Supplement No. 1 to ANSI/AISC 358-05 Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications

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PREFACE

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This supplement to the 2005 standard, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications,* modifies an existing prequalified moment connection and adds three new prequalified moment connections. The principal additions contained in the supplement may be summarized as follows:

- Bolted unstiffened extended end plate (BUEEP) moment connections and bolted stiffened extended end plate (BSEEP) moment connections have been prequalified for SMFs with concrete structural slabs, subject to the requirements of Chapter 6.

- Bolted flange plate (BFP) moment connections are prequalified per Chapter 7.

- Welded unreinforced flange–welded web (WUF–W) moment connections are prequalified per Chapter 8.

- Kaiser bolted bracket (KBB) moment connections are prequalified per Chapter 9.

TABLE OF CONTENTS

CHAPTER 1

GENERAL

1.2.   References

Replace this section with the following:

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* (herein referred to as the AISC *Seismic Provisions*)

2004 AWS *D1.1 Structural Welding Code—Steel* (herein referred to as AWS D1.1)

2005 AWS *D1.8 Structural Welding Code—Seismic Supplement* (herein referred to as AWS D1.8)

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)*

2008 ASTM F2280 *Standard Specification for "Twist-Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 150 ksi Minimum Tensile Strength*

CHAPTER 2

DESIGN REQUIREMENTS

2.1. Special and Intermediate Moment Frame Connection Types

Replace this section and Table 2.1 with the following:

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
Bolted flange plate (BFP)	Chapter 7	SMF, IMF
Welded unreinforced flange-welded web (WUF–W)	Chapter 8	SMF, IMF
Kaiser bolted bracket (KBB)	Chapter 9	SMF, IMF

2.4.4. Beam Flange Continuity Plates

Add the following after Exception (3):

(4) For the Kaiser bolted bracket connection, the provisions of Chapter 9 shall apply. When continuity plates are required by Chapter 9, thickness and detailing shall be in accordance with the provisions of Section 2.4.4.

CHAPTER 5

REDUCED BEAM SECTION (RBS) MOMENT CONNECTION

5.3.2.   Column Limitations

Replace this section with the following:

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) maximum. 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 of Section 9.4 or 10.4 of the AISC *Seismic Provisions,* as appropriate.

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

5.8.   Design Procedure

Add the following User Note to <u>Step 8</u>:

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

CHAPTER 6

BOLTED UNSTIFFENED AND STIFFENED EXTENDED END-PLATE MOMENT CONNECTIONS

6.2.   Systems

Replace this section with the following:

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

Exception: End-plate moment connections in SMF systems with *concrete structural slabs* are prequalified only if:

(1)   in addition to the limitations of Table 6.1 the nominal beam depth is not less than 24 in. (610 mm);

(2)   there are no shear connectors within 1.5 times the beam depth from the face of the connected column flange; and

(3)   the concrete structural slab is kept at least 1 in. (25 mm) from both sides of both column flanges. It is permitted to place compressible material in the gap between the column flanges and the concrete structural slab.

6.3.   Prequalification Limits

Replace this section and Table 6.1 with the following:

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					
	Four-Bolt Unstiffened (4E)		**Four-Bolt Stiffened (4ES)**		**Eight-Bolt Stiffened (8ES)**	
Parameter	**Maximum, in. (mm)**	**Minimum, in. (mm)**	**Maximum, in. (mm)**	**Minimum, in. (mm)**	**Maximum, in. (mm)**	**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)	7 (178)	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)	1⅝ (41)
p_b	–	–	–	–	3¾ (95)	3½ (89)
d	55 (1400)	13¾ (349)	24 (610)	13¾ (349)	36 (914)	18 (457)
t_{bf}	¾ (19)	⅜ (10)	¾ (19)	⅜ (10)	1 (25)	9⁄16 (14)
b_{bf}	9¼ (235)	6 (152)	9 (229)	6 (152)	12¼ (311)	7½ (190)

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)

6.10.   Design Procedure

Add the following User Note to (1):

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

Add the following new chapter:

CHAPTER 7

BOLTED FLANGE PLATE (BFP) MOMENT CONNECTION

7.1.   General

Bolted flange plate (BFP) moment connections utilize plates welded to column flanges and bolted to beam flanges. The top and bottom plates must be identical. Flange plates are welded to the column flange using *complete joint penetration (CJP) groove welds* and beam flange connections are made with high strength bolts. The beam web is connected to the column flange using a bolted shear tab with bolts in short-slotted holes. A detail for this connection type is shown in Figure 7.1. Initial yielding and plastic hinge formation are intended to occur in the beam in the region near the end of the flange plates.
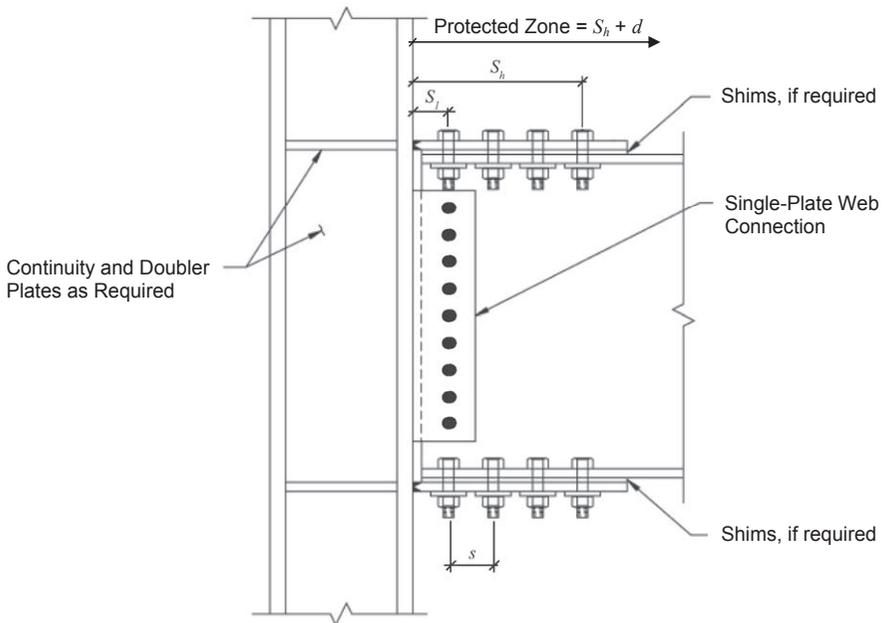
Fig. 7.1 Bolted flange plate moment connection.

7.2. Systems

Bolted flange plate connections are prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limitations of these provisions.

Exception: Bolted flange plate connections in SMF systems with *concrete structural slabs* are only prequalified if the concrete structural slab is kept at least 1 in. (25 mm) from both sides of both column flanges. It is permissible to place compressible material in the gap between the column flanges and the concrete structural slab.

7.3. Prequalification Limits

7.3.1. Beam Limitations

Beams shall satisfy the following limitations:

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

(2) Beam depth is limited to a maximum of 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 a maximum of 150 lb/ft (224 kg/m).

(4) Beam flange thickness is limited to a maximum of 1 in. (25 mm).

(5) The clear span-to-depth ratio of the beam is limited to 9 or greater for SMF and 7 or greater for IMF systems.

(6) Width-thickness ratios for the flanges and web of the beam shall conform to the limits of Section 9.4 or 10.4 of the AISC *Seismic Provisions,* as appropriate.

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

Lateral bracing of beams shall conform to Section 9.8 or 10.8 for SMF or IMF, as appropriate, in the AISC *Seismic Provisions.* To satisfy the requirement of Section 9.8 and 10.8 of the AISC *Seismic Provisions* for lateral bracing at plastic hinges, supplemental lateral bracing shall be provided at both the top and bottom beam flanges, and shall be located a distance of d to $1.5d$ from the bolt farthest from the face of the column. No attachment of lateral bracing shall be made within the protected zone.

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

(8) The protected zone consists of the flange plates and the portion of the beam between the face of the column and a distance one-beam depth, d, beyond the bolt farthest from the face of the column.

7.3.2. Column Limitations

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

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

(3) Rolled shaped column depth shall be limited to W36 (W920) maximum when a concrete structural slab is provided. In the absence of a concrete structural slab, rolled shape column depth is limited to W14 (W360) maximum. 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 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 of Section 9.4 or 10.4 of the AISC *Seismic Provisions*, as appropriate.

(7) Lateral bracing of columns shall conform to Section 9.7 or 10.7 of the AISC *Seismic Provisions,* as appropriate.

7.4. Beam-Column Relationship Limitations

(1) Panel zones shall conform to the requirements of Section 9.3 or 10.3 of the AISC *Seismic Provisions,* as appropriate.

(2) The column-beam moment ratio shall conform to the requirements of Section 9.6 or 10.6 of the AISC *Seismic Provisions,* as appropriate.

7.5. Connection Detailing

7.5.1. Plate Material Specifications

All connection plates shall conform to one of the following specifications: ASTM A36/A36M or A572/A572M Grade 50 (345).

7.5.2. Beam Flange Plate Welds

Flange plates shall be connected to the column flange using CJP groove welds and shall be considered demand critical. *Backing,* if used, shall be removed. The *root* pass shall be *backgouged* to sound weld metal and back welded.

7.5.3. Single-Plate Shear Connection Welds

The single-plate shear connection shall be welded to the column flange. The single-plate to column-flange connection shall consist of CJP groove welds, two-sided partial-joint-penetration (PJP) groove welds, or two-sided fillet welds.

7.5.4. Bolt Requirements

Bolts shall be arranged symmetrically about the axes of the beam and shall be limited to two bolts per row in the flange plate connections. The length of the bolt group shall not exceed the depth of the beam. Standard holes shall be used in beam flanges. Holes in flange plates shall be standard or oversized holes. Bolt holes in beam flanges and in flange plates shall be made by drilling or by sub-punching and reaming. Punched holes are not permitted.

> **User Note**: Although standard holes are permitted in the flange plate, their use will likely result in field modifications to accommodate erection tolerances.

Bolts in the flange plates shall be ASTM A490 or A490M or ASTM F2280 assemblies. Threads shall be excluded from the shear plane. Bolt diameter is limited to 1⅛ in. (28 mm) maximum.

7.5.5. Flange Plate Shims

Shims with a maximum overall thickness of ¼ in. (6 mm) may be used between the flange plate and beam flange as shown in Figure 7.1. Shims, if required, may be finger shims or may be made with drilled or punched holes.

7.6 Design Procedure

<u>Step 1</u> – Compute the probable maximum moment at the beam plastic hinge using the requirements of Section 2.4.3.

$$M_{pr} = C_{pr}R_yF_yZ_e \tag{7.6-1}$$

<u>Step 2</u> – Compute the maximum bolt diameter preventing beam flange tensile rupture.

For standard holes with two bolts per row:

$$d_b \le \frac{b_f}{2}\left(1 - \frac{R_yF_y}{R_tF_u}\right) - \tfrac{1}{8} \text{ in.} \tag{7.6-2}$$

$$d_b \le \frac{b_f}{2}\left(1 - \frac{R_yF_y}{R_tF_u}\right) - 3 \text{ mm} \qquad \text{(S.I.) (7.6-2M)}$$

<u>Step 3</u> – Considering bolt shear and bolt bearing, determine the controlling nominal shear strength per bolt.

$$r_n = \min \begin{cases} 1.1F_{nv}A_b \\ 2.4F_{ub}d_bt_f \\ 2.4F_{up}d_bt_p \end{cases} \tag{7.6-3}$$

where

A_b = nominal unthreaded body area of bolt, in.^{2} (mm^2)
D_b = nominal bolt diameter, in. (mm)
F_{nv} = nominal shear stress from AISC *Specification* Table J3.2, ksi (MPa)
F_{uv} = specified minimum tensile strength of beam material, ksi (MPa)
F_{up} = specified minimum tensile strength of plate material, ksi (MPa)
t_f = beam flange thickness, in. (mm)
t_p = flange plate thickness, in. (mm)

<u>Step 4</u> – Select a trial number of bolts.

> **User Note:** The following equation may be a useful way of estimating the trial number of bolts.
>
> $$n \geq \frac{1.25 M_{pr}}{\phi_n r_n \left(d + t_p \right)} \qquad (7.6\text{-}4)$$
>
> where
>
> n = number of bolts rounded to the next higher even number increment
> d = beam depth, in. (mm)

<u>Step 5</u> – Determine the beam *plastic hinge location,* as dimensioned from the face of the column.

$$S_h = S_1 + s\left(\frac{n}{2} - 1 \right) \qquad (7.6\text{-}5)$$

where

S_1 = distance from the face of the column to the nearest row of bolts, in. (mm)
s = bolt spacing between rows, in. (mm)

The bolt spacing between rows, s, and the edge distance shall be sufficiently large to ensure that L_c, as defined in AISC *Specification* Section J3.10, is greater than or equal to $2d_b$.

<u>Step 6</u> – Compute the shear force at the beam plastic hinge location at each end of the beam.

The shear force at the hinge location, V_h, shall be determined by a free body diagram of the portion of the beam between the plastic hinge locations. This calculation shall assume the moment at the plastic hinge location is M_{pr} and shall include gravity loads acting on the beam based on the load combination $1.2D + f_1 L + 0.2S$.

where

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

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

Step 7 – Calculate the moment expected at the face of the column flange.

$$M_f = M_{pr} + V_h S_h \tag{7.6-6}$$

where

V_h = larger of the two values of shear force at the beam hinge location at each end of the beam, kips (N)

Step 8 – Compute the force in the flange plate due to M_f.

$$F_{pr} = \frac{M_f}{\left(d + t_p\right)} \tag{7.6-7}$$

where

d = beam depth, in. (mm)

t_p = flange plate thickness, in. (mm)

Step 9 – Confirm that the number of bolts selected in Step 4 is adequate.

$$n \geq \frac{F_{pr}}{\phi_n r_n} \tag{7.6-8}$$

Step 10 – Determine the required thickness of the flange plate.

$$t_p \geq \frac{F_{pr}}{\phi_d F_y b_{fp}} \tag{7.6-9}$$

where

F_y = specified minimum yield stress of flange plate, ksi (MPa)

b_{fp} = width of flange plate, in. (mm)

Step 11 – Check the flange plate for tensile rupture.

$$F_{pr} \leq \phi_n R_n \tag{7.6-10}$$

Where R_n is defined in AISC *Specification* Section J4.1.

Step 12 – Check the beam flange for block shear.

$$F_{pr} \leq \phi_n R_n \tag{7.6-11}$$

where R_n is as defined in AISC *Specification* Section J4.3.

Step 13 – Check the flange plate for compression buckling.

$$F_{pr} \leq \phi_n R_n \tag{7.6-12}$$

where R_n is defined in AISC *Specification* Section J4.4.

> **User Note:** When checking compression buckling of the flange plate, the effective length, *KL*, may be taken as $0.65S_1$.

<u>Step 14</u> – Determine the required shear strength, V_u, of the beam and the beam web-to-column connection from:

$$V_u = \frac{2M_{pr}}{L'} + V_{gravity} \qquad (7.6\text{-}13)$$

where
   V_u = required shear strength of beam and beam web to column connection, kips (N)
   L' = distance between hinge locations, 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

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

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

<u>Step 15</u> – Design a single-plate shear connection for the required strength, V_u, calculated in Step 14 and located at the face of the column, meeting the requirements of the AISC *Specification*.

<u>Step 16</u> – Check the continuity plate requirements according to Chapter 2.

<u>Step 17</u> – Check the column panel zone according to Section 7.4.

The required shear strength of the panel zone shall be determined from the summation of the moments at the column faces as determined by projecting moments equal to $R_yF_yZ_e$ at the plastic hinge points to the column faces. For *d*, add twice the thickness of the flange plate to the beam depth.

<u>Step 18</u> – Check the column beam moment ratio according to Section 7.4.

Add the following new chapter:

CHAPTER 8

WELDED UNREINFORCED FLANGE–WELDED WEB (WUF–W) MOMENT CONNECTION

8.1   General

In the welded unreinforced flange–welded web (WUF–W) moment connection, inelastic rotation is developed primarily by yielding of the beam in the region adjacent to the face of the column. Connection fracture is controlled through special detailing requirements associated with the welds joining the beam flanges to the column flange, the welds joining the beam web to the column flange, and the shape and finish of the weld access holes. An overall view of the connection is shown in Figure 8.1.
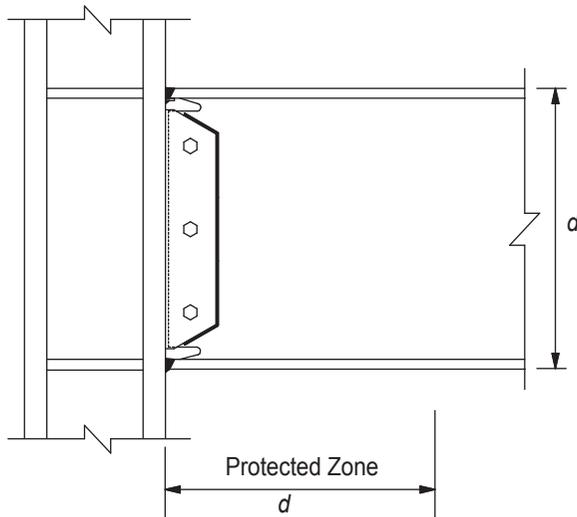
Fig. 8.1. WUF–W moment connection.

8.2.   Systems

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

8.3.   Prequalification Limits

8.3.1.   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 a maximum of 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 a maximum of 150 lb/ft (224 kg/m).

(4)   Beam flange thickness is limited to a maximum of 1 in. (25 mm).

(5)   The clear span-to-depth ratio of the beam is 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* Section 9.4 or Section 10.4, as applicable.

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

   Lateral bracing of beams shall conform to Section 9.8 or 10.8 for SMF or IMF, as applicable, in the AISC *Seismic Provisions*. To satisfy the requirement of Sections 9.8 and 10.8 of the AISC *Seismic Provisions* for lateral bracing at plastic hinges, supplemental lateral bracing shall be provided at both the top and bottom beam flanges, and shall be located at a distance of d to $1.5d$ from the face of the column. No attachment of lateral bracing shall be made to the beam in the region extending from the face of the column to a distance d from the face of the column.

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

(8)   The protected zone consists of the portion of beam between the face of the column and a distance one beam depth, d, from the face of the column.

8.3.2   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 a maximum of 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 Section 9.4 or 10.4, as applicable, of the AISC *Seismic Provisions*.

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

8.4. Beam-Column Relationship Limitations

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

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

(a) For SMF systems, the column-beam moment ratio shall conform to the requirements of Section 9.6 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 Step 1 in Section 8.7 and M_v is the additional moment due to shear amplification from the plastic hinge to the centerline of the column. M_v is permitted to be computed as $V_h(d_c/2)$, where V_h is the shear at the plastic hinge computed per Step 3 of Section 8.7, 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*.

8.5. Beam Flange to Column Flange Welds

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 AWS D1.8 Section 6.9.1.2. Weld access hole quality requirements shall conform to the requirements of AWS D1.8 Section 6.9.2.

8.6.　Beam Web to Column Connection Limitations

The overall details of the beam-web-to-column-flange connection are shown in Figure 8.2. Singleplate shear connection shall conform to the requirements shown in Figure 8.2. Beam-web-to-column-flange connections shall satisfy the following limitations:

(1) A single-plate shear connection shall be provided with a thickness equal at least to that of the beam web. The height of the single plate shall allow a ¼-in. (6-mm)-minimum and ½-in. (13-mm)-maximum overlap with the weld access hole at the top and bottom. The width shall extend 2 in. (50 mm) minimum beyond the end of the weld access hole.

(2) The single-plate shear connection shall be welded to the column flange. The design shear strength of the welds shall be at least $h_p t_p(0.6R_y F_{yp})$, where h_p is defined as the height of the plate, as shown in Figure 8.2, and t_p is the thickness of the plate.

(3) Connect the single plate to the beam web with fillet welds, as shown in Figure 8.2. The size of the fillet weld shall equal the thickness of the single plate minus $\frac{1}{16}$ in (2 mm). The fillet welds shall extend along the sloped top and bottom portions of the single plate, and along the full single plate height, as shown in Figure 8.2. The fillet welds on the sloped top and bottom portions of the single plate shall be terminated at least ½ in. (13 mm) but not more than 1 in. (25 mm) from the edge of the weld access hole, as shown in Figure 8.3.

(4) Erection bolts in standard holes or horizontal short slots are permitted as needed for erection loads and safety.

CJP beam web to column flange weld

single plate to column flange weld

Erection bolts in standard holes or horizontal short slots are permitted as needed for erection loads and safety

single plate to beam web weld

h_p
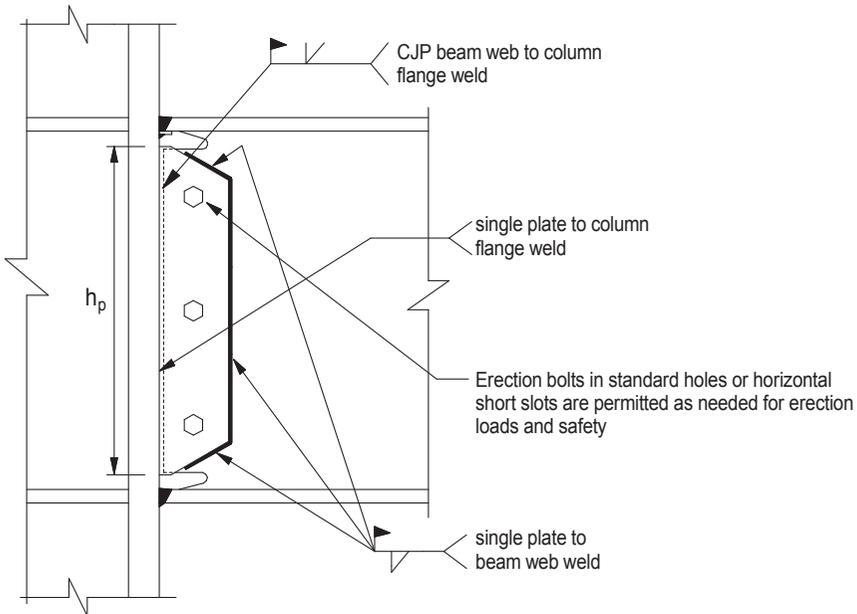
Fig. 8.2. General details of beam-web-to-column-flange connection.

(5) A CJP groove weld shall be provided between the beam web and the column flange. This weld shall be provided over the full length of the web between weld access holes, and shall conform to the requirements for demand critical welds in Section 7.3 and Appendix W of the AISC *Seismic Provisions*. *Weld tab*s are not required. Weld tabs, if used, must be removed after welding in accordance with the requirements of Section 3.4. When weld tabs are not used, the use of cascaded ends within the weld groove shall be permitted at a maximum angle of 45°. Nondestructive testing (NDT) of cascaded weld ends need not be performed.
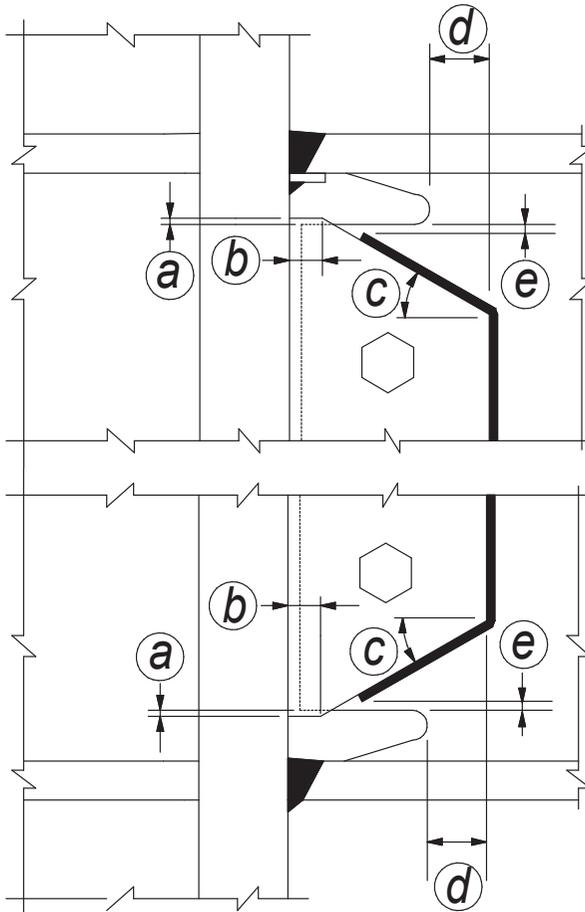
Notes:
a. ¼ in. (3 mm) minimum, ½ in. (6 mm) maximum.
b. 1 in. (25 mm) minimum
c. 30° (±10°)
d. 2 in. (50 mm) minimum
e. ½-in. (6 mm) minimum distance, 1-in. (25 mm) maximum distance, from end of fillet weld to edge of weld access hole.

Fig. 8.3. Details at top and bottom of single-plate shear connection.

8.7. Connection Design Procedure

Step 1 – Compute the *probable maximum moment at plastic hinge*, M_{pr}, in accordance with Section 2.4.3. The value of Z_e shall be taken as equal to Z_x of the beam section and the value of C_{pr} shall be taken as equal to 1.4.

Step 2 – The plastic hinge location shall be taken to be at the face of the column. That is, $S_h = 0$.

Step 3 – Compute the shear force, V_h, at the plastic hinge location at each end of the beam.

The shear force at the plastic hinge locations shall be determined by a free body diagram of the portion of the beam between the plastic hinges. This calculation shall assume the moment at each plastic hinge is M_{pr} and shall include gravity loads acting on the beam between the hinges based on the load combination $1.2D + f_1 L + 0.2S$.

> **User Note:** The load combination of $1.2D + f_1 L + 0.2S$ is in conformance with SEI/ASCE 7. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 when the roof configuration is such that it does not shed snow off of the structure.

Step 4 – Check column-beam moment ratios per Section 8.4(2).

Step 5 – Check the panel zone per Section 8.4(1). For SMF, the required shear strength of the panel zone, per Section 9.3a of the AISC *Seismic Provisions*, shall be determined from the summation of the probable maximum moments at the face of the column. The probable maximum moment at the face of the column shall be taken as M_{pr}, computed per Step 1.

Step 6 – Check beam design shear strength:

The required shear strength, V_u, of the beam shall be taken equal to the larger of the two values of V_h computed at each end of the beam in Step 3.

Step 7 – Check column continuity plate requirements per Section 2.4.4.

Add the following new chapter:

CHAPTER 9

KAISER BOLTED BRACKET (KBB) MOMENT CONNECTION

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

9.1.   General

In a Kaiser bolted bracket (KBB) moment connection, a cast high-strength steel bracket is fastened to each beam flange and bolted to the column flange as shown in Figure 9.1. The bracket attachment to the beam flange is permitted to be either welded (Figure 9.1a) or bolted (Figure 9.1b). If welded to the beam flange, the five W-series bracket configurations available are shown in Figure 9.2. If bolted to the beam flange, the two B-series bracket configurations available are shown in Figure 9.3. The bracket configuration is proportioned to develop the probable maximum moment capacity of the connected beam. Yielding and plastic hinge formation are intended to occur primarily in the beam at the end of the bracket away from the column face.

9.2.   Systems

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

Exception: KBB SMF systems with *concrete structural slabs* are prequalified only if the concrete structural slab is kept at least 1 in. (25 mm) from both sides of both column flanges and the vertical flange of the bracket. It is permitted to place compressible material in the gap in this location.
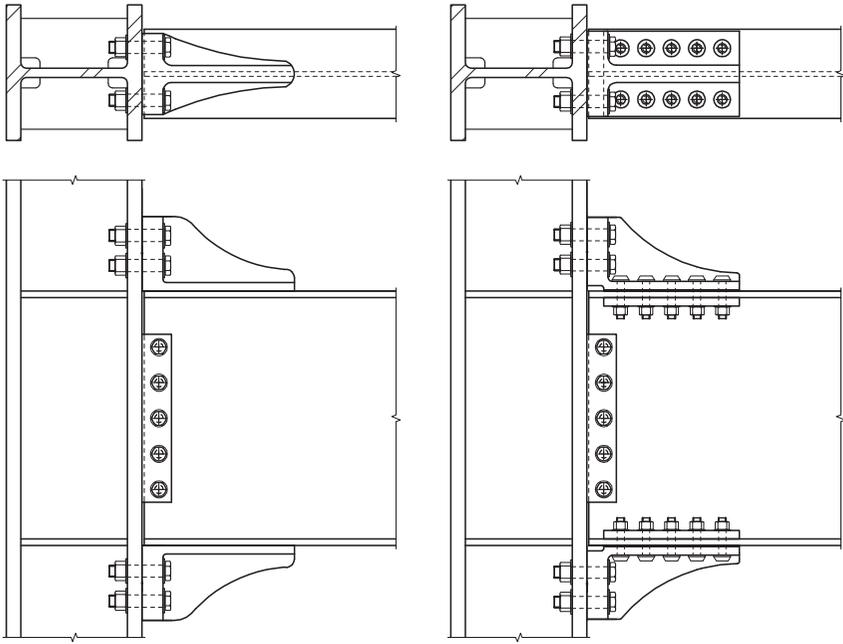
(a) W-series connection.

(b) B-series connection.

Fig. 9.1. Kaiser bolted bracket connection.
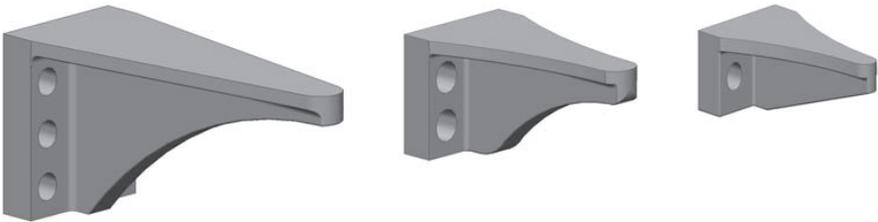
(a) Six-column bolt, W1.0. (b) Four-column bolt, W2.0 and W2.1. (c) Two-column bolt, W3.0 and W3.1.

Fig. 9.2. Kaiser bolted bracket W-series configurations.
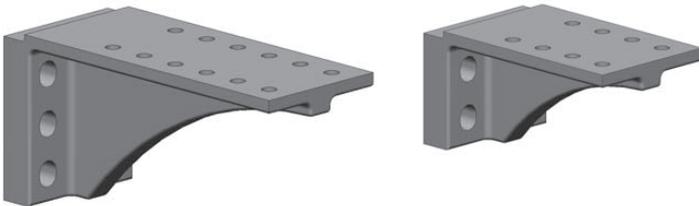
(a) Six-column bolt, B1.0.

(b) Four-column bolt, B2.1.

Fig. 9.3 Kaiser bolted bracket B-series configurations.

9.3. Prequalification Limits

9.3.1. 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 a maximum of W33 (W840) 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 a maximum of 130 lb/ft (195 kg/m).

(4) Beam flange thickness is limited to a maximum of 1 in. (25 mm).

(5) Beam flange width shall be at least 6 in. (152 mm) for W-series brackets and at least 10 in. (250 mm) for B-series brackets.

(6) The clear span-to-depth ratio of the beam shall be limited to 9 or greater for both SMF and IMF systems.

(7) Width-thickness ratios for the flanges and web of the beam shall conform to the limits of Section 9.4 or Section 10.4, as applicable, of the AISC *Seismic Provisions*.

(8) 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 expected plastic hinge in conformance with Section 9.8.

When supplemental lateral bracing is provided, attachment of supplemental lateral bracing to the beam shall be located at a distance d to $1.5d$ from the end of the bracket 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 the expected hinge location.

(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 at maximum of 12 in. (305 mm) on center, supplemental top and bottom flange bracing at the expected hinge is not required.

(9) The protected zone consists of the portion of beam between the face of the column and one beam depth, d, beyond the tip of the bracket.

9.3.2. Column Limitations

The 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) The column flange width shall be at least 12 in. (305 mm).

(4) Rolled shape column depth shall be limited to W36 (W920) maximum when a concrete structural slab is provided. In the absence of a concrete structural slab, rolled shape column depth is limited to W14 (W360) maximum. 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 16 in. (406 mm). Boxed wide-flange columns shall not have a width or depth exceeding 16 in. (406 mm) if participating in orthogonal moment frames.

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

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

(7) Width-thickness ratios for the flanges and web of columns shall conform to the limits in Section 9.4 or Section 10.4, as applicable, of the AISC *Seismic Provisions*.

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

9.3.3. Bracket Limitations

The high strength cast-steel brackets shall satisfy the following limitations:

(1) Bracket castings shall conform to the requirements of Appendix A.

(2) Bracket configuration and proportions shall conform to Section 9.8.

(3) Holes in the bracket for the column bolts shall be vertical short-slotted holes. Holes for the beam bolts shall be standard holes.

(4) Material thickness, edge distance and end distance shall have a tolerance of $\pm\frac{1}{16}$ in. (2 mm). Hole location shall have a tolerance of $\pm\frac{1}{16}$ in. (2 mm). The overall dimensions of the bracket shall have a tolerance of $\pm\frac{1}{8}$ in. (3 mm).

9.4. Beam-Column Relationship Limitations

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

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

(a) For SMF systems, the column-beam moment ratio shall conform to the requirements of Section 9.6 of the AISC *Seismic Provisions*.

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

9.5.   Bracket-to-Column Flange Limitations

Bracket-to-column-flange connections shall satisfy the following limitations:

(1) Column flange fasteners shall be pretensioned ASTM A490, A490M, A354 Grade BD bolts, or A354 Grade BD threaded rods, and shall conform to the requirements of Chapter 4.

(2) Column flange bolt holes shall be ⅛ in. (3 mm) larger than the nominal bolt diameter. Bolt holes shall be drilled or subpunched and reamed. Punched holes are not permitted.

(3) The use of finger shims on either or both sides at the top and/or bottom of the bracket connection is permitted, subject to the limitations of RCSC *Specification* Section 5.1.

(4) When bolted to a box column, a steel washer plate shall be inserted between the box column and the bracket on both faces of the column. The washer plate shall be ASTM A572/A572M Grade 50 (345) or better and shall be designed to transfer the bolt forces to the outside edges of the column. Where required, the vertical plate depth may exceed the contact surface area by 4 in. (102 mm). The plate thickness shall not exceed 3 in. (76 mm). The fasteners shall pass through the interior of the box column and be anchored on the opposite face. The opposite face shall also have a steel washer plate.

(5) When connected to the orthogonal face of a box column concurrent with a connection on the primary column face, a 1¾-in. (44-mm) steel spacer plate shall be inserted between the beam flanges and the brackets of the orthogonal connection. The spacer plate shall be made of any of the structural steel materials included in Section A3 of the AISC *Specification* and shall be the approximate width and length matching that of the bracket contact surface area.

9.6.   Bracket-to-Beam-Flange Connection Limitations

Bracket-to-beam-flange connections shall satisfy the following limitations:

(1) When welded to the beam flange, the bracket shall be connected using fillet welds. Bracket welds shall conform to the requirements for demand critical welds in Section 7.3 of the AISC *Seismic Provisions* and AWS D1.8, and to the requirements of AWS D1.1. The weld procedure specification (WPS) for the fillet weld joining the bracket to the beam flange shall be qualified with the casting material. Welds shall not be started or stopped within 2 in. (51 mm) of the bracket tip and shall be continuous around the tip.

(2) When bolted to the beam flange, fasteners shall be pretensioned ASTM A490 or A490M bolts with threads excluded from the shear plane and shall conform to the requirements of Chapter 4.

TABLE 9.1. Kaiser Bolted Bracket Proportions						
Bracket Designation	Bracket Length, L_{bb}, in. (mm)	Bracket Height, h_{bb}, in. (mm)	Bracket Width, b_{bb}, in. (mm)	Number of Column Bolts, n_{cb}	Column Bolt Gage, g, in. (mm)	Column Bolt Diameter, in. (mm)
W3.0	16 (406)	5½ (140)	9 (229)	2	5½ (140)	1⅜ (35)
W3.1	16 (406)	5½ (140)	9 (229)	2	5½ (140)	1½ (38)
W2.0	16 (406)	8¾ (222)	9½ (241)	4	6 (152)	1⅜ (35)
W2.1	18 (457)	8¾ (222)	9½ (241)	4	6½ (165)	1½ (38)
W1.0	25½ (648)	12 (305)	9½ (241)	6	6½ (165)	1½ (38)
B2.1	18 (457)	8¾ (222)	10 (254)	4	6½ (165)	1½ (38)
B1.0	25½ (648)	12 (305)	10 (254)	6	6½ (165)	1½ (38)

(3) Beam flange bolt holes shall be 1⁵⁄₃₂ in. (29 mm) and shall be drilled using the bracket as a template. Punched holes are not permitted.

(4) When bolted to the beam flange, a ⅛-in. (3-mm)-thick brass washer plate with an approximate width and length matching that of the bracket contact surface area shall be placed between the beam flange and the bracket. The brass shall be a half-hard tempered ASTM B19 or B36 sheet.

(5) When bolted to the beam flange, a 1-in. (25-mm)-thick by 4-in. (102-mm)-wide ASTM A572/A572M Grade 50 (345) plate washer shall be used on the opposite side of the connected beam flange.

9.7.   Beam-Web-to-Column Connection Limitations

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

(1) The required shear strength of the beam web connection shall be determined according to Section 9.9.

(2) The single-plate shear connection shall be connected to the column flange using a two-sided fillet weld, two-sided PJP groove weld or CJP groove weld.

9.8.   Connection Detailing

If welded to the beam flange, Figure 9.4 shows the connection detailing for the W-series bracket configurations. If bolted to the beam flange, Figure 9.5 shows the connection detailing for the B-series bracket configurations. Table 9.1 summarizes the KBB proportions and column bolt parameters. Table 9.2 summarizes the design proportions for the W-series bracket configuration. Table 9.3 summarizes the design proportions for the B-series bracket configurations.

	TABLE 9.2. W-Series Bracket Design Proportions					
Bracket Designation	Column Bolt Edge Distance, d_e, in. (mm)	Column Bolt Pitch, p_b, in. (mm)	Bracket Stiffener Thickness, t_s, in. (mm)	Bracket Stiffener Radius, r_v, in. (mm)	Bracket Horizontal Radius, r_h, in. (mm)	Minimum Fillet Weld Size, in. (mm)
W3.0	2½ (64)	n.a.	1 (25)	n.a.	28 (711)	½ (13)
W3.1	2½ (64)	n.a.	1 (25)	n.a.	28 (711)	⅝ (16)
W2.0	2¼ (57)	3½ (89)	2 (51)	12 (305)	28 (711)	¾ (19)
W2.1	2¼ (57)	3½ (89)	2 (51)	16 (406)	38 (965)	⅞ (22)
W1.0	2 (51)	3½ (89)	2 (51)	28 (711)	n.a.	⅞ (22)

Fig. 9.4. W-series connection detailing.

TABLE 9.3. B-Series Bracket Design Proportions						
Bracket Designation	Colum Bolt Edge Distance, d_e, in. (mm)	Column Bolt Pitch, p_b, in. (mm)	Bracket Stiffener Thickness, t_s, in. (mm)	Bracket Stiffener Radius, r_v, in. (mm)	Number of Beam Bolts, n_{bb}	Beam Bolt Diameter, in. (mm)
B2.1	2 (51)	3½ (89)	2 (51)	16 (406)	8 or 10	1⅛ (29)
B1.0	2 (51)	3½ (89)	2 (51)	28 (711)	12	1⅛ (29)
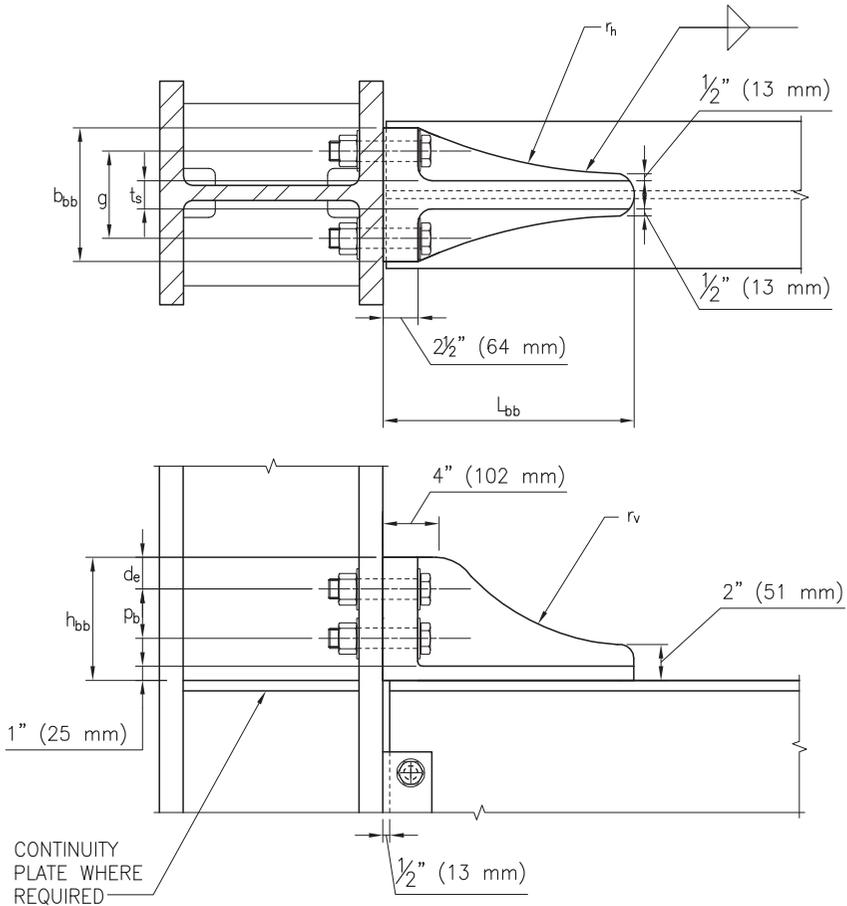
Fig. 9.5. B-series connection detailing.

9.9.   Design Procedure

<u>STEP 1</u> – Select beam and column elements according to Section 9.3.

<u>STEP 2</u> – Compute the probable maximum moment, M_{pr}, at the location of the plastic hinge according to Section 2.4.3.

<u>STEP 3</u> – Select a trial bracket from Table 9.1.

<u>STEP 4</u> – Compute the shear force at the beam hinge location at each end of the beam. The shear force at the hinge location, V_h, shall be determined by a free-body diagram of the portion of the beam between the hinge locations. This calculation shall assume the moment at the hinge location is M_{pr} and shall include gravity loads acting on the beam based on the load combination $1.2D + f_1L + 0.2S$, kips (N).

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

where

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

<u>STEP 5</u> – Compute the probable maximum moment at the face of the column:

$$M_f = M_{pr} + V_hS_h \qquad (9.9\text{-}1)$$

where

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

V_h = larger of the two values of shear force at the beam hinge location at each end of the beam, kips (N)

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

  = L_{bb} per Table 9.1, in. (mm)

<u>STEP 6</u> – The following relationship shall be satisfied for the bracket column bolt tensile strength:

$$\frac{r_{ut}}{\phi_n F_n A_b} < 1.0 \qquad (9.9\text{-}2)$$

where

$$r_{ut} = \frac{M_f}{d_{eff}n_{cb}} \qquad (9.9\text{-}3)$$

F_n = nominal tension bolt strength per unit area, ksi (MPa)

  = 113 ksi (780 MPa) for ASTM A490/A490M bolts

A_b = bolt nominal cross-sectional area, in.^{2} (mm^2)

d_{eff} = effective beam depth calculated as the centroidal distance between bolt groups in the upper and lower brackets, in. (mm)

n_{cb} = number of column bolts per Table 9.1

<u>STEP 7</u> – Determine the minimum column flange width to prevent flange tensile rupture:

$$b_{cf} \geq \frac{2\left[d_b + \text{⅛ in.}\right]}{\left(1 - \dfrac{R_y F_{yf}}{R_t F_{uf}}\right)}$$ (9.9-4)

$$b_{cf} \geq \frac{2\left[d_b + 3 \text{ mm}\right]}{\left(1 - \dfrac{R_y F_{yf}}{R_t F_{uf}}\right)} \qquad \text{(S.I.)} \qquad \text{(9.9-4M)}$$

where

b_{cf} = column flange width, in. (mm)

d_b = column flange bolt diameter, in. (mm)

F_{yf} = specified minimum yield stress of the flange material, ksi (MPa)

F_{uf} = specified minimum tensile strength of the flange material, ksi (MPa)

R_y = ratio of expected yield stress to the specified minimum yield stress for the flange material

R_t = ratio of expected tensile strength to the specified minimum tensile strength for the flange material

<u>STEP 8</u> – Check the minimum column flange thickness to eliminate prying action:

$$t_{cf} \geq \sqrt{\frac{4.44 r_{ut} b'}{\phi_d p F_y}}$$ (9.9-5)

where

$$b' = 0.5 \left(g - k_1 - 0.5 t_{cw} - d_b\right)$$ (9.9-6)

t_{cf} = minimum column flange thickness required to eliminate prying action, in. (mm)

p = perpendicular tributary length per bolt, in. (mm)

  = 3.5 in. (89 mm) for W1.0/B1.0

  = 5.0 in. (127 mm) for all other brackets

g = column bolt gage, in. (mm)

k_1 = column web centerline distance to the flange toe of the fillet per Table 1-1 of the AISC *Steel Construction Manual,* in. (mm)

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

If the selected column flange thickness is less than that required to eliminate prying action, select a column with a satisfactory flange thickness or include the bolt prying force in Equation 9.9-2 per Part 9 of the AISC *Steel Construction Manual*.

<u>STEP 9</u> – The column flange thickness shall satisfy the following requirement:

$$t_{cf} > \sqrt{\frac{M_f}{\phi_d F_{yf} d_{eff} Y_m}} \tag{9.9-7}$$

where

Y_m = simplified column flange yield line mechanism parameter

= 5.9 for W3.0/W3.0

= 6.5 for W2.0/W2.1/W2.1

= 7.5 for W1.0/W1.0

t_{cf} = minimum column flange thickness required to eliminate continuity plates, in. (mm)

<u>STEP 10</u> – Continuity Plate Requirements

Continuity plates shall be provided for column sections deeper than W14. For W14 and shallower sections, continuity plates are not required.

<u>STEP 11</u> – If the bracket is welded to the beam flange proceed to Step 14; otherwise, determine the minimum beam flange width to prevent beam flange tensile rupture:

$$b_{bf} \geq \frac{2\left[d_b + \frac{1}{32} \text{ in.}\right]}{\left(1 - \frac{R_y F_{yf}}{R_t F_{uf}}\right)} \tag{9.9-8}$$

$$b_{bf} \geq \frac{2\left[d_b + 1 \text{ mm}\right]}{\left(1 - \frac{R_y F_{yf}}{R_t F_{uf}}\right)} \quad \text{(S.I.)} \tag{9.9-8M}$$

where

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

d_b = beam flange bolt diameter, in. (mm)

30

STEP 12 – The following relationship shall be satisfied for the beam bolt shear strength:

$$\frac{M_f}{1.1\phi_n F_n A_b d_{eff} n_{bb}} < 1.0 \qquad (9.9\text{-}9)$$

where

F_n = nominal shear bolt stress per unit area, ksi (MPa)

= 75 ksi (518 MPa) for ASTM A490/A490M bolts with threads excluded from the shear plane

n_{bb} = number of beam bolts per Table 9.3

STEP 13 – Check the beam flange for block shear per the following:

$$\frac{M_f}{d_{eff}} \leq \phi_n R_n \qquad (9.9\text{-}10)$$

where

R_n is as defined in Section J4.3 of the AISC *Specification.*

STEP 14 – If the bracket is bolted to the beam flange proceed to Step 15. Otherwise, the following relationship shall be satisfied for the fillet weld attachment of the bracket to the beam flange:

$$\frac{1.4 M_f}{\phi_n F_w d_{eff} l_w t_f} < 1.0 \qquad (9.9\text{-}11)$$

where

F_w = nominal weld design strength per the AISC *Specification*

= $0.60 F_{EXX}$

L_{bb} = bracket length per Table 9.3, in. (mm)

l = bracket overlap distance, in. (mm)

= 0 if $b_{bf} \geq b_{bb}$

= 5 in. (125 mm) if $b_{bf} < b_{bb}$

l_w = $2(L_{bb} - 2.5 \text{ in.} - l)$ (9.9-12)

l_w = $2(L_{bb} - 64 \text{ mm} - l)$ (S.I.) (9.9-12M)

l_w = length of available fillet weld, in. (mm)

t_f = minimum fillet weld thickness per Table 9.2, in. (mm)

<u>STEP 15</u> – Determine the required shear strength, V_u of the beam and beam web-to-column connection from:

$$V_u = (2M_{pr}/L') + V_{gravity} \qquad (9.9\text{-}13)$$

where

    L' = distance between the plastic hinges, in. (mm)

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

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

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

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

<u>STEP 16</u> – Design the beam web-to-column connection according to Section 9.7.

<u>STEP 17</u> – Check column panel zone according to Section 9.4. Substitute the effective depth, d_{eff}, of the beam and brackets for the beam deep, d.

<u>STEP 18</u> – Check column-beam moment ratio according to Section 9.4.

<u>STEP 19 (Supplemental)</u>—If the column is a box configuration, determine the size of the steel washer plate between the column flange and the bracket such that:

$$Z_x \geq \frac{M_f(b_{cf} - t_{cw} - g)}{4\phi_d F_y d_{eff}} \qquad (9.9\text{-}14)$$

where

    F_y = specified minimum yield stress of the washer material, ksi (MPa)

    Z_x = plastic section modulus of the washer plate, in.^{3} (mm^3)

    g = column bolt gage, in. (mm)

Add the following new appendix:

APPENDIX A

CASTING REQUIREMENTS

A.1. Cast Steel Grade

Cast steel grade shall be in accordance with ASTM A958 Grade SC8620 class 80/50.

A.2. Quality Control (QC)

A.2.1. Inspection and Nondestructive Testing Personnel

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

A.2.2. First Article Inspection (FAI) of Castings

The first article is defined as the first production casting made from a permanently mounted and rigged pattern. FAI shall be performed on the first casting produced from each pattern. The first article casting dimensions shall be measured and recorded. FAI includes visual inspection in accordance with Section A.2.3, nondestructive testing in accordance with Section A.2.4(2), tensile testing in accordance with Section A.2.6, and Charpy V-notch testing in accordance with Section A.2.7.

A.2.3. Visual Inspection of Castings

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

A.2.4. Nondestructive Testing (NDT) of Castings

(1) Procedures

Radiographic testing (RT) shall be performed by quality assurance (QA) according to the procedures prescribed in ASTM E446 and ASTM E186 with an acceptance Level III or better.

Ultrasonic testing (UT) shall be performed by QA according to the procedures prescribed by ASTM A609 Procedure A with an acceptance Level 3, or better.

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

(2) Required NDT

(a) First Article

RT and MT shall be performed on the first article casting.

(b) Production Castings

UT shall be performed on 100% of the castings.

MT shall be performed on 50% of the castings.

(c) Reduction of Percentage of UT

The UT rate is permitted to be reduced if approved by the engineer of record and the *authority having jurisdiction.* The UT rate may be reduced to 25%, provided the number of castings not conforming to Section A.2.4 (1) is demonstrated to be 5% or less. A sampling of at least 40 castings shall be made for such reduction evaluation. This reduction is not permitted for castings with weld repairs.

(d) Reduction of Percentage of MT

The MT rate is permitted to be reduced if approved by the engineer of record and the *authority having jurisdiction.* The MT rate may be reduced to 10%, provided the number of castings not conforming to Section A.2.4(1) is demonstrated to be 5% or less. A sampling of at least 20 castings shall be made for such reduction evaluation. This reduction is not permitted for castings with weld repairs.

A.2.5. Weld Repair Procedures

Castings with discontinuities that exceed the requirements of Section A.2.4(1) shall be weld repaired. Weld repair of castings shall be performed in accordance with ASTM A488/A488M. The same testing method that discovered the discontinuities shall be repeated on repaired castings to confirm the removal of all discontinuities that exceed the requirements of Section A.2.4(1).

A.2.6. Tensile Requirements

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

A.2.7. Charpy V-Notch (CVN) Requirements

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

A.2.8. Casting Identification

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

A.3. Manufacturer Documents

A.3.1. Submittal to Patent Holder

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

(1) Material chemical composition report.

(2) First article inspection report.

A.3.2. Submittal to Engineer of Record and Authority Having Jurisdiction

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

(1) Production inspection and NDT reports.

(2) Tensile and CVN test reports.

(3) Weld repair reports.

(4) Letter of approval by the patent holder of the manufacturer's FAI report.

Add the following new commentary:

C7. BOLTED FLANGE PLATE (BFP)
MOMENT CONNECTION

C7.1.   General

The bolted flange plate (BFP) connection is a field-bolted connection. The fundamental seismic behaviors expected with the BFP moment connection include:

(1) initial yielding of the beam at the last bolt away from the face of the column;

(2) slip of the flange plate bolts, which occurs at similar resistance levels to the initial yielding in the beam flange, but the slip does not contribute greatly to the total deformation capacity of the connection;

(3) secondary yielding in the column panel zone, which occurs as the expected moment capacity and as strain hardening of beam hinge occurs; and,

(4) limited yielding of the flange plate, which may occur at the maximum deformations.

This sequence of yielding has resulted in very large inelastic deformation capacity for the BFP moment connection, but the design procedure is somewhat more complex than some other prequalified connections.

The flange plates and web shear plate are shop welded to the column flange and field bolted to the beam flanges and web, respectively. ASTM A490 or A490M bolts with threads excluded from the shear plane are used for the beam flange connections because the higher shear strength of the A490 or A490M bolts reduces the number of bolts required and reduces the length of the flange plate. The shorter flange plates that are therefore possible reduce the seismic inelastic deformation demands on the connection and simplify the balance of the resistances required for different failure modes in the design procedure. Flange plate connections with A325 or A325M bolts may be possible, but will be significantly more difficult to accomplish because of the reduced bolt strength, greater number of bolts, and longer flange plates required. As a result, the connection is not prequalified for use with A325 or A325M bolts.

Prequalification of the BFP moment connection is based upon 20 BFP moment connection tests under cyclic inelastic deformation (FEMA 355D, 2000; Schneider and Teeraparbwong, 1999; Sato, Newall and Uang, 2007). Additional evidence supporting prequalification is derived from bolted T-stub connection tests (FEMA 355D, 2000; Swanson, Leon, and Smallridge, 2000), since the BFP moment connection shares many yield mechanisms, failure modes, and connection behaviors with the bolted T-stub connection. The tests were performed under

several deformation-controlled test protocols, but most use variations of the ATC24 (Krawinkler, 1992) or the SAC steel protocol (SAC/BD-00/10, 2000), which are both very similar to the prequalification test protocol of Appendix S of ANSI/AISC 341-05. The 20 BFP tests were performed on connections with beams ranging in depth from W8 to W36 sections, and the average total demonstrated ductility capacity exceeded 0.057 radian. Hence, the inelastic deformation capacity achieved with BFP moment connections is among the best achieved from seismic testing of moment frame connections. However, the design of the connection is relatively complex because numerous yield mechanisms and failure modes must be considered in the design process. Initial and primary yielding in the BFP moment connection is flexural yielding of the beam near the last row of bolts at the end of the flange plate. However, specimens with the greatest ductility achieve secondary yielding through shear yielding of the column panel zone and limited tensile yielding of the flange plate. Hence, a balanced design, which achieves some yielding from multiple yield mechanisms, is encouraged.

Most past tests have been conducted on specimens with single-sided connections, and the force-deflection behavior is somewhat pinched as shown in Figure C-7.1. Because plastic hinging at the end of the flange plate is the controlling yield mechanism, the expected plastic moment at this location dominates the connection design. The pinching is caused by a combination of bolt slip and the sequence of yielding and strain hardening encountered in the connection. Experiments have shown that the expected peak moment capacity at the plastic hinge
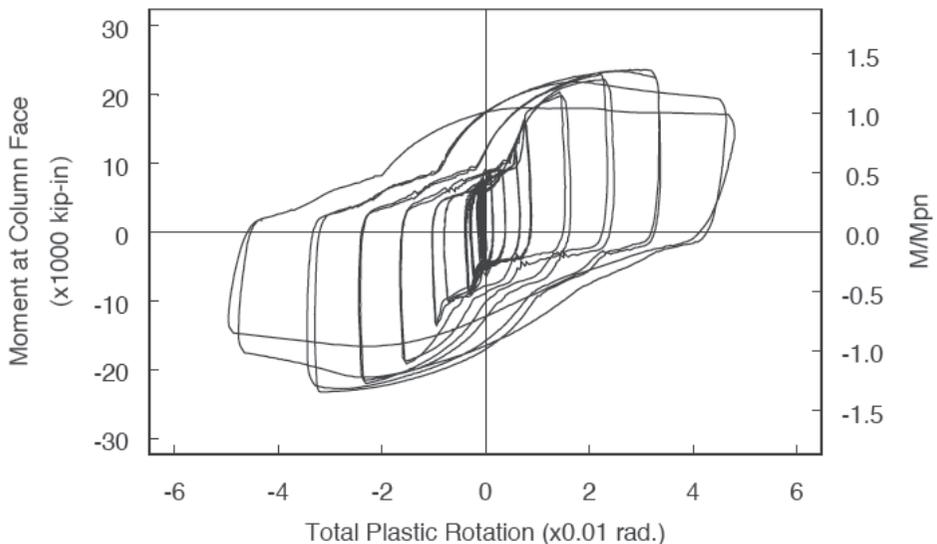

Fig. C-7.1. Moment at face of column vs. total connection rotation for a BFP moment connection with a W30×108 beam and a W14×233 column.

is typically on the order of 1.15 times the expected M_p of the beam, as defined in the AISC *Seismic Provisions,* and the expected moment at the face of the column is on the order of 1.3 to 1.5 times the expected M_p of the beam depending upon the span length, number of bolts, and length of the flange plate. The stiffness of this connection is usually slightly greater than 90% of that anticipated with a truly rigid, fully restrained (FR) connection. This reduced stiffness is expected to result in elastic deflection no more than 10% larger than computed with an FR connection, and so elastic calculations with rigid connections are considered to be adequate for most practical design purposes.

C7.2.   Systems

Review of the research literature shows that BFP moment connections meet the qualifications and requirements of both special moment resisting (SMF) and intermediate moment resisting (IMF) frames. However, no test data are available for BFP moment connections with composite slabs, and so the BFP moment connection is not prequalified with reinforced concrete structural slabs that contact the face of the columns. Reinforced concrete structural slabs, which make contact with the column, may:

- significantly increase the moment at the face of the column,

- cause significant increases of the force and strain demands in the bottom flange plate, and

- result in reduced inelastic deformation capacity of the connection.

Therefore, prequalification of the BFP moment connection is restricted to the case where the concrete structural slab has a minimum separation or isolation from the column. In general, isolation is achieved if shear connectors are not included in the protected zone and if the slab is separated from all surfaces of the column by an open gap or by use of soft foam like material.

C7.3.   Prequalification Limits
C7.3.1. Beam Limits

The SMF prequalification limits largely reflect the range of past testing of the BFP moment connection. Limits for IMF connections somewhat exceed these limits because 18 of the past 20 tests used to prequalify the connection developed plastic rotations larger than those required to qualify as a SMF connection, and all 20 tests greatly exceed the rotation required to qualify as an IMF connection.

BFP moment connections have been tested with beams as large as the W36×150 (W920×223) while achieving the ductility required for qualification as an SMF. Consequently, the W36 (W920) beam depth, 150 lb/ft weight limit (223 kg/m mass limit), and 1 in. (25 mm) flange thickness limits are adopted in this provision. Past tests have shown adequate inelastic rotation capacity to qualify as an SMF in tests with span-to-depth ratios less than 5 and greater than 16, and so lower bound span-to-depth ratio limits of 7 and 9 are conservatively adopted for

the IMF and SMF applications, respectively. Inelastic deformation is expected for approximately one beam depth beyond the end of the flange plate, and limited yielding is expected in the flange plate. As a result, the protected zone extends from the column face to a distance equal to the depth of the beam beyond the flange plate.

Primary plastic hinging of the BFP moment connection occurs well away from the face of the column, and lateral-torsional deformation will occur as extensive yielding develops in the connection. As a result, lateral bracing of the beam is required at the end of the protected zone. The bracing is required within the interval between 1 and 1.5 beam depths beyond the flange bolts farther from the face of the column. This permits some variation in the placement of the lateral support to allow economical use of transverse framing for lateral support where possible. As with other moment frame connections, lateral bracing is also assumed at the column flange connection because of diaphragm stiffness and transverse framing.

As for other prequalified connections, the BFP moment connection requires compact flanges and webs as defined by the AISC *Seismic Provisions*, and built-up I-shaped beams conforming to Section 2.3 are permitted. It should be noted, however, that the BFP and most other prequalified connections do not have specific seismic test data to document the prequalification of built-up beam sections. This prequalification is provided, because long experience shows that built-up steel sections provide similar flexural behavior as hot-rolled shapes with comparable materials and proportions.

C7.3.2. Column Limits

BFP moment connections have been tested with wide flange columns up to W14×233 sections. The SMF prequalification limits largely reflect the range of past testing of the BFP moment connection. All 20 tests were completed with strong axis bending of the column, and the prequalification of the BFP moment connections is limited to connections made to the column flange.

As with most other prequalified connections, the BFP moment connection has not been tested with columns deeper than W14 sections or with built-up column sections. It was the judgment of the committee that the BFP moment connection places similar or perhaps smaller demands on the column than other prequalified connections. The demands may be smaller because of the somewhat smaller strain-hardening moment increase achieved with the BFP moment connection as compared to the welded web-welded flange and other FR connections. The location of yielding of the BFP moment connection is somewhat analogous to the reduced beam section (RBS) connection, and therefore, prequalification limits for the column are comparable to those used for the RBS connection in Section C5.3.2.

C7.4. Beam-Column Relationship Limitations

The BFP moment connection is expected to sustain primary yielding in the beam starting at the last flange plate bolt away from the face of the column. Secondary yielding is expected in the column panel zone and very limited subsequent yielding is expected in the flange plate. Yielding in the column outside the connection panel zone is strongly discouraged. Therefore, the BFP moment connection employs a similar weak beam-strong column check and panel zone resistance check as used for other prequalified connections.

C7.5. Connection Detailing

The BFP moment connection requires plate steel for the flange plate, shear plate, and possibly panel zone doubler plates. Past tests have been performed with plates fabricated both from ASTM A36/A36M and A572/A572M Grade 50 (345) steels. Therefore, the prequalification extends to both plate types. The designer should be aware of potential pitfalls with the material selection for the flange plate design. The flange plate must develop tensile yield strength over the gross section and ultimate tensile fracture resistance over the effective net section. A36/A36M steel has greater separation of the nominal yield stress and the minimum tensile strength, and this may simplify the satisfaction of these dual requirements. However, variation in expected yield stress is larger for A36/A36M steel, and design calculations may more accurately approximate actual flange plate performance with A572/A572M steel.

The flange plate welds are shop welds, and these welds are subject to potential secondary yielding caused by strain hardening at the primary yield location in the beam. As a result, the welds are required to be demand-critical complete-joint-penetration (CJP) groove welds. If *backing* is used, it must be removed and the weld must be *backgouged* to sound material and back welded to assure that the weld can sustain yielding of the flange plate. Since the welds are shop welds, considerable latitude is possible in the selection of the weld process as long as the finished weld meets the demand critical weld requirements stipulated in the AISC *Seismic Provisions*. In the test specimens used to prequalify this connection, electroslag, gas shielded metal arc, and flux cored arc welding have been used.

The BFP moment connection places somewhat less severe demands on the web connection than most FR connections, because of the somewhat greater flexibility of the bolted flange connection. As a result, the shear plate may be welded with CJP groove welds or partial-joint-penetration (PJP) groove welds or fillet welds.

Bolts in the flange plate are limited to two rows of bolts, and the bolt holes must be made by drilling or sub-punching and reaming. These requirements reflect testing used to prequalify the BFP moment connection, but they also reflect practical limitations in the connection design. Net section fracture is a clear possibility in the beam flange and flange plates, and it is very difficult to meet the net section fracture criteria if more than two rows of bolts were employed.

A single row of bolts causes severe eccentricity in the connection and would lead to an excessively long connection. Punched bolt holes without reaming are not permitted, because punching may induce surface roughness in the hole that may initiate cracking of the net section under high tensile stress. As noted earlier, the connection is prequalified only for A490 or A490M bolts with threads excluded from the shear plane. Bolt diameter is limited to a maximum of 1⅛ in. (28 mm), because larger bolts are seldom used and the 1⅛ in. (28 mm) diameter is the maximum used in past BFP tests. The bolt diameter must be selected to ensure that flange yielding over the gross area exceeds the net section capacity of the beam flange.

Oversized bolt holes were included in some past tests, because the oversized holes permit easier alignment of the bolts and erection of the connection and resulted in good performance of the connection. Further, the beam must fit between two welded flange plates with full consideration of rolling and fabrication tolerances. As a result, shims may be used to simplify erection while ensuring a tight connection fit.

C7.6.   Design Procedure

The BFP moment connection is somewhat more complex than some other connections, because a larger number of yield locations and failure modes are encountered with this connection. Step 1 of this procedure defines the maximum expected moment, M_{pr}, at the last bolt away from the face of the column in the flange plate. The beam flange must have greater net section fracture resistance than its yield resistance, because tensile yield of the flange is a ductile yield mechanism and net section fracture is a brittle fracture for the connection. Step 2 establishes the maximum bolt diameter that can meet this balance criterion. While this requirement is rational, it should be noted that net section fracture of the beam flange has not been noted in any past BFP tests, since the beam web clearly reduces any potential for flange fracture.

The shear strength of the flange bolts is the smallest strength permitted by bolt shear with threads excluded from the shear plane, bolt bearing on flange plate, and bolt bearing on the beam flange. Step 3 provides this evaluation. Step 4 is an approximate evaluation of the number of bolts needed to develop the BFP moment connection. The moment for the bolts is larger than M_{pr} because the centroid of the bolt group is at a different location than the primary hinge location. However, this moment cannot be accurately determined until the geometry of the flange plate and bolt spacing are established. The 1.25 factor is used as an empirical increase in this moment to provide this initial estimate for the number of bolts required. The bolts are tightened to meet slip-critical criteria, but they should be designed as bearing bolts.

Once the required number of bolts is established, bolt spacing and an initial estimate of the flange plate length can be established. This geometry is illustrated and summarized in Figure 7.1, and Step 5 defines critical dimensions of this geometry for later design checks.

Step 6 is similar to other connection types in that the shear force at the plastic hinge is based upon the maximum shear achieved with maximum expected moments at the plastic hinges at both ends of the beam plus the shear associated with appropriate gravity loads on the beam.

Step 7 uses the geometry established in Step 5 and the maximum shear force established in Step 6 to determine the maximum expected moment, M_f, at the face of the column flange. The maximum expected force in the flange plate, F_{pr}, is determined from M_f in Step 8.

The flange plate bolts cannot experience a tensile force larger than F_{pr}, and so Step 9 checks the actual number of bolts required in the connection. If this number is larger or smaller than that estimated in Step 4, it may be necessary to change the number of bolts and repeat Steps 5 through 9 until convergence is achieved.

Steps 10 and 11 check the flange plate width and thickness to ensure that tensile yield strength and tensile rupture strength, respectively, exceed the maximum expected tensile force in the flange. The net section fracture check of Step 11 employs the nonductile resistance factor, while the flange yielding check of Step 10 employs the ductile resistance factor; this check also allows limited yielding in the flange plate and ensures ductility of the connection. Step 12 checks block shear of the bolt group in the flange plate, and Step 13 checks the flange plate for buckling, when F_{pr} is in compression. Both block shear and buckling of the flange plate are treated as nonductile behaviors.

Step 14 is somewhat parallel to Step 6 except that the beam shear force at the face of the column is established, and this shear force is then used to size and design the single shear plate connection is Step 15.

Continuity plates, panel zone shear strength, and weak beam–strong column design requirements are checked in Steps 16, 17, and 18, respectively. These checks are comparable to those used for other prequalified connections.

As previously noted, the BFP moment connection has provided quite large inelastic rotational capacity in past research. It has done this by attaining primary yielding in the beam at the end of the flange plate away from the column and through secondary yielding as shear yielding in the column panel zone and tensile yielding in the flange plate. Bolt slip occurs but does not contribute greatly to connection ductility. This rather complex design procedure attempts to achieve these goals by balancing the resistances for different yield mechanisms and failure modes in the connection and by employing somewhat greater conservatism for brittle behaviors than for ductile behaviors.

Add the following new commentary:

C8. WELDED UNREINFORCED FLANGE–WELDED WEB (WUF–W) MOMENT CONNECTION

C8.1 General

The welded unreinforced flange–welded web (WUF–W) moment connection is an all-welded moment connection, wherein the beam flanges and the beam web are welded directly to the column flange. A number of welded moment connections that came into use after the 1994 Northridge Earthquake, such as the reduced beam section and connections provided with beam flange reinforcement, were designed to move the plastic hinge away from the face of the column. In the case of the WUF–W moment connection, the plastic hinge is not moved away from the face of the column. Rather, the WUF–W moment connection employs design and detailing features that are intended to permit the connection to achieve SMF performance criteria without fracture. Key features of the WUF–W moment connection that are intended to control fracture are described below.

- The beam flanges are welded to the column flange using CJP groove welds that meet the requirements of *demand critical welds* in the AISC *Seismic Provisions*, along with the requirements for treatment of *backing* and *weld tabs* and welding quality control and quality assurance requirements, as specified in Chapter 3 of this standard.

- The beam web is welded directly to the column flange using a CJP groove weld that extends the full-depth of the web—that is, from weld access hole to weld access hole. This is supplemented by a single-plate connection, wherein a single plate is welded to the column flange and is then fillet welded to the beam web. Consequently, the beam web is attached to the column flange with both a CJP groove weld and a welded single-plate connection. The single-plate connection adds stiffness to the beam web connection, drawing stress toward the web connection and away from the beam flange to column connections. The single plate also serves as backing for the CJP groove weld connecting the beam web to the column flange.

- Instead of using a conventional weld access hole detail as specified in Section J1.6 of the AISC *Specification*, the WUF–W moment connection employs a special seismic weld access hole with requirements on size, shape and finish that reduce stress concentrations in the region around the access hole. See Figure 11-1 in the AISC *Seismic Provisions*.

Prequalification of the WUF–W moment connection is based on the results of two major research and testing programs. Both programs combined large-scale tests with extensive finite element studies. Both are briefly described herein.

The first research program on the WUF–W moment connection was conducted at Lehigh University as part of the SAC-FEMA program. Results are reported in several publications (Ricles, Mao, Lu and Fisher, 2000 and 2002). This test program formed the basis of prequalification of the WUF–W moment connection in FEMA 350 (2000). As part of the Lehigh program, tests were conducted on both interior and exterior type specimens. The exterior specimens consisted of one beam attached to a column. The interior specimens consisted of a column with beams attached to both flanges. One of the interior specimens included a composite floor slab. All specimens used W36×150 beams. Three different column sizes were used: W14×311, W14×398 and W27×258. All WUF–W moment connection specimens tested in the Lehigh program satisfied the rotation criteria for SMF connections (±0.04 radian total rotation). Most specimens significantly exceeded the qualification criteria. Considering that the interior type specimens included two WUF–W moment connections each, a total of 12 successful WUF–W moment connections were tested in the Lehigh program. This research program included extensive finite element studies, which supported the development of the special seismic weld access hole and the details of the web connection.

The second major research program on the WUF–W moment connection was conducted at the University of Minnesota. The purpose of this research program was to examine alternative doubler plate details, continuity plate requirements, and effects of a weak panel zone. All test specimens used the WUF–W moment connection. Results are reported in several publications (Lee, Cotton, Hajjar, Dexter and Ye, 2002, 2005a and 2005b). A total of six interior type specimens were tested in the Minnesota program. All specimens used W24×94 beams. Three different column sizes were used: W14×283, W14×176 and W14×145. All specimens were designed with panel zones weaker than permitted by the 2005 AISC *Seismic Provisions*. Two of the test specimens, CR1 and CR4, were inadvertently welded with low-toughness weld metal. This resulted in premature weld failure in specimen CR4 (failure occurred at about 0.015 radian rotation). With the exception of CR4, all specimens achieved a total rotation of ±0.04 radian, and sustained numerous cycles of loading at ±0.04 radian prior to failure. All successful specimens exhibited substantial panel zone yielding, due to the weak panel zone design. Considering that the interior type specimens included two WUF–W moment connections each, a total of 12 WUF–W moment connections were tested in the Minnesota program. Ten connections satisfied the rotation qualification criteria for SMF. Two of the connections failed to achieve adequate rotations, due to presence of low-toughness beam flange welds. This test program was also supported by extensive finite element studies.

Considering the two WUF–W moment connection research programs (Lehigh and University of Minnesota) taken together, WUF–W moment connection specimens have shown excellent performance in tests. There is only one reported failed test, due to the inadvertent use of low-toughness weld metal for beam flange CJP groove welds (Minnesota Specimen CR4). Of all of the WUF–W moment connection specimens that showed good performance (achieved rotations of at least ±0.04 radian), approximately one-half had panel zones weaker than permitted by the 2005 AISC *Seismic Provisions*. The other half satisfied the panel zone strength criteria of the 2005 AISC *Seismic Provisions*. This suggests that that the WUF–W moment connection performs well for both strong and weak panel zones, and is therefore not highly sensitive to panel zone strength.

Based on a review of the two major research programs on the WUF–W moment connection, which included tests on 24 connections supplemented by extensive finite element studies, it was the judgment of the CPRP that the WUF–W moment connection satisfied the prequalification requirements in Appendix P of the 2005 AISC *Seismic Provisions*, subject to the prequalification limits and other design and detailing requirements specified in Chapter 8 of this standard.

The *protected zone* for the WUF–W moment connection is defined as the portion of the beam extending from the face of the column to a distance d from the face of the column, where d is the depth of the beam. Tests on WUF–W moment connection specimens show that yielding in the beam is concentrated near the face of the column, but extends to some degree over a length of the beam approximately equal to its depth.

C8.3 Prequalification Limits

The WUF–W moment connection is prequalified for beams up to W36 in depth, up to 150 lb/ft in weight, and up to a beam flange thickness of 1 in. (25 mm). This is based on the fact that a W36×150 is the deepest and heaviest beam tested with the WUF–W moment connection. The 1-in. (25-mm) flange thickness limitation represents a small extrapolation of the 0.94-in. (23.9-mm) flange thickness for the W36×150. Limits are also placed on span-to-depth ratio based on the span-to-depth ratios of the tested connections and based on judgment of the CPRP.

Beam lateral bracing requirements for the WUF–W moment connection are identical to those for the RBS moment connection. The effects of beam lateral bracing on cyclic loading performance have been investigated more extensively for the RBS moment connection than for the WUF–W moment connection. However, the available data for the WUF–W moment connection suggests that beams are less prone to lateral torsional buckling than with the RBS moment connections. Consequently, it is believed that lateral bracing requirements established for the RBS moment connection are satisfactory, and perhaps somewhat conservative, for the WUF–W moment connection.

Column sections used in WUF–W moment connection test specimens have been W14 and W27 sections. However, column limitations for the WUF–W moment connection have been taken the same as for the RBS moment connection, which includes wide-flange shapes up to W36 and box columns up to 24 in. by 24 in. A primary concern with deep columns in moment frames has been the potential for twisting and instability of the column driven by lateral torsional buckling of the beam. Because beams with WUF–W moment connections are viewed as somewhat less prone to lateral torsional buckling than beams with RBS moment connections, the column limitations established for the RBS moment connection were judged as appropriate for the WUF–W moment connection.

C8.4   Beam-Column Relationship Limitations

WUF–W moment connection test specimens have shown good performance with a range of panel zone shear strengths, ranging from very weak to very strong panel zones. Tests conducted at the University of Minnesota (Lee and others, 2005b) showed excellent performance on specimens with panel zones substantially weaker than required in the AISC *Seismic Provisions*. However, there are concerns that very weak panel zones may contribute to premature connection fracture under some circumstances, and it is believed further research is needed before weak panel zone designs can be prequalified. Consequently, the minimum panel zone strength required in Section 9.3a of the 2005 AISC *Seismic Provisions* is required for prequalified WUF–W moment connections for SMF. For IMF systems, the 2005 AISC *Seismic Provisions* have no special panel zone strength requirements, beyond the *Specification*. This may lead to designs in which inelastic action is concentrated within the panel zone. As described earlier, based on successful tests on WUF–W moment connection specimens with weak panel zones, this condition is not viewed as detrimental for IMF systems. Consequently, no panel zone strength requirements beyond those specified in Section 10.3 of the 2005 AISC *Seismic Provisions* are required for IMF systems with WUF–W moment connections.

C8.5   Beam Flange to Column Flange Welds

Beam flanges are required to be connected to column flanges with CJP groove welds. The welds must meet the requirements of *demand critical welds* in the AISC *Seismic Provisions*, as well as the detailing and quality control and quality assurance requirements specified in Chapter 3 of this standard. These beam flange to column flange weld requirements reflect the practices used in the test specimens that form the basis for prequalification of the WUF–W moment connection and reflect what are believed to be best practices for beam flange groove welds for SMF and IMF applications.

A key feature of the WUF–W moment connection is the use of a special weld access hole. The special seismic weld access hole has specific requirements on the size, shape and finish of the access hole. This special access hole was developed in research on the WUF–W moment connection (Ricles and others, 2000 and 2002) and is intended to reduce stress concentrations introduced by the presence

of the weld access hole. The size, shape and finish requirements for the special access hole are specified in AWS D1.8, Sections 6.9.1.2 and 6.9.2. Since these requirements are specified in AWS D1.8, they are not repeated herein, but are called out by reference to AWS D1.8. See also Figure 11-1 in the AISC *Seismic Provisions*.

C8.6   Beam Web to Column Connection Limitations

The beam web is connected to the column flange with a full-depth (weld access hole to weld access hole) CJP groove weld and with a single plate that is welded to the column flange and fillet welded to the beam web. This single plate also serves as backing for the beam web CJP groove weld. The use of the CJP groove weld combined with the fillet-welded single plate is believed to increase the stiffness of the beam web connection. The stiffer beam web connection serves to draw stress away from the beam flanges and therefore reduces the demands on the beam flange groove welds.

Most of the details of the beam web to column connection are fully prescribed in Section 8.6; thus few design calculations are needed for this connection. An exception to this is the connection of the single plate to the column. This connection must develop the shear strength of the single plate, as specified in Section 8.6(2). This can be accomplished by the use of CJP groove welds, PJP groove welds, fillets welds, or combinations of these welds. The choice of these welds is left to the discretion of the designer. In developing the connection between the single plate and the column flange, designers should consider the following issues:

- The use of a single-sided fillet weld between the single plate and the column flange should be avoided. If the single plate is inadvertently loaded or struck in the out-of-plane direction during erection, the fillet weld may break and may lead to erection safety concerns.

- The end of the beam web must be set back from the face of the column flange a specified amount to accommodate the web CJP *root* opening dimensional requirements. Consequently, the single plate to column weld that is placed in the web CJP root opening must be small enough to fit in that specified root opening. For example, if the CJP groove weld is detailed with a ¼-in. (6-mm) root opening, a fillet weld between the single plate and the column flange larger than ¼ in. (6 mm) will cause the root of the CJP groove weld to exceed ¼ in. (6 mm).

- Placement of the CJP groove weld connecting the beam web to the column flange will likely result in intermixing of weld metal, with the weld attaching the single plate to the column flange. Requirements for intermix of filler metals specified in Section 6.3.4 of AWS D1-8 should be followed in this case.

The CJP groove weld connecting the beam web to the column flange must meet the requirements of *demand critical welds*. Note that weld tabs are permitted, but not required, at the top and bottom ends of this weld. If weld tabs are used, they should be removed after welding according to the requirements of Section 3.4. If weld tabs are not used, the CJP groove weld should be terminated in a manner that minimizes notches and stress concentrations, such as with the use of cascaded ends.

The fillet weld connecting the beam web to the single plate should be terminated a small distance from the weld access hole, as shown in Figure 8.3. This is to avoid introducing notches at the edge of the weld access hole.

C8.7 Connection Design Procedure

For the WUF–W moment connection, many of the details of the connection of the beam to the column flange are fully prescribed in Sections 8.5 and 8.6. Consequently, the design procedure for the WUF–W moment connection largely involves typical checks for continuity plates, panel zone shear strength, column-beam moment ratio, and beam shear strength.

With the WUF–W moment connection, yielding of the beam (i.e., plastic hinge formation) occurs over the portion of the beam extending from the face of the column to a distance of approximately one beam depth beyond the face of the column. For purposes of the design procedure, the location of the plastic hinge is taken to be at the face of the column. That is, $S_h = 0$ for the WUF–W moment connection. It should be noted that the location of the plastic hinge for design calculation purposes is somewhat arbitrary, since the plastic hinge does not occur at a single point but instead occurs over some length of the beam. The use of $S_h = 0$ is selected to simplify the design calculations. The value of C_{pr} was calibrated so that when used with $S_h = 0$, the calculated moment at the column face reflects values measured in experiments. Note that the moment in the beam at the column face is the key parameter in checking panel zone strength, column-beam moment ratio, and beam shear strength.

The value of C_{pr} for the WUF–W moment connection is specified as 1.4, based on an evaluation of experimental data. Tests on WUF–W moment connections with strong panel zones (Ricles and others, 2000) showed maximum beam moments, measured at the face of the column, as high as $1.49M_p$, where M_p was based on measured values of F_y. The average maximum beam moment at the face of the column was $1.33M_p$. Consequently, strain hardening in the beam with a WUF–W moment connection is quite large. The value of C_{pr} of 1.4 was chosen to reflect this high degree of strain hardening. Combining the value of $C_{pr} = 1.4$ with $S_h = 0$ results in a moment at the face of the column, $M_f = M_{pr} = 1.4R_yF_yZ$, which reasonably reflects maximum column face moments measured in experiments.

Add the following new commentary:

C9. KAISER BOLTED BRACKET (KBB) MOMENT CONNECTION

C9.1. General

The Kaiser bolted bracket (KBB) moment connection is designed to eliminate field welding and facilitate frame erection. Depending on fabrication preference, the brackets can be either fillet welded (W-series) or bolted (B-series) to the beam. The B-series can also be utilized to improve the strength of weak or damaged connections, although it is not prequalified for that purpose. Information on the cast steel and the process used to manufacture the brackets is provided in Appendix A.

The proprietary design of the brackets is protected under US patent number 6,073,405 held by Steel Cast Connections LLC. Information on licensing rights can be found at http://www.steelcastconnections.com. The connection is not prequalified when brackets of an unlicensed design and/or manufacture are used.

Connection prequalification is based on 21 full-scale bolted bracket tests representing both new and repaired applications (Kasai and Bleiman, 1996; Gross, Engelhardt, Uang, Kasai and Iwankiw, 1999; Newell and Uang, 2006; and Adan and Gibb, 2009). These tests were performed using beams ranging in depth from W16 to W36 and columns using W12, W14 and W27 sections. Built-up box columns have also been tested. The test subassemblies have included both single cantilever and double-sided column configurations. During testing, inelastic deformation was achieved primarily through the formation of a plastic hinge in the beam. Some secondary yielding was also achieved in the column panel zone. Peak strength typically occurred at an interstory drift angle between 0.025 and 0.045 radian. Specimen strength then gradually decreased with additional yielding and deformation. In the KBB testing reported by Adan and Gibb (2009), the average specimen maximum interstory drift angle exceeded 0.055 radian.

C9.2. Systems

Review of the research literature and testing referenced in this document indicates that the Kaiser bolted bracket moment connection meets the prequalification requirement for special and intermediate moment frames.

The exception associated with concrete structural slab placement at the column and bracket flanges is based on testing conducted on the stiffened extended end-plate moment connection (Seek and Murray, 2005). While bolted bracket testing has been conducted primarily on bare-steel specimens, some limited testing has also been performed on specimens with a *concrete structural slab*. In these tests, the presence of the slab provided a beneficial effect by maintaining the stability

of the beam at larger interstory drift angles (Gross and others, 1999; Newell and Uang, 2006). However, in the absence of more comprehensive testing with a slab, the placement of the concrete is subject to the exception.

C9.3. Prequalification Limits

C9.3.1. Beam Limitations

A wide range of beam sizes have been tested with bolted brackets. The lightest beam size reported in the literature was a W16×40. The heaviest beam reported was a W36×210. In the W36×210 test, the specimen met the requirements, but subsequently experienced an unexpected nonductile failure of the bolts connecting the bracket to the column. The next heaviest beams reported to have met the requirements were W33×130 and W36×150. Based on the judgment of the CPRP, the maximum beam depth and weight was limited to match that of the W33×130. The maximum flange thickness was established to match a modest increase above that of the W36×150.

The limitation associated with minimum beam flange width is required to accommodate fillet weld attachment of the W-series bracket and to prevent beam flange tensile rupture when using the B-series bracket.

Bolted bracket connection test assemblies have used configurations approximating beam spans between 24 and 30 feet (7310 and 9140 mm). The beam span-to-depth ratios have been in the range of 8 to 20. Given the degree to which most specimens significantly exceeded the requirement, it was judged reasonable to set the minimum span-to-depth ratio at 9 for both SMF and IMF systems.

As with other prequalified connections, beams supporting a concrete structural slab are not required to have a supplemental brace near the expected plastic hinge. If no floor slab is present, then a supplemental brace is required. The brace should not be located within the protected zone.

C9.3.2. Column Limitations

Bolted bracket connection tests have been performed with the brackets bolted to the column flange (i.e., strong-axis connections). In the absence of additional testing with brackets bolted to the column web (weak-axis connections), the prequalification is limited to column flange connections.

Test specimen wide flange column sizes have ranged from a W12×65 to a W27×281. Testing performed by Ricles, Zhang, Lu and Fisher (2004) of deep-column RBS connections have concluded that deep columns do not behave substantially different from W14 columns when a slab is present or when adequate lateral bracing is provided for the beam and/or column in the absence of a slab. Based on the similarity in performance to that of the RBS connection, the KBB is prequalified to include column sizes up to W36.

The behavior of a flanged cruciform column in KBB connections is expected to be similar to that of a rolled wide-flange. Therefore, flanged cruciform columns are prequalified, subject to the limitations imposed on rolled wide-flange shapes.

Two of the tests were successfully conducted using a built-up box column. In the first box column test, connections were made on two opposing column faces. Then, in the second test, a connection was made to the orthogonal face of the same column. These two tests were intended to prequalify a box column participating in orthogonal moment frames. The tested box column was 15⅝ in. (397 mm) square (Adan and Gibb, 2009). Consequently, bolted bracket connections are prequalified for use with built-up box columns up to 16 in. (406 mm) square.

Based on both successful wide-flange and built-up box column testing, acceptable performance would also be expected for boxed wide-flange columns. Therefore, the use of boxed wide-flange columns is also prequalified. When moment connections are made only to the flanges of the wide-flange portion of the boxed wide-flange, subject to the bracing limitations mentioned previously, the column may be up to W36 in depth. When the boxed wide-flange column participates in orthogonal moment frames, neither the depth nor the width of the column is allowed to exceed 16 in. (406 mm), applying the same limit as a built-up box.

C9.3.3. Bracket Limitations

The ASTM cast steel material specification used to manufacture the brackets is based on recommendations from the Steel Founder's Society of America (SFSA).

The cast brackets have been configured and proportioned to resist applied loads in accordance with the limit states outlined by Gross and others (1999). These limit states include flange local buckling, bolt prying action, combined bending and axial loading, shear, and for the B-series, bolt bearing deformation and block shear rupture.

In tests representing new applications, the bracket column bolt holes have been cast vertically short-slotted. The vertically slotted holes provide field installation tolerance. In tests representing a repair application, the holes have been cast standard diameter. There has been no difference in connection performance using either type of cast hole (Adan and Gibb, 2009).

C9.5.   Bracket-to-Column Flange Limitations

In the prequalification tests, fasteners joining the bracket to the column flange have been pretensioned ASTM A490 or A490M bolts. The column bolt head can be positioned on either the column or bracket side of the connection. Where possible, the column bolts are tightened prior to the bolts in the web shear tab.

When needed, finger shims between the bracket and column face allow for fit between the bracket and column contact surfaces. The testing has indicated that the use of finger shims does not affect the performance of the connection.

Because the flanges of a box column are stiffened only at the corners, tightening of the column bolts can cause excessive local flange bending. Therefore, as shown in Figure C-9.1, a washer plate is required between the box column flange and the bracket.

As shown in Figure C-9.1, orthogonally connected beams framing into a box column shall be raised half of the column bolt spacing distance to avoid overlapping the column bolts.

C9.6. Bracket-to-Beam-Flange Connection Limitations

The cast steel brackets are not currently listed as a prequalified material in AWS D1.1. Therefore, the weld procedure specification (WPS) for the fillet weld joining the bracket to the beam flange is required to be qualified by test with the specific cast material.
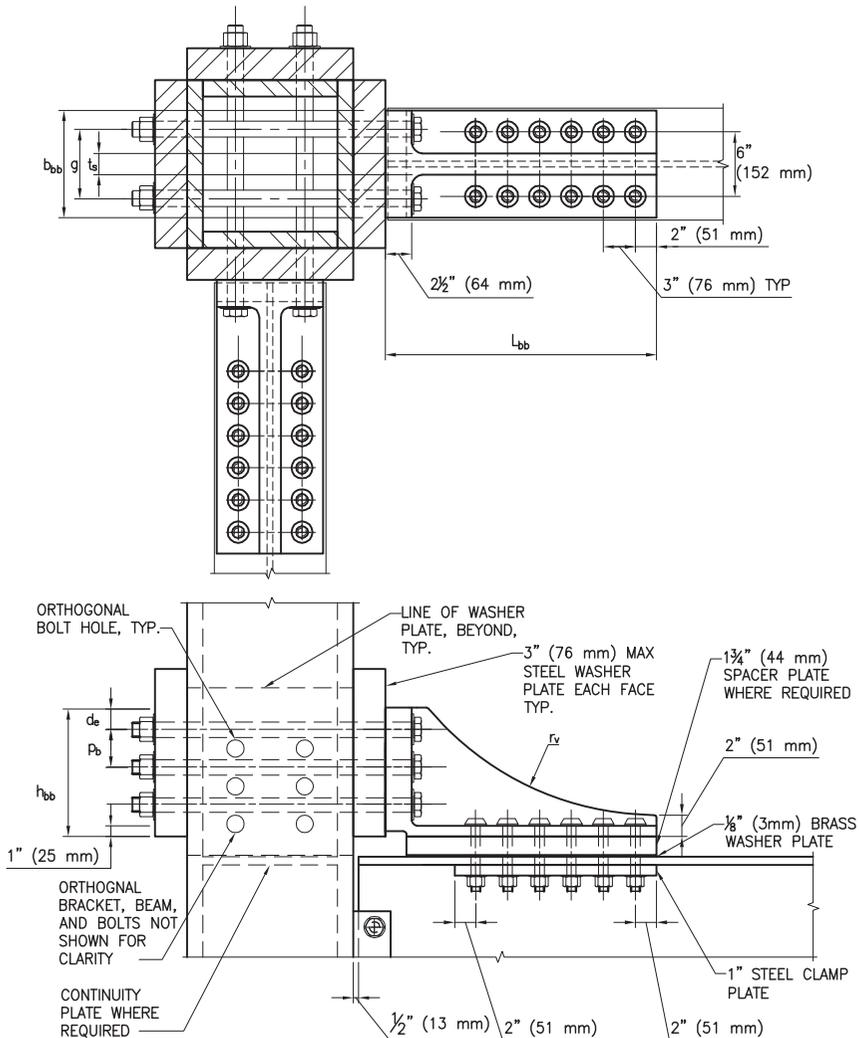
Fig. C-9.1. Box column connection detailing.

Bolts joining the bracket to the beam flange in prequalification tests have been conducted with pretensioned ASTM A490 or A490M bolts with the threads excluded from the shear plane. The beam bolt head can be positioned on either the beam or bracket side of the connection. Given the beam bolt pattern and hole size, it is necessary to use the bracket as a template when drilling the beam bolt holes. The holes must be aligned to permit insertion of the bolts without undue damage to the threads.

The brass washer plate prevents abrading of the beam and bracket contact surfaces. In the initial developmental stages of the connection, several specimens configured without the brass plate experienced flange net section fracture through the outermost bolt holes. Observation of the failed specimens indicated that fracture likely initiated at a notch created by the abrading contact surfaces near the hole. Furthermore, energy released through the beam-bracket slip-stick mechanism caused loud, intermittent bursts of noise, particularly at high levels of inelastic drift (Kasai and Bleiman, 1996). In order to overcome these problems, the brass plate was inserted between the bracket and the beam flange. The idea is based on the use of a brass plate as a special friction based seismic energy dissipator (Grigorian, Yang and Popov, 1992). Although not intended to dissipate energy in the bolted bracket connection, the brass plate provides a smooth slip mechanism at the bracket-to-beam interface.

When bolting the bracket to a beam flange, a steel washer or clamp plate is positioned on the opposite side of the connected flange. The restraining force of the clamp plate prevents local flange buckling from occurring near the outermost bolt holes. In tests performed without the clamp plates, flange distortion increased the strains near the holes. The increased strain caused necking and fracture through the flange net area. In similar tests performed with the clamp plates, yielding and fracture occurred outside the connected region through the flange gross area (Kasai and Bleiman, 1996).

C9.7.   Beam-Web-to-Column Connection Limitations

All of the bolted bracket connection tests have been performed with a bolted web connection, where pretensioned high-strength bolts were used. Therefore, the KBB is prequalified for a bolted beam web-to-column connection.

C9.8.   Connection Detailing

Both Figures 9.4 and 9.5 show the connection configured with continuity plates where required. The use of continuity plates is dictated by the need to satisfy prescribed limit states for the flange and web of the column. In a bolted connection, the configuration of the fasteners can impede the ability of the continuity plates to effectively address these limit states. The design intent for the KBB is to satisfy the prescribed limit states without continuity plates. In tests of wide flange columns without continuity plates, the absence of the continuity plates did not appear to promote local flange bending or lead to other detrimental effects (Adan and Gibb, 2009). However, in the absence of additional tests on deeper column sections, prequalification without continuity plates is limited to W12 and W14 sections.

TABLE C-9.1. Recommended W-Series Bracket-Beam Combinations	
Bracket Designation	**Beam Designations**
W1.0	W33×130, W30×124, W30×116, W24×131, W21×122, W21×111
W2.1	W30×108, W27×114, W27×102, W24×103, W21×93, W18×106, W18×97
W2.0	W27×94, W24×94, W24×84, W24×76, W21×83, W21×73, W21×68, W21×62, W18×86, W18×71, W18×65
W3.1	W24×62, W24×55, W21×57, W18×60, W18×55, W16×57
W3.0	W21×50, W21×44, W18×50, W18×46, W18×35, W16×50, W16×45, W16×40, W16×31

Table C-9.2. Recommended B-Series Bracket-Beam Combinations	
Bracket Designation	**Beam Designations**
B1.0	W33×130, W30×124, W30×116, W24×131, W21×122, W21×111
B2.1	W30×108, W27×114, W27×102, W27×94, W18×106, W18×97

C9.9. Design Procedure

The design procedure presented herein for prequalified KBB connections is intended to develop the probable maximum moment capacity of the connecting beam. Test data indicate that connecting the brackets and beam to the column in accordance with the requirements herein allows the connection to resist this level of moment.

Tables C-9.1 and C-9.2 can be used as a guide in selecting trial bracket-beam combinations in conjunction with Steps 1 and 3. The tables are based on beams that satisfy the limitations of Section 9.3.1 for ASTM A992/A992M or A572/A572 Grade 50 (345) wide flange shapes.

Step 4 of the procedure requires computation of the shear force at the expected plastic hinge. This shear force is a function of the gravity load on the beam and the plastic moment capacity. A similar calculation for the RBS moment connection is required for the case of a beam with a uniformly distributed gravity load as shown in Figure C-5.1. For the KBB, L' is the distance between the expected plastic hinge locations and S_h is the distance from the face of the column to the hinge. The brief explanation associated with equations C-5.8-1 and C-5.8-2 also applies to the KBB.

Step 6 is based on the limit state of bolt tensile rupture as defined in Section J3.6 of the AISC *Specification*, where the required bolt tensile strength is determined in Equation 9.9-3.

Steps 7 and 11 of the procedure apply to rolled or built-up shapes with flange holes, proportioned on the basis of flexural strength of the gross section. The flexural strength is limited in accordance with the limit state of flange tensile rupture as defined in Section F13.1(a) of the AISC *Specification*. When the flange width is adequate, the tensile rupture limit state does not apply.

Step 8 of the procedure requires a column flange prying action check as outlined in the Chapter 9 requirements of the AISC *Manual*. The computations include provisions from the research performed by Kulak, Fisher and Struik (1987).

Step 9 of the procedure is based on the limit state of column flange local bending as defined in Section J10.1 of the AISC *Specification*. The limit state determines the strength of the flange using a simplified yield line analysis. Yield line analysis is a method that determines the flexural load at which a collapse mechanism will form in a flat plate structure and employs the principle of virtual work to develop an upper bound solution for plate strength. Given the bolted bracket configuration, the solution can be simplified to determine the controlling yield line pattern that produces the lowest failure load. Because a continuity plate would interfere with the installation of the connecting bolts, the procedure requires that the column flange thickness adequately satisfies the limit state without the requirement to provide continuity plates.

Although Step 9 requires a flange thickness that will adequately satisfy the column flange local bending limit state, the limit states of web local yielding, web crippling, and web compression buckling as defined in Sections J10.2, J10.3, and J10.5 of the AISC *Specification*, respectively, may also be applicable. In shallow seismically compact W12 (W310) and W14 (W360) sections these additional limit states will not control. However, in some deeper sections, the additional limit states may govern. Therefore, Step 10 requires continuity plates in the deeper sections to adequately address the limit states and to stabilize deep column sections. The plates are positioned at the same level as the beam flange as shown in Figures 9.4 and 9.5.

Step 12 of the procedure is based on the limit state of bolt shear rupture as defined in Section J3.6 of the AISC *Specification*. The procedure includes a bolt shear overstrength factor of 1.1 based on research reported by Tide (2006).

With respect to Step 12, the procedure omits a bolt-bearing limit state check per Section J4.10 of the AISC *Specification* as the provisions of Sections 9.3.1(5) and (7) preclude the use of beams where the bolt bearing would limit the strength of the connection.

Step 14 of the procedure is based on the limit state of weld shear rupture as defined in Section J2.4 of the AISC *Specification.* The procedure assumes a linear weld group loaded in-plane through the center of gravity.

With respect to Step 18, the reduction of column axial and moment capacity due to the column bolt holes need not be considered when checking column-beam moment ratios. Research performed by Masuda, Tamaka, Hirabayashi and Genda (1998) indicated that a 30 to 40% loss of flange area due to bolt holes showed only a corresponding 10% reduction in the yield moment capacity.

Step 19 of the procedure is supplemental if the column is a built-up box configuration. The procedure is based on the limit state of yielding (plastic moment) as defined in Section F11.1 of the AISC *Specification*. The design assumes a simply supported condition with symmetrical point loads applied at the bolt locations.

Add the following new commentary:

CA. CASTING REQUIREMENTS

CA.1. Cast Steel Grade

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

CA.2. Quality Control

See Section C3.7.

CA.2.2. First Article Inspection (FAI) of Castings

FAI shall be submitted to the patent holder or designee prior to the initiation of production. At least one casting of each pattern shall undergo FAI. When a casting pattern is replaced or when the rigging is modified, the FAI shall be repeated.

CA.2.3. Visual Inspection of Castings

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

CA.2.4. Nondestructive Testing (NDT) of Castings

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

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

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

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

(1) The vertical flange and the horizontal flange.

(2) The vertical flange and the vertical stiffener.

(3) The horizontal flange and the vertical stiffener.

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

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

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

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

CA.2.6. Tensile Testing

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

CA.3. Manufacturer Documents

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

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