section of the beam.

5.2 Systems

RBS connections are prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limits of these provisions.

5.3 Prequalification Limits

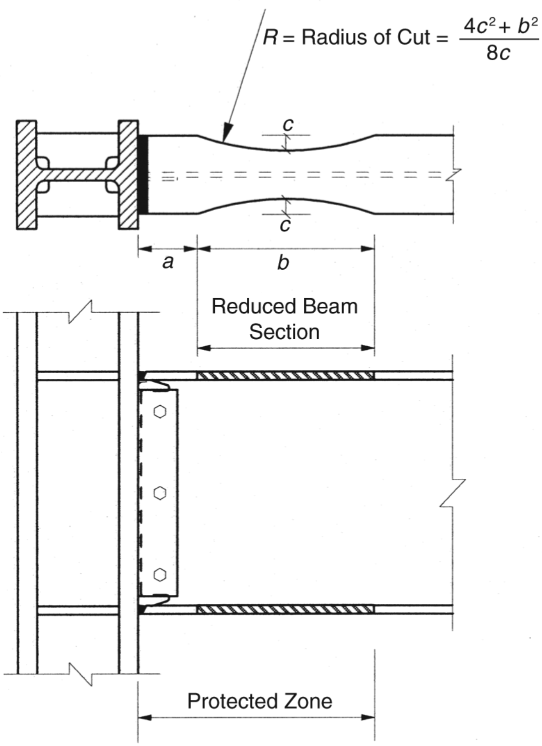


Fig. 5.1. Reduced beam section connection.

5.3.1 Beam Limitations

Beams shall satisfy the following limitations:

(1) Beams shall be rolled wide-flange or built-up I-shaped members conforming to the requirements of Section 2.3.

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

(3) Beam weight is limited to 300 lbs/ft (447 kg/m).

(4) Beam flange thickness is limited to $1^3/_4$ in. (44.5 mm).

(5) The clear span-to-depth ratio of the beam shall be limited as follows:

(a) For SMF systems, 7 or greater.

(b) For IMF systems, 5 or greater.

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

When determining the width-thickness ratio of the flange, the value of b_f shall not be taken as less than the flange width at the ends of the center two-thirds of the reduced section provided that gravity loads do not shift the location of the plastic hinge a significant distance from the center of the reduced beam section.

(7) Lateral bracing of beams shall be provided as follows:

(a) For SMF systems, in conformance with Section 9.8 of the AISC *Seismic Provisions*. Supplemental lateral bracing shall be provided at the reduced section in conformance with Section 9.8 of the AISC *Seismic Provisions* for lateral bracing provided adjacent to the plastic hinges. References to the tested assembly in Section 9.8 of the AISC *Seismic Provisions* do not apply.

When supplemental lateral bracing is provided, attachment of supplemental lateral bracing to the beam shall be located no greater than $d/2$ beyond the end of the reduced beam section farthest from the face of the column, where d is the depth of the beam. No attachment of lateral bracing shall be made to the beam in the region extending from the face of the column to end of the reduced section farthest from the face of the column.

(b) For IMF systems, in conformance with Section 10.8 of the AISC *Seismic Provisions*.

Exception: For both systems, where the beam supports a *concrete structural slab* that is connected between the protected zones with welded shear connectors spaced a maximum of 12 in. (300 mm) on center, supplemental top and bottom flange bracing at the reduced section is not required.

(8) The protected zone consists of the portion of beam between the face of the column and the end of the reduced beam section cut farthest from the face of the column.

5.3.2 Column Limitations

Columns shall satisfy the following limitations:

(1) Columns shall be any of the rolled shapes or built-up sections permitted in Section 2.3.

(2) The beam shall be connected to the flange of the column.

(3) Rolled shape column depth shall be limited to W36 (W920). 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 or depth exceeding 24 in. (610 mm). Boxed wide-flange columns shall not have a width or depth exceeding 24 in. (610 mm) if participating in orthogonal moment frames.

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

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

(6) Width-thickness ratios for the flanges and web of columns shall conform to the limits in Table I–8–1 of the AISC *Seismic Provisions*.

(7) Lateral bracing of columns shall conform to Section 9.7 or 10.7 for SMF or IMF, as applicable, in the AISC *Seismic Provisions*.

5.4 Beam-Column Relationship Limitations

Beam-column connections shall satisfy the following limitations:

(1) Panel zones shall conform to the requirements for Sections 9.3 or 10.3 for SMF or IMF, as applicable, in the AISC *Seismic Provisions*.

(2) Column-beam 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*. The value of ΣM^*_{pb} shall be taken equal to $\Sigma (M_{pr} + M_v)$, where M_{pr} is computed according to Equation 5.8–5, and where M_v is the additional moment due to shear amplification from the center of the reduced beam section to the centerline of the column. M_v can be computed as $V_{RBS} (a + b/2 + d_c/2)$, where V_{RBS} is the shear at the center of the reduced beam section computed per Step 4 of Section 5.8, a and b are the dimensions shown in Figure 5.1, and d_c is the depth of the column.

(b) For IMF systems, the column-beam moment ratio shall conform to the requirements of Section 10.6 of the AISC *Seismic Provisions*.

5.5 Beam Flange to Column Flange Weld Limitations

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

(1) Beam flanges shall be connected to column flanges using *complete joint penetration (CJP) groove welds*. Beam flange welds shall conform to the requirements for demand critical welds in Section 7.3 and Appendix W of the AISC *Seismic Provisions*.

(2) Weld access hole geometry shall conform to the requirements of the AISC *Specification* Section J1.6.

5.6 Beam Web to Column Connection Limitations

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

(1) The required shear strength of the beam web connection shall be determined according to Equation 5.8-9.

(2) Web connection details shall be limited as follows:

(a) For SMF systems, the beam web shall be connected to the column flange using a CJP groove weld extending between weld access holes. The single plate shear connection shall be permitted to be used as *backing* for the CJP groove weld. The thickness of the plate shall be at least $^3/_8$ in. (10 mm). *Weld tabs* are not required at the ends of the CJP groove weld at the beam web. Bolt holes in the beam web for the purpose of erection are permitted.

(b) For IMF systems, the beam web shall be connected to the column flange per as required for SMF systems.

Exception: For IMF, it is permitted to connect the beam web to the column flange using a bolted single plate shear connection. The bolted single plate shear connection shall be designed as a slip-critical connection, with the design slip resistance per bolt determined according to Section J3.8 of the AISC *Specification*. The nominal bearing strength at bolt holes shall not be taken greater than the value given by Equation J3-6a of the AISC *Specification*. The design shear strength of the single plate shear connection shall be determined based on shear yielding of the gross section and on shear fracture of the net section. The plate shall be welded to the column flange with a CJP groove weld, or with fillet welds on both sides of the plate. The minimum size of the fillet weld on each side of the plate shall be 75 percent of the thickness of the plate. Standard-size holes shall be provided in the beam web and in the plate, except that short-slotted holes (with the slot parallel to the beam flanges) may be used in either the beam web or in the plate, but not in both. Bolts are permitted to be pretensioned either before or after welding.

5.7 Fabrication of Flange Cuts

The reduced beam section shall be made using *thermal cutting* to produce a smooth curve. The maximum surface roughness of the thermally cut surface shall be 500 microinches (13 microns) in accordance with ANSI B46.1, as measured using AWS C4.1–77 Sample 4 or similar visual comparator. All transitions between the reduced beam section and the unmodified beam flange shall be rounded in the direction of the flange length to minimize notch effects due to abrupt transitions. Corners between the reduced section surface and the top and bottom of the flanges shall be ground to remove sharp edges, but a minimum chamfer or radius is not required.

Thermal cutting tolerances shall be plus or minus 1/4 in. (6 mm) from the theoretical cut line. The beam effective flange width at any section shall have a tolerance of plus or minus 3/8 in. (10 mm).

Gouges and notches that occur in the thermally cut RBS surface may be repaired by grinding if not more than 1/4 in. (6 mm) deep. The gouged or notched area shall be faired by grinding so that a smooth transition exists, and the total length of the area ground for the transition shall be no less than five times the depth of the removed gouge on each side of the gouge. If a sharp notch exists, the area shall be inspected by MT after grinding to ensure that the entire depth of notch has been removed. Grinding that increases the depth of the RBS cut more than 1/4 in. (6 mm) beyond the specified depth of cut is not permitted.

Gouges and notches that exceed 1/4 in. (6 mm) in depth, but not to exceed 1/2 in. (12 mm) in depth, and those notches and gouges where repair by grinding would increase the effective depth of the RBS cut beyond tolerance, may be repaired by welding. The notch or gouge shall be removed and ground to provide a smooth radius of not less than 1/4 in. in preparation for welding. The repair area shall be preheated to a temperature of 150º F or the value specified in AWS D1.1 Table 3.2, whichever is greater, measured at the location of the weld repair.

Notches and gouges exceeding 1/2 in. (12 mm) in depth shall be repaired only with a method approved by the engineer of record.

5.8 Design Procedure

STEP 1 – Choose trial values for the beam sections, column sections and RBS dimensions a, b, and c (Figure 5.1) subject to the limits:

$$0.5b_{bf} \leq a \leq 0.75b_{bf} \tag{5.8-1}$$

$$0.65d \leq b \leq 0.85d \tag{5.8-2}$$

$$0.1b_{bf} \leq c \leq 0.25b_{bf} \tag{5.8-3}$$

where

b_{bf} = width of beam flange, in. (mm)

d = depth of beam, in. (mm)

a = distance from face of column to start of RBS cut, in. (mm)

b = length of RBS cut, in. (mm)

c = depth of cut at center of the reduced beam section, in. (mm)

Confirm that the beams and columns are adequate for all load combinations specified by the applicable building code, including the reduced section of the beam, and that the design story drift for the frame complies with applicable limits specified by the applicable building code. Calculation of elastic drift shall consider the effect of the reduced beam section. In lieu of specific calculations, effective elastic drifts may be calculated by multiplying elastic drifts based on gross beam sections by 1.1 for flange reductions up to 50 percent of the beam flange width. Linear interpolation may be used for lesser values of beam width reduction.

STEP 2 – Compute the plastic section modulus at the center of the reduced beam section:

$$Z_e = Z_x - 2ct_{bf}(d - t_{bf}) \tag{5.8-4}$$

where

Z_e = plastic section modulus at center of the reduced beam section, in.^{3} (mm^3)

Z_x = plastic section modulus for full beam cross-section, in.^{3} (mm^3)

t_{bf} = thickness of beam flange, in. (mm)

STEP 3 – Compute the probable maximum moment at the center of the reduced beam section:

$$M_{pr} = C_{pr} R_y F_y Z_e \tag{5.8-5}$$

where

M_{pr} = probable maximum moment at center of the reduced beam section, kip-in. (N-mm)

STEP 4 – Compute the shear force at the center of the reduced beam sections at each end of the beam.

The shear force at the center of the reduced beam sections shall be determined by a free body diagram of the portion of the beam between the centers of the reduced beam sections. This calculation shall assume the moment at the center of each reduced beam section is M_{pr} and shall include gravity loads acting on the beam based on the load combination $1.2D + f_1L + 0.2S$

where

f_1 = load factor determined by the applicable building code for live loads, but not less than 0.5

STEP 5 – Compute the probable maximum moment at the face of the column.

The moment at the face of the column shall be computed from a free-body diagram of the segment of the beam between the center of the reduced beam section and the face of the column, as illustrated in Figure 5.2.

Based on this free-body diagram, the moment at the face of the column is computed as follows:

$$M_f = M_{pr} + V_{RBS}\, S_h \tag{5.8-6}$$

where

M_f = probable maximum moment at face of column, kip-in. (N-mm)

V_{RBS} = larger of the two values of shear force at the center of the reduced beam section at each end of the beam, kips (N)

S_h = $a + b/2$, in. (mm)

Equation 5.8-6 neglects the gravity load on the portion of the beam between the center of the reduced beam section and the face of the column. If desired, the gravity load on this small portion of the beam is permitted to be included in the free-body diagram shown in Figure 5.2 and in Equation 5.8-6.

STEP 6 – Compute the plastic moment of the beam based on the expected yield stress:

$$M_{pe} = Z_b R_y F_y \tag{5.8-7}$$

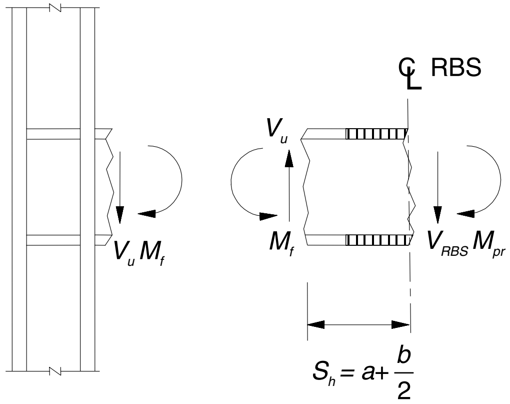


Fig. 5.2. Free-body diagram between center of RBS and face of column.

where

M_{pe} = plastic moment of beam based on expected yield stress, kip-in. (N-mm)

STEP 7 – Check that M_f does not exceed $\phi_d M_{pe}$, as follows:

$$M_f \le \phi_d M_{pe} \tag{5.8-8}$$

If Equation 5.8-8 is not satisfied, increase the value of c and/or decrease the values of a and b, and repeat Steps 2 through 7.

STEP 8 – Determine the required shear strength V_u of beam and beam web-to-column connection from:

$$V_u = \frac{2M_{pr}}{L'} + V_{gravity} \tag{5.8-9}$$

where

V_u = required shear strength of beam and beam web-to-column connection, kips (N)

L' = distance between the centers of the reduced beam sections, in. (mm)

$V_{gravity}$ = beam shear force resulting from $1.2D + f_1L + 0.2S$, kips (N)

f_1 = load factor determined by the applicable building code for live loads, but not less than 0.5

Check design shear strength of beam according to Chapter G of the AISC *Specification*.

STEP 9 – Design the beam web-to-column connection according to Section 5.6.

STEP 10 – Check continuity plate requirements according to Chapter 2.

STEP 11 – Check column panel zone according to Section 5.4.

STEP 12 – Check column-beam moment ratio according to Section 5.4.

CHAPTER 6

BOLTED UNSTIFFENED AND STIFFENED EXTENDED END-PLATE MOMENT CONNECTIONS

6.1 General

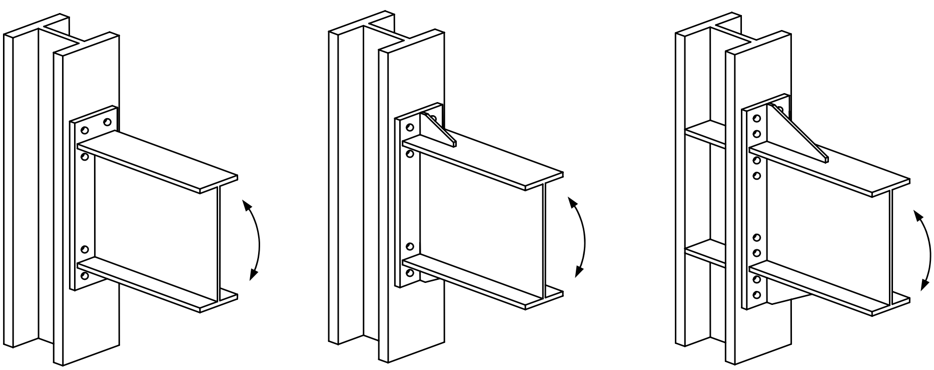
Bolted end-plate connections are made by welding the beam to an end-plate and bolting the end-plate to a column flange. The three end-plate configurations shown in Figure 6.1 are covered in this section and are prequalified under the AISC *Seismic Provisions* within the limitations of this Standard.

The behavior of this type of connection can be controlled by a number of different limit states including flexural yielding of the beam section, flexural yielding of the end-plates, yielding of the column panel zone, tension failure of the end-plate bolts, shear failure of the end-plate bolts, or failure of various welded connections. The intent of the design criteria provided here is to provide sufficient strength in the elements of the connections to ensure that the inelastic deformation of the connection is achieved by beam yielding.

6.2 Systems

Extended end-plate connections are prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems.

Exception: SMF systems in direct contact with *concrete structural slabs* are not prequalified.



(a) Four-Bolt
Unstiffened, 4E

(b) Four-Bolt
Stiffened, 4ES

(c) Eight-Bolt
Stiffened, 8ES

Fig. 6.1. Extended end-plate configurations.

6.3 Prequalification Limits

Table 6.1 is a summary of the range of parameters that have been satisfactorily tested. All connection elements shall be within the ranges shown.

TABLE 6.1.
Parametric Limitations on Prequalification

Parameter	Four-Bolt Unstiffened (4E) Maximum in. (mm)	Four-Bolt Unstiffened (4E) Minimum in. (mm)	Four-Bolt Stiffened (4ES) Maximum in. (mm)	Four-Bolt Stiffened (4ES) Minimum in. (mm)	Eight-Bolt Stiffened (8ES) Maximum in. (mm)	Eight-Bolt Stiffened (8ES) Minimum in. (mm)
t_p	2¼ (57)	½ (13)	1½ (38)	½ (13)	2½ (64)	¾ (19)
b_p	10¾ (273)	7 (178)	10¾ (273)	10¾ (273)	15 (381)	9 (229)
g	6 (152)	4 (102)	6 (152)	3¼ (83)	6 (152)	5 (127)
p_{fi}, p_{fo}	4½ (114)	1½ (38)	5½ (140)	1¾ (44)	2 (51)	1¾ (44)
p_b	—	—	—	—	3¾ (95)	3½ (89)
d	55 (1400)	25 (635)	24 (610)	13¾ (349)	36 (914)	18½ (470)
t_{bf}	¾ (19)	⅜ (10)	¾ (19)	⅜ (10)	1 (25)	19/32 (16)
b_{bf}	9¼ (235)	6 (152)	9 (229)	6 (152)	12¼ (311)	7¾ (197)

where

t_p = thickness of the end-plate, in. (mm)
b_p = width of the end-plate, in. (mm)
g = horizontal distance between bolts, in. (mm)
p_{fi} = vertical distance between beam flange and the nearest inner row of bolts, in. (mm)
p_{fo} = vertical distance between beam flange and the nearest outer row of bolts, in. (mm)
p_b = distance between the inner and outer row of bolts in an eight-bolt connection, in. (mm)
d = depth of the connecting beam, in. (mm)
t_{bf} = thickness of beam flange, in. (mm)
b_{bf} = width of beam flange, in. (mm)

6.4 Beam Limitations

Beams shall satisfy the following limitations:

(1) Beams shall be rolled or welded built-up wide-flange shapes.

At moment-connected ends of welded built-up sections, within at least the depth of beam or 3 times the width of flange, whichever is less, the beam web and flanges shall be connected using either a CJP groove weld or a pair of fillet welds each having a size $^3/_4$ times the beam web thickness but not less than $^1/_4$ in. (6 mm). For the remainder of the beam, the weld size shall not be less than that required to accomplish shear transfer from the web to the flanges.

(2) Beam depth, *d*, is limited to values shown in Table 6.1.

(3) There is no limit on the weight per foot of beams.

(4) Beam flange thickness is limited to the values shown in Table 6.1.

(5) The clear span-to-depth ratio of the beam shall be limited as follows:

(a) For SMF systems, 7 or greater.

(b) For IMF systems, 5 or greater.

(6) Width–thickness ratios for the flanges and web of the beam shall conform to the limits of the AISC *Seismic Provisions*.

(7) Lateral bracing of beams shall be provided as follows:

(a) For SMF systems, in conformance with Section 9.8 of the AISC *Seismic Provisions*.

(b) For IMF systems, in conformance with Section 10.8 of the AISC *Seismic Provisions*.

(8) The protected zone shall be determined as follows:

(a) For unstiffened extended end-plate connections: the portion of beam between the face of the column and a distance equal to the depth of the beam or 3 times the width of flange from the face of the column, whichever is less.

(b) For stiffened extended end-plate connections: the portion of beam between the face of the column and a distance equal to the location of the end of the stiffener plus one-half the depth of the beam or 3 times the width of the beam flange, whichever is less.

6.5 Column Limitations

Columns shall satisfy the following limitations:

(1) The end-plate shall be connected to the flange of the column.

(2) The column depth shall be limited to the beam depth or shallower.

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

(4) There are no additional requirements for flange thickness.

(5) Width–thickness ratios for the flanges and web of the column shall conform to the limits in Table I–8–1 of the AISC *Seismic Provisions*.

6.6 Beam-Column Relationship Limitations

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

(1) Panel zones shall conform to the requirements of Sections 9.3 or 10.3 for SMF or IMF, as applicable, in the AISC *Seismic Provisions*.

(2) The column-beam moment ratio shall conform to the requirements for SMF or IMF, as applicable, in the AISC *Seismic Provisions*.

6.7 Continuity Plates

Continuity plates shall satisfy the following limitations:

(1) The need for continuity plates shall be determined in accordance with Section 6.10.

(2) When provided, continuity plates shall conform to the requirements of Section 6.10.

(3) Continuity plates shall be attached to columns by welds in accordance with Section 2.4.4b and Section 3.6.

Exception: Continuity plates less than or equal to $^3/_8$ in. (10 mm) shall be permitted to be welded to column flanges using double-sided fillet welds. The required strength of the fillet weld shall not be less than F_yA_c, where A_c is defined as the contact areas between the continuity plate and the column flanges that have attached beam flanges and F_y is defined as the specified minimum yield stress of the continuity plate.

6.8 Bolts

Bolts shall conform to the requirements of Section 4.

6.9 Connection Detailing

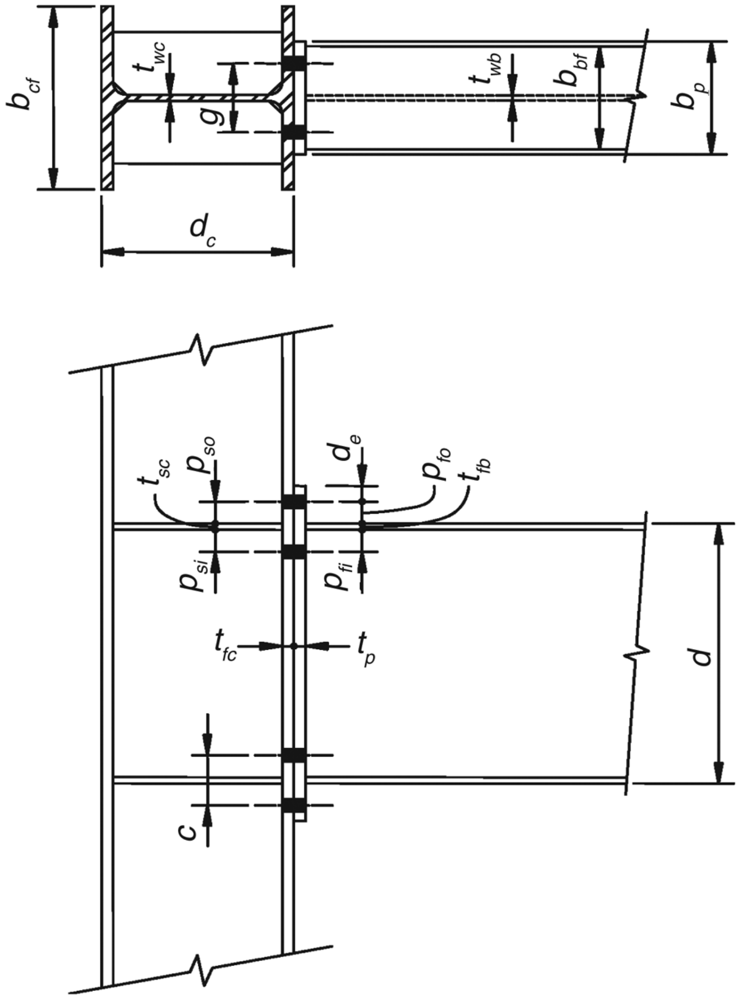


Fig. 6.2. Four-bolt unstiffened extended (4E) end-plate geometry.

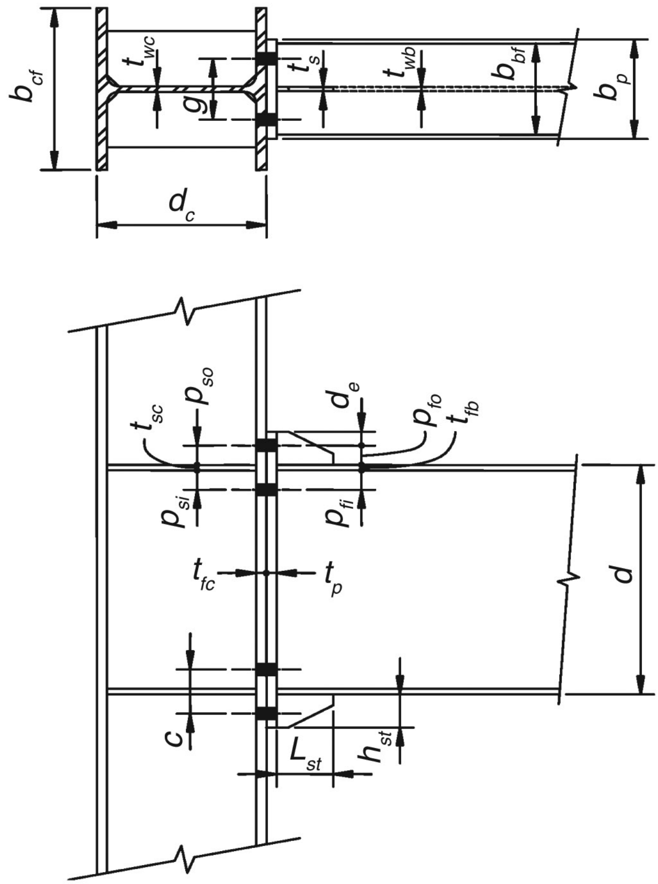


Fig. 6.3. Four-bolt stiffened extended (4ES) end-plate geometry.

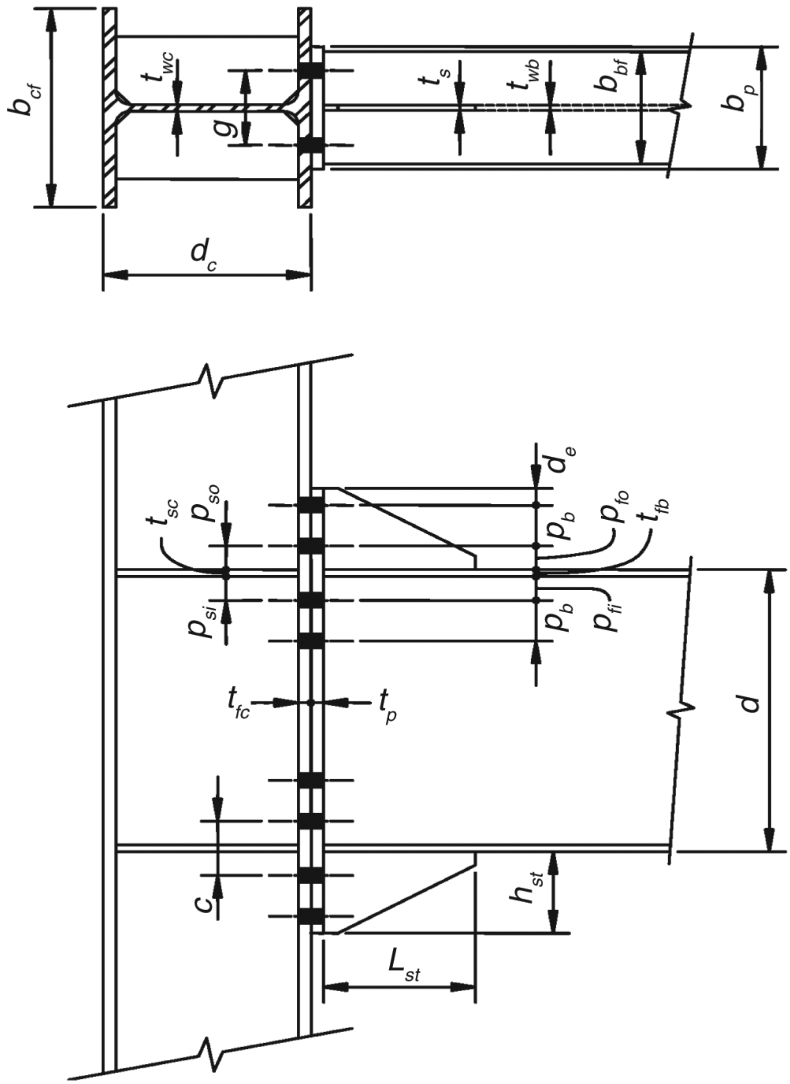


Fig. 6.4. Eight-bolt stiffened extended (8ES) end-plate geometry.

6.9.1 Gage

The gage, g, is as defined in Figures 6.2 through 6.4. The maximum gage dimension is limited to the width of the connected beam flange.

6.9.2 Pitch and Row Spacing

The minimum pitch distance is the bolt diameter plus $^1/_2$ in. (12 mm) for bolts up to 1-in. (25-mm) diameter, and the bolt diameter plus $^3/_4$ in. (19 mm) for larger diameter bolts. The pitch distance, p_{fi} and p_{fo}, is the distance from the face of the beam flange to the centerline of the nearer bolt row, as shown in Figures 6.2 through 6.4.

The spacing, p_b, is as defined in Figure 6.4. The spacing of the bolt rows shall be at least $2^2/_3$ times the bolt diameter.

> **User Note:** A distance of 3 times the bolt diameter is preferred. The distance shall be sufficient to provide clearance for any welds in the region.

6.9.3 End-Plate Width

The width of the end-plate shall be greater than or equal to the connected beam flange width. The effective end-plate width shall not be taken as greater than the connected beam flange plus 1 in. (25 mm).

6.9.4 End-Plate Stiffener

The two extended stiffened end-plate connections, Figures 6.1(b) and (c), require a gusset plate welded between the connected beam flange and the end-plate. The minimum stiffener length shall be:

$$L_{st} = \frac{h_{st}}{\tan 30^\circ} \tag{6.9-1}$$

where h_{st} is the height of the end-plate from the outside face of the beam flange to the end of the end-plate (see Figure 6.5).

The stiffener plates shall be terminated at the beam flange and at the end of the end-plate with landings approximately 1 in. (25 mm) long. The stiffener shall be clipped where it meets the beam flange and end-plate to provide clearance between the stiffener and the beam flange weld.

When the beam and end-plate stiffeners have the same material strengths, the thickness of the stiffeners shall be greater than or equal to the beam web thickness. If the beam and end-plate stiffener have different material strengths, the thickness of the stiffener shall be greater than the ratio of the beam-to-stiffener plate material yield stress times the beam web thickness.

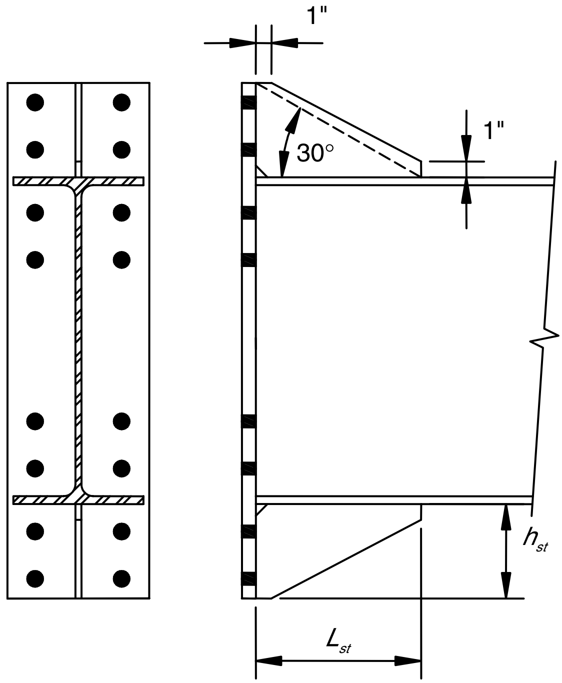


Fig. 6.5. End-plate stiffener layout and geometry (8ES)
(4ES geometry similar).

6.9.5 Finger Shims

The use of finger shims at the top and/or bottom of the connection and on either or both sides is permitted, subject to the limitations of RCSC Specification Section 5.1, as illustrated in Figure 6.6.

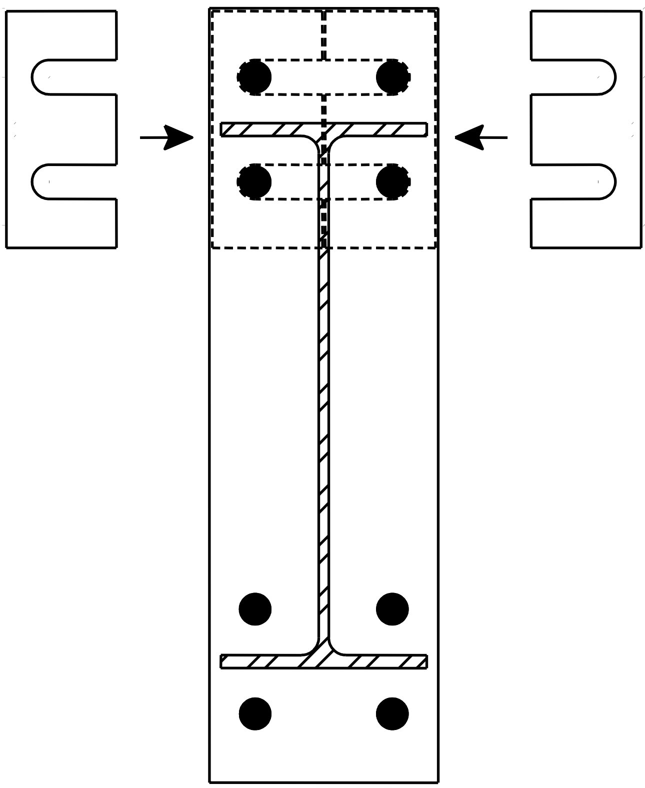


Fig. 6.6. Typical use of finger shims.

6.9.6 Composite Slab Detailing for IMF

In addition to the protected zone limitations, welded shear stud connectors shall not be placed along the top flange of the beam for a distance equal to $1\frac{1}{2}$ times the depth of the beam, measured from the face of the column.

Compressible expansion joint material, at least $\frac{1}{2}$-in. (12-mm) thick, shall be installed between the slab and the column face, in the protected zone.

6.9.7 Welding Details

Welding of the beam to the end-plate shall conform to the following limitations:

(1) Weld access holes shall not be used.

(2) The beam web to end-plate joint shall be made using either fillet welds or *complete joint penetration (CJP) groove welds*. When used, the fillet welds shall be sized to develop the full strength of the beam web in tension from the inside face of the flange to 6 in. (150 mm) beyond the bolt row farthest from the beam flange.

(3) The beam flange to end-plate joint shall be made using a CJP groove weld without *backing*. The CJP groove weld shall be made such that the *root* of the weld is on the beam web side of the flange. The inside face of the flange shall have a $^5/_{16}$-in. (8-mm) fillet weld. These welds shall be demand critical.

(4) Backgouging of the *root* is not required in the flange directly above and below the beam web for a length equal to $1.5k_1$. A full-depth PJP groove weld shall be permitted at this location.

(5) When used, all end-plate stiffener joints shall be made using CJP groove welds.

Exception: When the stiffener is $^3/_8$-in. (10-mm) thick or less, it shall be permitted to use fillet welds that develop the strength of the stiffener.

6.10 Design Procedure

Connection geometry is shown in Figures 6.2, 6.3, and 6.4 for the 4E, 4ES, and 8ES connections, respectively.

<u>End-Plate and Bolt Design</u>

(1) Determine the sizes of the connected members (beams and column) and compute the moment at the face of the column, M_f.

$$M_f = M_{pe} + V_u S_h \qquad (6.9\text{-}2)$$

where

$$M_{pe} = C_{pr} R_y F_y Z_x \qquad (6.9\text{-}3)$$

$V_u = 2M_{pe}/L' + V_{gravity}$

S_h = distance from the face of the column to the plastic hinge, in. (mm)

= the lesser of $d/2$ or $3b_{bf}$ for unstiffened connection (4E) (6.9-4)

$= L_{st} + t_p$ (6.9-5)

R_y = the ratio of the expected yield stress to the specified minimum yield stress, from the AISC *Seismic Provisions*

d = depth of the connecting beam, in. (mm)
b_{bf} = width of the beam flange, in. (mm)
L_{st} = length of the end-plate stiffener, as shown in Figure 6.2, in. (mm)
t_p = thickness of the end-plate, in. (mm)
M_{pe} = probable maximum moment at plastic hinge, kip-in. (N-mm)
L' = distance between plastic hinges, in. (mm)
$V_{gravity}$ = beam shear force resulting from $1.2D + f_1 L + 0.2S$, kips (N)
C_{pr} = Factor to account for peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions, as given in Equation 2.4.3-2
V_u = shear force at the end of the beam, kips (N)
f_1 = load factor determined by the applicable building code for live loads, but not less than 0.5

(2) Select one of the three end-plate moment connection configurations and establish preliminary values for the connection geometry (g, p_{fi}, p_{fo}, p_b, g, h_{st}, L_{st}, etc.) and bolt grade.

(3) Determine the required bolt diameter, $d_{b\ \text{req'd}}$, using one of the following expressions:

$$d_{b\ req'd} = \sqrt{\frac{2M_f}{\pi\phi_n F_{nt}\left(h_0 + h_1\right)}} \quad \text{for four-bolt connections (4E, 4ES)} \qquad (6.9\text{-}6)$$

$$d_{b\ req'd} = \sqrt{\frac{2M_f}{\pi\phi_n F_{nt}\left(h_1 + h_2 + h_3 + h_4\right)}} \quad \text{for eight-bolt connections (8ES)} \qquad (6.9\text{-}7)$$

where
F_{nt} = nominal tensile stress of bolt, 90 ksi (620 MPa) for A325 bolts and 113 ksi (780 MPa) for A490 bolts
h_i = distance from the centerline of the beam compression flange to the centerline of the i^{th} tension bolt row.

(4) Select a trial bolt diameter, d_b, greater than that required in Step 3.

(5) Determine the required end-plate thickness, $t_{p\ req'd}$.

$$t_{p\ req'd} = \sqrt{\frac{1.11\,M_f}{\phi_d F_{yp} Y_p}} \tag{6.9-8}$$

where

F_{yp} = specified minimum yield stress of the end-plate material, ksi (N/mm^2)

Y_p = the end-plate yield line mechanism parameter from Table 6.2, 6.3, or 6.4, in. (mm)

(6) Select an end-plate thickness, t_p, not less than the required value.

(7) Calculate the factored beam flange force.

$$F_{fu} = \frac{M_f}{d - t_{bf}} \tag{6.9-9}$$

where

d = depth of the beam, in. (mm)

t_{bf} = thickness of beam flange, in. (mm)

(8) Check shear yielding resistance of the extended portion of the four-bolt extended unstiffened end-plate (4E):

$$\frac{F_{fu}}{2} < \phi_d R_n = \phi_d\, 0.6\, F_{yp} b_p t_p \tag{6.9-10}$$

where

b_p = width of the end-plate, in. (mm)

If Equation 6.10 is not satisfied, increase the end-plate thickness until it is satisfied.

(9) Check shear rupture resistance of the extended portion of the end-plate in the four-bolt extended unstiffened end-plate (4E):

$$\frac{F_{fu}}{2} < \phi_n R_n = \phi_n\, 0.6\, F_{up} A_n \tag{6.9-11}$$

where

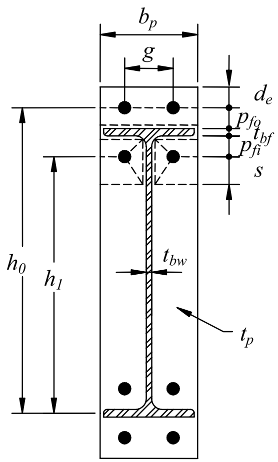
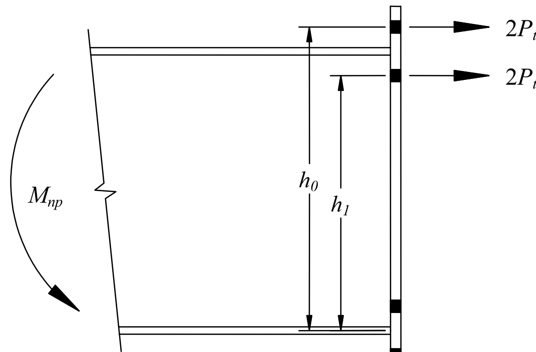
F_{up} = specified minimum tensile strength of the end-plate, ksi (MPa)

A_n = net area of the end-plate = $[b_p - 2(d_b + {}^1\!/_8)]\ t_p$ when standard holes are used, in.^{2} (mm^2) (6.9-12)

d_b = bolt diameter, in. (mm)

If Equation 6.11 is not satisfied, increase the end-plate thickness until it is satisfied.

Table 6.2. Summary of Four-Bolt Extended Unstiffened End-Plate Yield Line Mechanism Parameter

End-Plate Geometry and Yield Line Pattern	Bolt Force Model
	

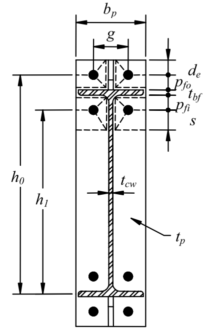
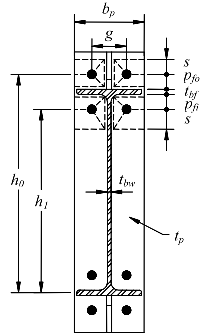
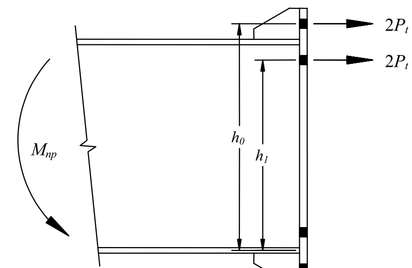
End-Plate

$$Y_p = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{fi}} + \frac{1}{s}\right) + h_0\left(\frac{1}{p_{fo}}\right) - \frac{1}{2}\right] + \frac{2}{g}\left[h_1\left(p_{fi} + s\right)\right]$$

$$s = \frac{1}{2}\sqrt{b_p\, g}$$

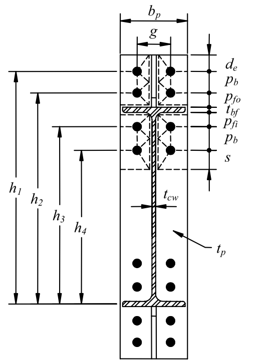
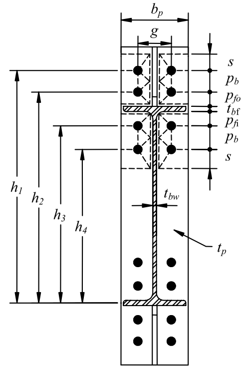
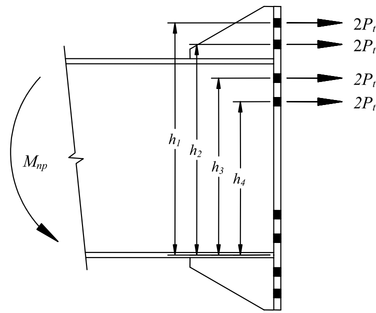
Note: If $p_{fi} > s$, use $p_{fi} = s$

Table 6.3. Summary of Four-Bolt Extended Stiffened End-Plate Yield Line Mechanism Parameter

End-Plate Geometry and Yield Line Pattern		Bolt Force Model
Case 1 ($d_e \le s$)	Case 2 ($d_e > s$)	
		

End-Plate	Case 1 $d_e \le s$	$Y_p = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{fi}} + \frac{1}{s}\right) + h_0\left(\frac{1}{p_{fo}} + \frac{1}{2s}\right)\right] + \frac{2}{g}\left[h_1(p_{fi} + s) + h_0(d_e + p_{fo})\right]$
	Case 2 $d_e > s$	$Y_p = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{fi}} + \frac{1}{s}\right) + h_0\left(\frac{1}{s} + \frac{1}{p_{fo}}\right)\right] + \frac{2}{g}\left[h_1\left(p_{fi} + s\right) + h_0\left(s + p_{fo}\right)\right]$
	$s = \frac{1}{2}\sqrt{b_p\, g}$ Note: If $p_{fi} > s$, use $p_{fi} = s$	

Table 6.4. Summary of Eight-Bolt Extended Stiffened End-Plate Yield Line Mechanism Parameter

End-Plate Geometry and Yield Line Pattern		Bolt Force Model
Case 1 ($d_e \le s$)	**Case 2 ($d_e > s$)**	
		

End-Plate	**Case 1** $d_e \le s$	$Y_p = \frac{b_p}{2}\left[h_1\left(\frac{1}{2d_e}\right)+h_2\left(\frac{1}{p_{fo}}\right)+h_3\left(\frac{1}{p_{fi}}\right)+h_4\left(\frac{1}{s}\right)\right] + \frac{2}{g}\left[h_1\left(d_e+\frac{p_b}{4}\right)+h_2\left(p_{fo}+\frac{3p_b}{4}\right)+h_3\left(p_{fi}+\frac{p_b}{4}\right)+h_4\left(s+\frac{3p_b}{4}\right)+p_b^2\right]+g$
	Case 2 $d_e > s$	$Y_p = \frac{b_p}{2}\left[h_1\left(\frac{1}{s}\right)+h_2\left(\frac{1}{p_{fo}}\right)+h_3\left(\frac{1}{p_{fi}}\right)+h_4\left(\frac{1}{s}\right)\right] + \frac{2}{g}\left[h_1\left(s+\frac{p_b}{4}\right)+h_2\left(p_{fo}+\frac{3p_b}{4}\right)+h_3\left(p_{fi}+\frac{p_b}{4}\right)+h_4\left(s+\frac{3p_b}{4}\right)+p_b^2\right]+g$
	$s = \frac{1}{2}\sqrt{b_p\,g}$ Note: If $p_{fi} > s$, use $p_{fi} = s$	

(10) If using either the four-bolt extended stiffened end-plate (4ES) or eight-bolt extended stiffened end-plate (8ES) connection, select the end-plate stiffener thickness and design the stiffener-to-beam flange and stiffener-to-end-plate welds.

$$t_{s,min} = t_{bw}\left(\frac{F_{yb}}{F_{ys}}\right) \tag{6.9-13}$$

where

t_{bw} = thickness of the beam web, in. (mm)

F_{yb} = specified minimum yield stress of beam material, ksi (MPa)

F_{ys} = specified minimum yield stress of stiffener material, ksi (MPa)

The stiffener geometry shall conform to the requirements of Section 6.9.4. In addition, to prevent local buckling of the stiffener plate, the following width-to-thickness criterion shall be satisfied.

$$\frac{h_{st}}{t_s} \le 0.56\sqrt{\frac{E}{F_{ys}}} \tag{6.9-14}$$

where

h_{st} = the height of the stiffener, in. (mm)

The stiffener-to-beam flange and stiffener-to-end-plate welds shall be designed to develop the stiffener plate in shear at the beam flange and in tension at the end-plate. Either fillet or c*omplete joint penetration (CJP) groove welds* are suitable for the weld of the stiffener plate to the beam flange. If the stiffener plate thickness is greater than $^3/_8$ in., CJP groove welds shall be used for the stiffener-to-end-plate weld. Otherwise, double-sided fillet welds are permitted to be used.

(11) The bolt shear rupture strength of the connection is provided by the bolts at one (compression) flange; thus

$$V_u < \phi_n R_n = \phi_n(n_b)F_v A_b \tag{6.9-15}$$

where

n_b = number of bolts at the compression flange, four for 4ES, and eight for 8ES connections

F_v = nominal shear stress of bolts from Table J3.2 of the AISC *Specification*, ksi (N/mm^2)

A_b = nominal gross area of bolt, in.^{2} (mm^2)

$$V_u = \frac{2M_{pe}}{L'} + V_{gravity} \tag{6.9-16}$$

(12) Check bolt-bearing/tear-out failure of the end-plate and column flange:

$$V_u < \phi_n R_n = \phi_n (n_i) r_{ni} + \phi_n (n_o) r_{no} \tag{6.9-17}$$

where

n_i = number of inner bolts (two for 4E and 4ES, and four for 8ES connections)

n_o = number of outer bolts (two for 4E and 4ES, and four for 8ES connections)

$r_{ni} = 1.2\, L_c t F_u < 2.4\, d_b t F_u$ for each inner bolt (6.9-18)

$r_{no} = 1.2\, L_c t F_u < 2.4\, d_b t F_u$ for each outer bolt (6.9-19)

L_c = clear distance, in the direction of force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in. (mm)

t = end-plate or column flange thickness, in. (mm)

F_u = specified minimum tensile strength of end-plate or column flange material, ksi (N/mm^2)

d_b = diameter of the bolt, in. (mm)

(13) Design the flange to end-plate and web to end-plate welds using the requirements of Section 6.9.7.

<u>Column Side Design</u>

(14) Check the column flange for flexural yielding

$$t_{cf\ req'd} = \sqrt{\frac{1.11\, M_f}{\phi_d F_{yc} Y_c}} \le t_{cf} \tag{6.9-20}$$

where

F_{yc} = specified minimum yield stress of column flange material, ksi (N/mm^2)

Y_c = unstiffened column flange yield line mechanism parameter from Table 6.4 or Table 6.5, in. (mm)

t_{cf} = column flange thickness, in. (mm)

If Equation 6.20 is not satisfied, increase the column size or add web stiffeners (continuity plates).

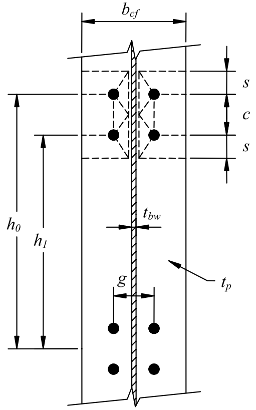
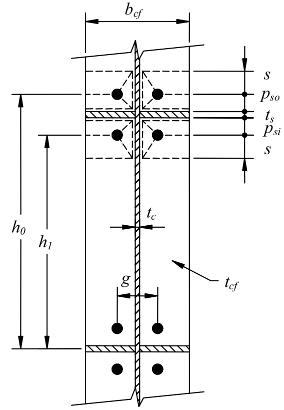
If stiffeners are added, Equation 6.20 must be checked using Y_c for the stiffened column flange from Tables 6.5 and 6.6.

(15) If stiffeners are required for column flange flexural yielding, determine the required stiffener force.

The column flange flexural design strength is

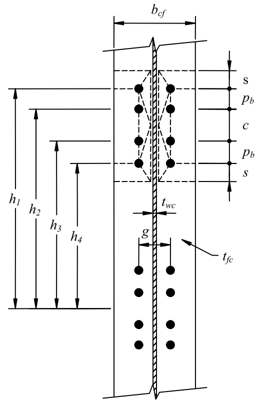
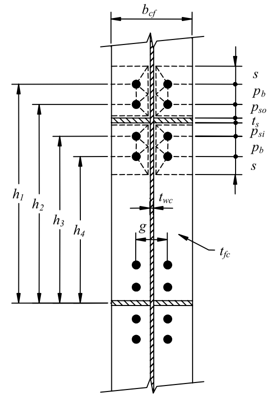
$$\phi_d M_{cf} = \phi_d F_{yc} Y_c t_{cf}^2 \tag{6.9-21}$$

Table 6.5. Summary of Four-Bolt Extended Column Flange Yield Line Mechanism Parameter

Unstiffened Column Flange Geometry and Yield Line Pattern	Stiffened Column Flange Geometry and Yield Line Pattern
	

Unstiffened Column Flange	$Y_c = \frac{b_{cf}}{2}\left[h_1\left(\frac{1}{s}\right) + h_0\left(\frac{1}{s}\right)\right] + \frac{2}{g}\left[h_1\left(s + \frac{3c}{4}\right) + h_0\left(s + \frac{c}{4}\right) + \frac{c^2}{2}\right] + \frac{g}{2}$ $s = \frac{1}{2}\sqrt{b_{cf}\, g}$
Stiffened Column Flange	$Y_c = \frac{b_{cf}}{2}\left[h_1\left(\frac{1}{s} + \frac{1}{p_{si}}\right) + h_0\left(\frac{1}{s} + \frac{1}{p_{so}}\right)\right] + \frac{2}{g}\left[h_1\left(s + p_{si}\right) + h_0\left(s + p_{so}\right)\right]$ $s = \frac{1}{2}\sqrt{b_{cf}\, g}$ Note: If $p_{si} > s$, use $p_{si} = s$

Table 6.6. Summary of Eight-Bolt Extended Stiffened Column Flange Yield Line Mechanism Parameter

	Unstiffened Column **Flange** Geometry and Yield Line Pattern	**Stiffened** Column **Flange** Geometry and Yield Line Pattern
		
Unstiffened Column Flange	$Y_c=\frac{b_{cf}}{2}\left[h_1\left(\frac{1}{s}\right)+h_4\left(\frac{1}{s}\right)\right]+\frac{2}{g}\left[h_1\left(p_b+\frac{c}{2}+s\right)+h_2\left(\frac{p_b}{2}+\frac{c}{4}\right)+h_3\left(\frac{p_b}{2}+\frac{c}{2}\right)+h_4(s)\right]+\frac{g}{2}$ $s=\frac{1}{2}\sqrt{b_{cf}\ g}$	
Stiffened Column Flange	$Y_c=\frac{b_{cf}}{2}\left[h_1\left(\frac{1}{s}\right)+h_2\left(\frac{1}{p_{so}}\right)+h_3\left(\frac{1}{p_{si}}\right)+h_4\left(\frac{1}{s}\right)\right]$ $+\frac{2}{g}\left[h_1\left(s+\frac{p_b}{4}\right)+h_2\left(p_{so}+\frac{3p_b}{4}\right)+h_3\left(p_{si}+\frac{p_b}{4}\right)+h_4\left(s+\frac{3p_b}{4}\right)+p_b^2\right]+g$ $s=\frac{1}{2}\sqrt{b_{cf}\ g}$ Note: If $p_{si}>s$, use $p_{si}=s$	

where

Y_c = unstiffened column yield line mechanism parameter from Table 6.4 or 6.5, in. (mm).

Therefore, the equivalent column flange design force is

$$\phi_d R_n = \frac{\phi_d M_{cf}}{\left(d - t_{bf}\right)} \tag{6.9-22}$$

Using $\phi_d R_n$, the required force for stiffener design is determined in Step 19.

(16) Check the local column web yielding strength of the unstiffened column web at the beam flanges.

Strength requirement: $\phi_d R_n \geq F_{fu}$ (6.9-23)

$$R_n = C_t \left(6k_c + t_{bf} + 2t_p\right) F_{yc} t_{cw} \tag{6.9-24}$$

where

C_t = 0.5 if the distance from the column top to the top face of the beam flange is less than the depth of the column

= 1.0 otherwise

k_c = distance from outer face of the column flange to web toe of fillet (design value) or fillet weld, in. (mm)

t_p = end-plate thickness, in. (mm)

F_{yc} = specified yield stress of the column web material, ksi (MPa)

t_{cw} = column web thickness, in. (mm)

t_{bf} = thickness of beam flange, in. (mm)

If the strength requirement ($\phi_d R_n \geq F_{fu}$) is not satisfied, then column web continuity plates are required.

(17) Check the unstiffened column web buckling strength at the beam compression flange.

Strength requirement:

$$\phi R_n \geq F_{fu} \tag{6.9-25}$$

where $\phi = 0.75$

(a) When F_{fu} is applied a distance greater than or equal to $d_c/2$ from the end of the column

$$R_n = \frac{24\, t_{cw}^3 \sqrt{E F_{yc}}}{h} \tag{6.9-26}$$

(b) When F_{fu} is applied a distance less than $d_c/2$ from the end of the column

$$R_n = \frac{12\, t_{cw}^3 \sqrt{EF_{yc}}}{h} \tag{6.9-27}$$

where

h = clear distance between flanges less the fillet or corner radius for rolled shapes; clear distance between flanges when welds are used for built-up shapes, in. (mm)

If the strength requirement ($\phi_n R_n \geq F_{fu}$) is not satisfied, then column web continuity plates are required.

(18) Check the unstiffened column web crippling strength at the beam compression flange.

Strength requirement:

$$\phi R_n \geq F_{fu} \tag{6.9-28}$$

where $\phi = 0.75$

(a) When F_{fu} is applied a distance greater than or equal to $d_c/2$ from the end of the column

$$R_n = 0.80\, t_{cw}^2 \left[1 + 3\left(\frac{N}{d_c}\right)\left(\frac{t_{cw}}{t_{cf}}\right)^{1.5}\right]\sqrt{\frac{E\, F_{yc} t_{cf}}{t_{cw}}} \tag{6.9-29}$$

(b) When F_{fu} is applied a distance less than $d_c/2$ from the end of the column

(i) for $N/d_c < 0.2$,

$$R_n = 0.40\, t_{cw}^2 \left[1 + 3\left(\frac{N}{d_c}\right)\left(\frac{t_{cw}}{t_{cf}}\right)^{1.5}\right]\sqrt{\frac{E\, F_{yc}\, t_{cf}}{t_{cw}}} \tag{6.9-30}$$

(ii) for $N/d_c > 0.2$,

$$R_n = 0.40\, t_{cw}^2 \left[1 + \left(\frac{4N}{d_c} - 0.2\right)\left(\frac{t_{cw}}{t_{cf}}\right)^{1.5}\right]\sqrt{\frac{E\, F_{yc} t_{cf}}{t_{cw}}} \tag{6.9-31}$$

where

N = thickness of beam flange plus 2 times the groove weld reinforcement leg size, in. (mm)

d_c = overall depth of the column, in. (mm)

If the strength requirement ($\phi_n R_n \geq F_{fu}$) is not satisfied, then column web continuity plates are required.

(19) If stiffener plates are required for any of the column side limit states, the required strength is

$$F_{su} = F_{fu} - min\ \phi R_n \tag{6.9-32}$$

where

$min\ \phi R_n$ = the minimum design strength value from Steps 15 (column flange bending), 16 (column web yielding), 17 (column web buckling), and 18 (column web crippling)

The design of the continuity plates shall also conform to section J10.8 of the AISC *Specification*, and the welds shall be designed in accordance with Section 6.7.3.

(20) Check the panel zone in accordance with Section 6.6.1.

COMMENTARY on the Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications

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

December 13, 2005

C1. GENERAL

C1.1 Scope

Special moment frames (SMF) and intermediate moment frames (IMF) designed in accordance with the AISC *Seismic Provisions* are designed with the 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 yielding of the beams in so-called zones of plastic hinging. 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 unanticipated zones of weakness in connection assemblies; and

(4) Welding practice and workmanship that fell outside the acceptable parameters under the AWS D1.1 Structural Welding Code.

A more complete listing of the causes of damage sustained in the Northridge earthquake may be found in a series of publications (FEMA 350, 2000; FEMA 351, 2000; FEMA 352, 2000; FEMA 353, 2000b; FEMA 355C, 2000; FEMA 355D, 2000) published 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* as well as a pending AWS seismic supplement to the AWS D1.1 Structural Welding Code.

Following the recommendations of the SAC Joint Venture, 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 Appendix S 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.

C1.2 References

References for this Standard are listed in the Bibliography, found at the end of the Commentary.

C1.3 General

Connections that are prequalified under this Standard are intended to withstand inelastic deformation through controlled yielding in specific behavioral modes. In order 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 capacity formulations contained in the LRFD method are consistent with this approach.

C2. DESIGN REQUIREMENTS

C2.1 Special and Intermediate Moment Frame Connection Types

Limitations included in this Standard for various prequalified connections include specification of permissible materials specifications 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.

C2.3 Members

C2.3.2 Built-up Members

The behavior of built-up I-shaped members has been extensively tested in bolted end-plate 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.

C2.3.2b 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

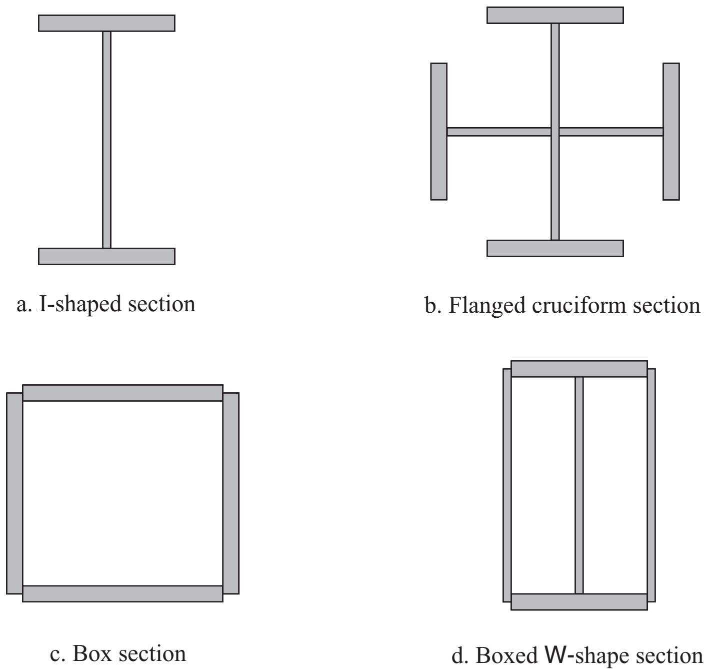


Fig. C–2.1. Column shapes.

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. All of these tests were uniaxial in nature. That is, the beam or beams connecting to the column were in a single plane and moments applied to the column during the test induced flexure about a single axis.

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 connections types where the predominant inelastic behavior is in the beam rather than the column.

Similarly, although there has been virtually no 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. Therefore, 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.

Limited testing of connections of wide-flange beams to the webs of I-shaped columns had been conducted prior to the Northridge earthquake by Popov, Engelhardt and others. 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 prequalified under this Standard.

(2) Boxed Wide-Flange Columns

Testing of connection assemblies comprising boxed wide-flange columns has not specifically been conducted. However, the behavior of columns of this cross-section type is believed to be bounded by the behavior of standard wide-flange section columns and that of box section columns, both of which have been tested.

(4) Flanged Cruciform Columns

Testing of connection assemblies comprised of cruciform wide-flange columns has not specifically been conducted. However, the behavior of columns of this cross-section type is believed to be similar to the behavior of standard wide-flange section columns that have been extensively tested.

C2.4 Connection Design Parameters

C2.4.1 Load Combinations and Resistance Factors

A significant factor considered in the formulation of resistance factors is the desirability or undesirability of various limit states. Limit states that are considered brittle and subject to sudden catastrophic failure are typically assigned lower resistance factors than those that exhibit relatively benign yielding failure. Since, 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.

C2.4.2 Plastic Hinge Location

This Standard specifies the 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.

C2.4.3 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.

C2.4.4 Beam Flange 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 beam flange and column due to non-uniform stiffness of the attached elements.

When the beam flange connects to the flange of a wide-flange, built-up I-shape or cruciform W-shaped column in which the column extends above and below the beam and the column flange thickness satisfies Equations 2.4.4-1 and 2.4.4-2, continuity plates are not required as beam-flange forces can be adequately transferred to the column webs without the stiffening effects and secondary load paths provided by these plates. However, these equations have been developed based on consideration of the behavior of columns in lower stories of buildings, where the column extends a considerable distance above the top flange of the connected beam. Equations 2.4.4-1 and 2.4.4-2 do not apply in the top story of a building, where the column terminates at approximately the level of the top flange of the beam. In such cases beam-flange continuity plates or column cap plates, having a thickness not less than that of the connected beam flange, should be provided. Figure C–2.2 presents a detail for such a connection, where the beam flange is welded directly to the cap plate and the cap plate is welded to the column so as to deliver the beam-flange forces to the column web.

Alternatively, if the column projects sufficiently above the beam top flange, Equations 2.4.4-1 and 2.4.4.-2 can be considered valid. Although comprehensive research to establish the necessary distance the column must extend above the beam for this purpose has not been performed, it may be sufficient if the column is extended a distance not less than $d_c/2$ or $b_f/2$, whichever is less, above the top beam flange.

For boxed wide-flange section columns in which the beams are connected to the flange of the I-shaped section, Equations 2.4.4-3 and 2.4.4-4 have been developed to provide a similar stiffness of column flange as that provided by Equations 2.4.4-1 and 2.4.4-2 for unboxed sections. As with Equations 2.4.4-1 and 2.4.4-2, Equations 2.4.4-3 and 2.4.4-4 are not strictly valid for the case of a moment

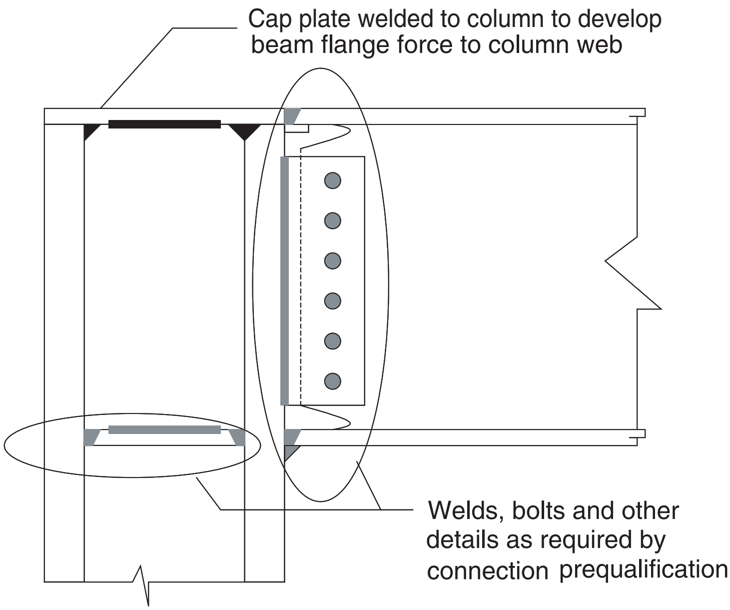


Fig. C–2.2. Cap plate detail at column top.

connection at the roof level of a building, in which the column does not extend significantly above the beam top flange. In these cases, a cap plate detail similar to that illustrated in Figure C–2.2 should be used.

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.

C2.4.4b Continuity Plate to Column Attachment

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 prequalification requires that continuity plates be attached to column flanges with CJP groove welds in order that 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 probably not necessary to use CJP groove welds to attach the continuity plate to the column flange that does not have a beam attached. In such cases, acceptable performance can probably be obtained by attaching the continuity plate to the column with a pair of minimum-size fillet welds.

C3. WELDING REQUIREMENTS

C3.3 Backing at Beam-to-Column and Continuity-Plate-to-Column Joints

At the root of groove welds between beam flanges or continuity plates to column flanges, the inherent lack of 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. 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 and others, 1997).

C3.3.1 Steel Backing at Continuity Plates

The stress and strain level at the groove weld between 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 reinforcing fillet weld reduces the stress concentration at the right-angle intersection of the continuity plate and the column flange.

C3.3.2 Steel Backing at Beam Bottom Flange

The removal of backing, whether fusible or non-fusible, followed by backgouging to sound weld metal, is performed 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 when this is done.

C3.3.3 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.

C3.3.4 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.

C3.3.5 Non-fusible Backing at Beam Flange-to-Column Joints

After backing is removed, backgouging to sound metal removes potential root flaws within the welded joint. A reinforcing fillet weld with a $^5/_{16}$-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 $^5/_{16}$ 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, therefore the horizontal leg of the fillet weld need not be extended to base metal.

C3.4 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.

C3.5 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.

C3.6 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, CVN toughness has been recorded to be less than 2 ft-lbs 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 that 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, NDT of cascaded weld ends in groove welds at this location need not and should not be performed.

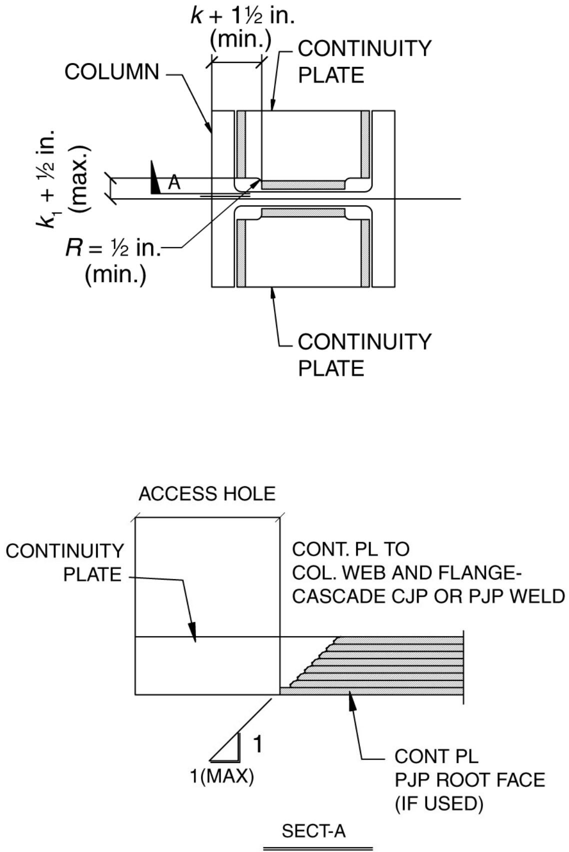


Fig. C–3.1. Continuity plate welding.

C3.7 Quality Control and Quality Assurance

Appendix Q of the *Seismic Provisions* specifies the minimum requirements for a quality assurance plan for the seismic load resisting system. It may be appropriate to adjust the Appendix Q provisions for a particular project based on the qualifications of the contractor(s) involved. 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.

C4. BOLTING REQUIREMENTS

C4.1 Fastener Assemblies

ASTM F1852 twist-off type tension-control fastener assemblies are appropriate substitutes for ASTM A325 bolts. At the time of publication of AISC 358, an ASTM standard for twist-off type tension-control fastener assemblies with a strength equivalent to that of ASTM A490 bolts had not yet been adopted, but was in development. Such assemblies are commonly produced and used, and are addressed by the RCSC Specification.

C4.2 Installation Requirements

Section 7.2 of the *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 *Seismic Provisions*.

C4.3 Quality Control and Quality Assurance

See Section C3.7.

C5. REDUCED BEAM SECTION (RBS) MOMENT CONNECTION

C5.1 General

In a reduced beam section (RBS) moment connection, portions of the beam flanges are selectively trimmed in the region adjacent to the beam-to-column connection. In an RBS connection, yielding and hinge formation are intended to occur primarily within the reduced section of the beam, and thereby limit the moment and inelastic deformation demands developed at the face of the column.

Review of the research literature indicates that a large number of RBS connections have been tested under a variety of conditions by different investigators at institutions throughout the world. A listing of relevant research is presented in the bibliography section at the end of this document. A significant amount of testing on RBS connections was also conducted under the FEMA/SAC program (FEMA 355D, 2000d). Consequently, a large body of test data was available for purposes of connection prequalification. Review of available test data indicates that RBS specimens, when designed and constructed according to the limits and procedures presented herein, have developed interstory drift angles of at least 0.04 radian under cyclic loading on a consistent basis.

Tests on RBS connections show that yielding is generally concentrated within the reduced section of the beam and may extend, to a limited extent, to the face of the column. Peak strength of specimens is usually achieved at an interstory drift angle of approximately 0.02 to 0.03 radian. 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 0.05 to 0.07 radian, by low cycle fatigue fracture at local flange buckles within the RBS.

RBS connections have been tested using single-cantilever type specimens (one beam attached to column), and double-sided specimens (specimen consisting of a single column, with beams attached to both flanges). Tests have been conducted primarily on bare-steel specimens, although some testing is also reported on specimens with composite slabs. Tests with composite slabs have shown that the presence of the slab provides a beneficial effect by helping to maintain the stability of the beam at larger interstory drift angles.

Most RBS test specimens have been tested pseudo-statically, using a loading protocol in which applied displacements are progressively increased, such as the loading protocol specified in ATC-24 (ATC, 1992) and the loading protocol developed in the FEMA/SAC program and adopted in Appendix S of the AISC *Seismic Provisions*. Two specimens were tested using a loading protocol intended to represent near-source ground motions that contain a large pulse. Several specimens were also tested dynamically. The radius-cut RBS specimens have performed well under all of these loading conditions.

C5.2 Systems

Review of the research literature presented in the reference section at the end of this document and summarized in Section C5.1 indicates that the RBS connection meets the prequalification requirements for special and intermediate moment frames in Appendix P of the AISC *Seismic Provisions*.

C5.3 Prequalification Limits

C5.3.1 Beam Limitations

A wide range of beam sizes has been tested with the radius-cut RBS. The smallest beam size reported in the literature was a W530×82 (Canadian designation), which is roughly equivalent to a W21×50. The heaviest beam reported is a W36×300 (FEMA 355D, 2000d). Although the AISC *Seismic Provisions* permit limited increases in beam depth and weight compared to the maximum sections tested, the prequalification limits for maximum beam depth and weight were established based on the test data for W36×300. It was the judgment of the CPRP that for the purposes of establishing initial prequalification limits, adherence to the maximum tested specimen would be appropriately conservative. There is no evidence that modest deviations from the maximum tested specimen would result in significantly different performance, and the limit on maximum flange thickness is approximately 4 percent thicker than the flange in a W36×300.

Both beam depth and beam span-to-depth ratio are significant in the inelastic behavior of beam-to-column connections. For the same induced curvature, deep beams will experience greater strains than shallower beams. Similarly, beams with shorter span-to-depth ratio will have a sharper moment gradient across the beam span, resulting in reduced length of the beam participating in plastic hinging and increased strains under inelastic rotation demands. Most of the beam-to-column assemblies that have been tested used configurations approximating beam spans of about 25 feet and beam depths varying from W30 to W36 so that beam span-to-depth ratios were typically in the range of eight to ten (FEMA 355D, 2000d). Given the degree to which most specimens significantly exceeded the minimum interstory drift demands, it was judged reasonable to set the minimum span-to-depth ratio at seven for SMF and five for IMF.

Local buckling requirements for members subjected to significant inelastic rotation are discussed in the AISC *Seismic Provisions*. For the purposes of calculating the width-to-thickness ratio, it is permitted to take the flange width at the two-thirds point of the RBS cut. This provision recognizes that the plastic hinge of the beam forms within the length of the RBS cut where the width of the flange is less than at the uncut section. This provision will result in a lower width-to-thickness ratio when taken at the RBS cut compared to that at the uncut section. Many of the RBS tests conducted as a part of the FEMA/SAC program used a W30×99 beam, which does not quite satisfy the flange width-to-thickness ratio at the uncut section. Nevertheless, the tests were successful. For these reasons, it was judged reasonable to permit the calculation of the width-to-thickness ratio a reasonable distance into the RBS cut.

In developing this prequalification, the CPRP also reviewed lateral bracing requirements for beams with RBS connections. Some concerns were raised in the past that the presence of the RBS flange cuts might make the beam more prone to lateral torsional buckling and that supplemental lateral bracing should be provided at the RBS. The issue of lateral bracing requirements for beams with RBS connections was subsequently investigated in both experimental and analytical studies (FEMA 355C, 2000c; Yu and others, 2000). These studies indicated that for bare steel specimens (no composite slab), interstory drift angles of 0.04 radian can be achieved without a supplemental lateral brace at the RBS, as long as the normal lateral bracing required for beams in SMF systems is provided, per Section 9.8 of the AISC *Seismic Provisions*.

Studies also indicated that although supplemental bracing is not required at the RBS to achieve 0.04 radian interstory drift angles, the addition of a supplemental brace can provide for improved performance. Tests on RBS specimens with composite slabs indicated that the presence of the slab provided a sufficient stabilizing effect that a supplemental brace at the RBS is not likely to provide significantly improved performance (FEMA 355C, 2000c; Engelhardt, 1999; Tremblay, 1997). Based on the available data, beams with RBS connections that support a concrete structural slab are not required to have a supplemental brace at the RBS. If no floor slab is present, then a supplemental brace is required at the RBS.

In cases where a supplemental brace is provided, the brace should not be connected within the reduced section (protected zone). Welded or bolted brace attachments in this highly strained region of the beam may serve as fracture initiation sites. Consequently, if a supplemental brace is provided, it should be located at or just beyond the end of the RBS that is farthest from the face of the column.

The protected zone is defined as shown in Figure 5.1 and extends from the face of the column to the end of the RBS farthest from the column. This definition is based on test observations that indicate yielding typically does not extend past the far end of the RBS cut.

C5.3.2 Column Limitations

Nearly all tests of RBS connections have been performed with the beam flange welded to the column flange (i.e., strong-axis connections). The limited amount of weak-axis testing has shown acceptable performance. In the absence of more tests, the CPRP recommended limiting prequalification to strong-axis connections only.

The majority of RBS specimens were constructed with W14 columns. However, a number of tests have also been conducted using deeper columns, including W18, W27 and W36 columns. Testing of deep-column RBS specimens under the FEMA/SAC program indicated that stability problems may occur when RBS connections are used with deep columns (FEMA 355C, 2000c). In FEMA 350, 2000a, RBS connections were only prequalified for W12 and W14 columns.

The specimens in the FEMA/SAC tests conducted showed a considerable amount of column twisting (Gilton and others, 2000). However, two of the three specimens tested achieved 0.04-radian rotation, albeit with considerable strength degradation. The third specimen just fell short of 0.04-radian rotation and failed by fracture of the column web near the *k*-area. Subsequent study attributed this fracture to column twisting.

Subsequent to the FEMA/SAC tests, an analytical study (Shen, 2002) concluded that boundary conditions used in these tests may not be representative of what would be found in an actual building. Consequently, the large-column twisting (and presumably resultant *k*-area column fracture) seen in the FEMA/SAC tests would not be expected in real buildings. The study also concluded that deep columns should not behave substantially different from W14 columns and that no special bracing is needed when a slab is present. This was followed by a more extensive analytical and large-scale experimental investigation on RBS connections with columns up to W36 in depth (Ricles and others, 2004). This investigation showed that good performance can be achieved with deep columns when a composite slab is present or when adequate lateral bracing is provided for the beam and/or column in the absence of a slab. Based on a review of this recent research, the prequalification of RBS connections is extended herein to include W36 columns.

The behavior of RBS connections with cruciform columns is expected to be similar to that of a rolled wide-flange column because the beam flange frames into the column flange, the principal panel zone is oriented parallel to that of the beam and the web of the cut wide-flange column is to be welded with a CJP groove weld to the continuous web one foot above and below the depth of the frame girder. Given these similarities and the lack of evidence suggesting behavioral limit states different from those associated with rolled wide-flange shape, the Committee determined to limit cruciform column depths to those imposed on wide-flange shapes.

Successful tests have also been conducted on RBS connections with built-up box columns. The largest box column for which test data was available was 24 in. by 24 in. Consequently, RBS connections have been prequalified for use with built-up box columns up to 24 in. Limits on the width-thickness ratios for the walls of built-up box columns are specified in Section 2.3.2b(3) and were chosen to reasonably match the box columns that have been tested.

The use of box columns participating in orthogonal moment frames, that is, with RBS connections provided on orthogonal beams, is also prequalified. Although no data were available for test specimens with orthogonal beams, it was the judgment of the committee that this condition should provide ostensibly the same performance as single-plane connections, since the RBS does not rely on panel zone yielding for good performance, and the column is expected to remain essentially elastic for the case of orthogonal connections.

Based on successful tests on wide-flange columns and on built-up box columns, it was the judgment of the Committee that boxed wide-flange columns would also be expected to provide acceptable performance. Consequently, RBS connections are prequalified for use with boxed wide-flange columns. When moment connections are made only to the flanges of the wide-flange portion of the boxed wide-flange, the column may be up to W36 in depth. When the boxed wide-flange column participates in orthogonal moment frames, then neither the depth nor the width of the column is allowed to exceed 24 in. (600 mm), applying the same limits as for built-up boxes.

C5.4 Beam-Column Relationship Limitations

Column panel zone strength provided on RBS test specimens has varied over a wide range. This includes specimens with very strong panel zones (no yielding in panel zone), specimens with very weak panel zones (essentially all yielding in panel zone and no yielding in beam), and specimens where yielding has been shared between the panel zone and the beam. Good performance has been achieved for all levels of panel zone strength, including panel zones that are substantially weaker than permitted in Section 9.3a of the AISC *Seismic Provisions* (FEMA 355C, 2000c). However, there are concerns that very weak panel zones may promote fracture in the vicinity of the beam-flange groove welds due to "kinking" of the column flanges at the boundaries of the panel zone. Consequently, the minimum panel zone strength specified in Section 9.3a of the AISC *Seismic Provisions* is required for prequalified RBS connections.

C5.5 Beam Flange-to-Column Flange Weld Limitations

Complete joint penetration groove welds joining the beam flanges to the column flanges provided on the majority of RBS test specimens have been made by the self-shielded flux cored arc welding process (FCAW-S) using electrodes with a minimum specified CVN toughness. Three different electrode designations have commonly been used in these tests: E71T-8, E70TG-K2, and E70T-6. Further, for most specimens, the bottom flange backing was removed and a reinforcing fillet added, top flange backing was fillet welded to the column, and weld tabs were removed at both the top and bottom flanges.

Test specimens have employed a range of weld access-hole geometries, and results suggest that connection performance is not highly sensitive to the weld access-hole geometry. Consequently, prequalified RBS connections do not require specific access-hole geometry. However, as a minimum, access holes must conform to the requirements of Figure C-J1.2 of the AISC *Specification*. Although not required, the access-hole geometry shown in Figure 11-1 of the AISC *Seismic Provisions* may result in improved reliability of the connection.

C5.6 Beam Web-to-Column Connection Limitations

Two types of web connection details have been used for radius-cut RBS test specimens: a welded and a bolted detail. In the welded detail, the beam web is welded directly to the column flange using a complete joint penetration groove weld. For the bolted detail, pretensioned high-strength bolts are used. Specimens with both types of web connections have achieved at least 0.04-radian interstory drift angles, and consequently both types of web connection details were permitted for RBS connections in (FEMA 350, 2000a).

Previous test data (Engelhardt and others, 2000) indicate that beyond an interstory drift angle of 0.04 radian, specimens with bolted web connections show a higher incidence of fracture occurring near the beam-flange groove welds, as compared to specimens with welded web connections. Thus, while satisfactory performance is possible with a bolted web connection, previous test data indicate that a welded web is beneficial in reducing the vulnerability of RBS connections to fracture at the beam-flange groove welds.

Subsequent to the SAC/FEMA testing on RBS connections, a test program (Lee and others, 2004) was conducted which directly compared RBS connections that were nominally identical, except for the web connection detail. The RBS specimens with welded web connections achieved 0.04-radian interstory drift angle, whereas as RBS specimens with bolted web connections failed to achieve 0.04 radian.

Thus, while past successful tests have been conducted on RBS connections with bolted web connections, recent data has provided contradictory evidence, suggesting bolted web connections may not be suitable for RBS connections when used for SMF applications. Until further data is available, it was the judgment of the CPRP to require a welded web connection for RBS connections prequalified for use in SMF. For IMF applications, bolted web connections are acceptable.

C5.7 Fabrication of Flange Cuts

Various shapes of flange cutouts are possible for RBS connections, including a constant cut, a tapered cut, and a radius cut. Experimental work has included successful tests on all of these types of RBS cuts. The radius cut avoids abrupt changes of cross section, reducing the chances of a premature fracture occurring within the reduced section. Further, the majority of tests reported in the literature used radius-cut RBS sections. Consequently, only the radius-cut RBS shape is prequalified.

An issue in the fabrication of RBS connections is the required surface finish and smoothness of the RBS flange cuts. No research data was found that specifically addressed this issue. Consequently, finish requirements for RBS cuts were chosen based on judgment and are consistent with those specified in (FEMA 350, 2000a).

C5.8 Design Procedure

Dimensions of the RBS cuts for the test specimens reported in the literature vary over a fairly small range. The distance from the face of the column to the start of the RBS cut (designated as “a” in Figure 5.1) ranged from 50 to 75 percent of the beam-flange width. The length of the cuts (designated as “b” in Figure 5.1) has varied from approximately 75 to 85 percent of the beam depth. The amount of flange width removed at the minimum section of the RBS has varied from about 38 to 55 percent. Flange removal for prequalified RBS connections is limited, based on judgment, to a maximum of 50 percent, to avoid excessive loss of strength or stiffness.

The design procedure presented herein for prequalified RBS connections is similar to that presented in FEMA 350, 2000a. The overall basis for sizing the RBS cut in this design procedure is to limit the maximum beam moment that can develop at the face of the column to the beam's actual plastic moment (based on expected yield stress) when the minimum section of the RBS is fully yielded and strain hardened. Test data indicate that connecting the beam at the face of the column in accordance with the requirements herein allows the connection to resist this level of moment while minimizing the chance of fracture at the beam-flange groove welds.

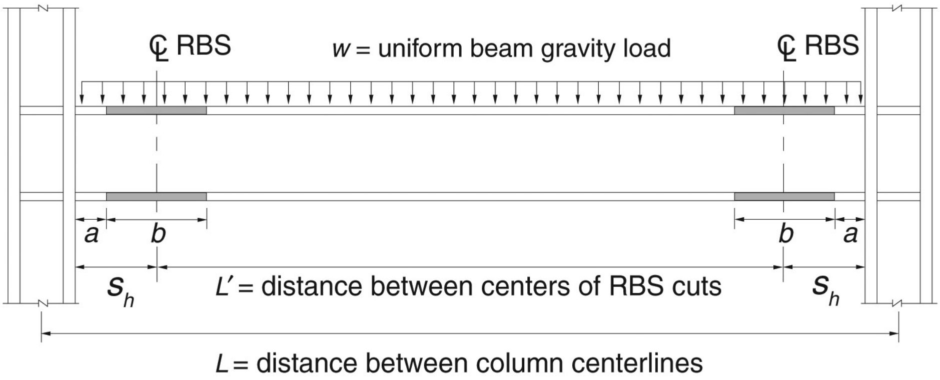
Step 4 of the design procedure requires computation of the shear force at the center of the RBS cut. This shear force is a function of the gravity load on the beam and the plastic moment capacity of the RBS. An example calculation is shown in Figure C–5.1 for the case of a beam with a uniformly distributed gravity load.

For gravity load conditions other than a uniform load, the appropriate adjustment should be made to the free-body diagram in Figure C–5.1 and to Equations C–5.8-1 and C–5.8-2.

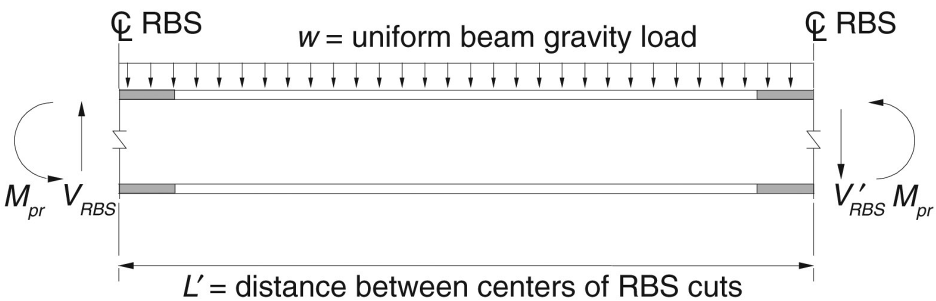
Equations C–5.8-1 and C–5.8-2 assume that plastic hinges will form at the RBS at each end of the beam. If the gravity load on the beam is very large, the plastic hinge at one end of the beam may move toward the interior portion of the beam span. If this is the case, the free-body diagram in Figure C–5.1 should be modified to extend between the actual plastic hinge locations. To determine whether Equations C–5.8-1 and C–5.8-2 are valid, draw the moment diagram for the segment of the beam shown in Figure C–5.1(b), that is, for the segment of the beam between the centers of the RBS cuts. If the maximum moment occurs at the ends of the span, then Equations C–5.8-1 and C–5.8-2 are valid. If the maximum moment occurs within the span and exceeds M_{pe} of the beam (see Equation 5-8), then the modification described above will be needed.

$$V_{RBS} = \frac{2M_{pr}}{L'} + \frac{wL'}{2} \tag{C-5.8-1}$$

$$V'_{RBS} = \frac{2M_{pr}}{L'} - \frac{wL'}{2} \tag{C-5.8-2}$$



(a) Beam with RBS cuts and uniform gravity load



(b) Free-body diagram of beam between RBS cuts and calculation of shear at RBS

Fig. C–5.1. Example calculation of shear at center of RBS cuts.

C6. BOLTED UNSTIFFENED AND STIFFENED EXTENDED END-PLATE MOMENT CONNECTIONS

C6.1 General

The three extended end-plate moment configurations currently addressed in this chapter are the most commonly used end-plate connection configurations in steel moment frames. *AISC Design Guide No. 4,* 2nd Edition (Murray and Sumner, 2003) provides background, design procedures, and complete design examples for the three configurations. The Guide was developed before this Standard was written, and there are small differences between the design procedures in the Guide and in Section C6.10. The primary difference is the resistance factor, ϕ-factor, values. The Standard supersedes the Design Guide in all instances.

Prequalification test results for the three extended end-plate moment connections are found in FEMA, 1997; Meng, 1996; Meng and Murray, 1997; Ryan and Murray, 1999; Sumner and others, 2000a; Sumner and others, 2000b; Sumner and Murray, 2001; Sumner and Murray, 2002. Results of similar testing but not used for prequalification are found in Adey and others, 1997; Adey and others, 1998; Adey and others, 2000; Castellani and others, 1998; Coons, 1999; Ghobarah and others, 1990; Ghobarah and others, 1992; Johnstone and Walpole, 1981; Korol and others, 1990; Popov and Tsai, 1989; Tsai and Popov, 1990.

The intent of the design procedure in Section 6.10 is to provide an end-plate moment connection with sufficient strength to develop the strength of the connected flexural member. The connection does not provide any contribution to inelastic rotation. All inelastic deformation for an end-plate connection is achieved by beam yielding and/or column panel zone deformation.

The design procedure in Section 6.10 is based on Borgsmiller and Murray, 1995, and is similar to the "thick plate" procedure in *AISC Design Guide No. 16* (Murray and Shoemaker, 2002). The procedure is basically the same as that in FEMA 350, 2000a, but with much clarification. Applicable provisions in FEMA 353, 2000b are incorporated into the procedure as well.

C6.2 Systems

The three extended end-plate moment connections in Figure 6.1 are prequalified for use in IMF and SMF systems, except in SMF systems where the beam is in direct contact with concrete structural slabs. The exception applies only when shear studs are used to attach the concrete slab to the connected beam and is because of the lack of test data to date. All of the prequalification testing has been performed with bare steel specimens, except for one test reported in Sumner and Murray, 2000. In this test, headed studs were installed from near the end-plate moment connection to the end of the beam, and the concrete was in contact with the column flanges and web. The lower bolts failed prematurely by tension rupture because of the increase in the distance from the neutral axis due to the presence of the composite slab.

C6.3 Prequalification Limits

The parametric limitations in Table 6.1 were determined from reported test data in the prequalification references. Only connections that are within these limits are prequalified.

Beams may be either hot-rolled or built-up. If built-up sections are used, the web-to-flange weld may be a one-sided fillet weld, except within the beam, depth, or three times the flange width of the face of the end-plate. Within this length, fillet welds on both sides are required of a size at least 1/4 in. (6 mm) for constructability or 0.75 times the beam-flange web thickness to develop the web material, whichever is greater. Complete joint penetration (CJP) groove welds may be used in lieu of fillet welds.

For tapered members, the depth of the beam at the connection is used to determine the limiting span-to-depth ratio.

C6.4 Beam Limitations

The beam size limitations in Table 6.1 are directly related to connection testing. Since many of the tested beam sections were built-up members, the limitations are in cross-section dimensions instead of rolled-beam designations as used in Section 5 for RBS. There is no evidence that modest deviations from these dimensions will result in significantly difference performance.

Similar to RBS testing, most of the tested beam-column assemblies had configurations approximating beam span-to-depth ratios in the range of eight to ten. However, it was judged reasonable to set the minimum span-to-depth ratio at seven for SMF and five for IMF.

The protected zone, the distance from the face of the end-plate for unstiffened, or from the end of the stiffener for stiffened, which is specified as the smaller of the beam depth and three times the flange width, is based on test observations.

C6.5 Column Limitations

Extended moment end-plate connections may be used only with rolled or built-up I-shaped sections and must be flange connected. There are no other specific column requirements for extended end-plate moment connections.

C6.6 Beam-Column Relationship Limitations

There are no specific beam-to-column relationship limitations for extended end-plate moment connections.

C6.7 Continuity Plates

Continuity plate design must conform to the requirements of Section 2.4.4. The design procedure in Section 6.10 contains provisions specific to end-plate moment connections, and the procedure is discussed generally in AISC Design Guide No. 13 (Carter, 1999).

C6.8 Bolts

Prequalification tests have been conducted with both pretensioned ASTM A325 and A490 bolts. Bolt length should be such that at least two complete threads are between the unthreaded portion of the shank and the face of the nut after the bolt is pretensioned. Slip-critical connection provisions are not required for end-plate moment connections.

C6.9 Connection Detailing

Maximum gage, that is, the horizontal distance between outer bolt columns, is limited to the width of the beam flange to ensure a stiff load path. Monotonic tests have shown that the stiffness and strength of an end-plate moment connection are decreased when the bolt gage is wider than the beam flange.

Inner bolt pitch, the distance between the face of the beam flange and the first row of inside or outside bolts, must be sufficient to allow bolt tightening. The minimum pitch values specified have been found to be satisfactory. An increase in pitch distance can significantly increase the required end-plate thickness.

The end-plate can be wider than the beam flange, but the width used in design calculations is limited to the beam flange width plus 1 in. (25 mm). This limitation is based on unpublished results of monotonic tests of end-plate connections.

The requirements for the length of beam-flange-to-end-plate stiffeners are established to ensure a smooth load path. The 30° angle is the same as used for determining the Whitmore section width in other types of connections. The required 1-in. (25-mm) land is needed to ensure the quality of the vertical and horizontal weld terminations.

Tests have shown that the use of finger shims between the end-plate and the column flange do not affect the performance of the connection (Sumner and others, 2000a).

Design procedures are not available for connections of beams with composite action at an end-plate moment connection. Therefore, careful composite slab detailing is necessary to prevent composite action, which may increase tension forces in the lower bolts. Welded stud shear connectors are not permitted within $1^1/_2$ times the beam depth, and compressible material is required between the concrete slab and the column face (Sumner and Murray, 2002; Yang and others, 2003).

Cyclic testing has shown that use of weld access holes can cause premature fracture of the beam flange at end-plate moment connections (Meng and Murray, 1997). Short to long weld access holes were investigated with similar results. Therefore, weld access holes are not permitted for end-plate moment connections.

Strain gage measurements have shown that the web plate material in the vicinity of the inside tension bolts generally reaches the yield strain (Murray and Kukreti, 1988). Consequently, it is required that the web-to-end-plate weld(s) in the vicinity of the inside bolts be sufficient to develop the strength of the beam web.

The beam-flange-to-end-plate and stiffener weld requirements equal or exceed the welding that was used to prequalify the three extended end-plate moment connections. Since weld access holes are not permitted, the beam-flange-to-end-plate weld at the beam web is necessarily a partial-joint-penetration (PJP) groove weld. The prequalification testing has shown that these conditions are not detrimental to the performance of the connection.

C6.10 Design Procedure

The design procedure in this section, with some modification, was used to design the prequalification test specimens. The procedure is very similar to that in the *AISC Design Guide No. 4,* 2nd Edition (Murray and Sumner, 2003) except that different resistance factors are used. Example calculations are found in the Design Guide. Column stiffening example calculations are also found in the *AISC Design Guide No. 13* (Carter, 1999).

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