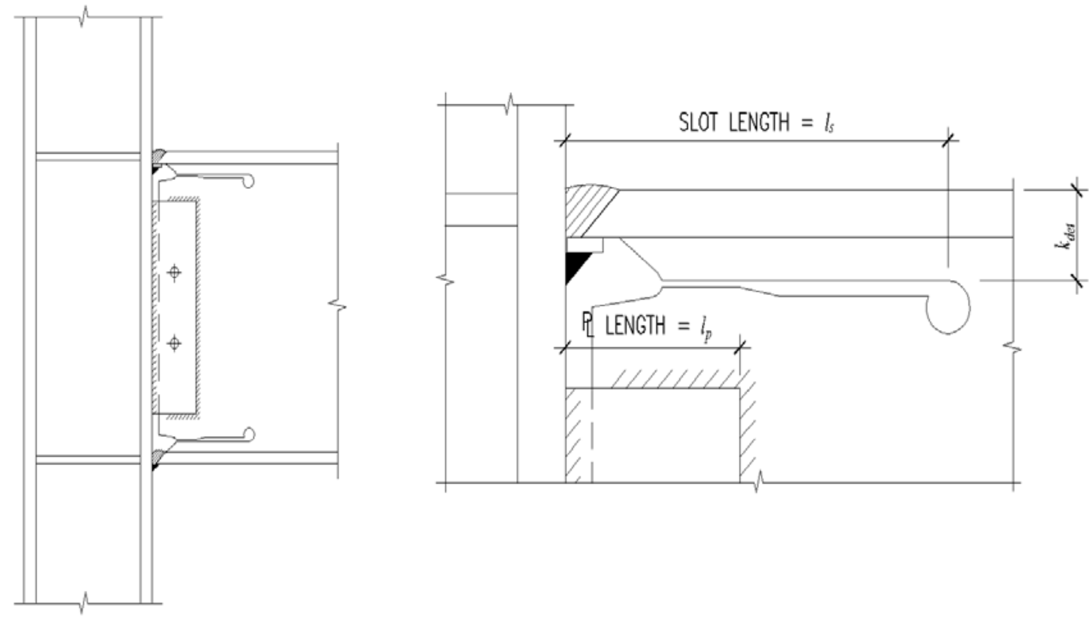
CHAPTER 14

SLOTTEDWEB™ (SW) MOMENT CONNECTION

The user's attention is called to the fact that compliance with this chapter of the standard requires use of an invention covered by patent rights.[1] By publication of this standard, no position is taken with respect to the validity of any claim(s) or of any patent rights in connection therewith. The patent holder has filed a statement of willingness to grant a license under these rights on reasonable and nondiscriminatory terms and conditions to applicants desiring to obtain such a license, and the statement may be obtained from the standards developer.

14.1. GENERAL

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



(a) (b)

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

14.2. SYSTEMS

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

14.3. PREQUALIFICATION LIMITS

[1]The SlottedWeb™ connection configuration illustrated herein is protected by one or more of the following U.S. patents: U.S. Pat. Nos. 5,680,738; 6,237,303; 7,047,695; all held by Seismic Structural Design Associates.

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

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

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

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

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

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

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

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

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

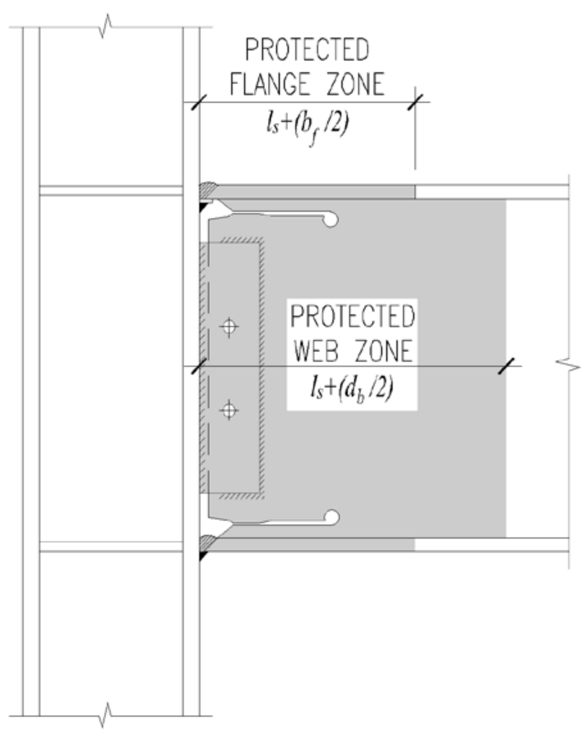


Fig. 14.2. Protected zones.

2. Column Limitations

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

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

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

(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 requirements of the AISC *Seismic Provisions.*

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

14.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

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

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

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

The moment ratio shall conform to the AISC *Seismic Provisions.* The value of $\sum M^{*}_{pb}$ shall be taken equal to $\sum\left(M_{pr}+M_{uv}\right)$, where M_{pr} is the probable maximum moment of the beam and where M_{uv} is the additional moment due to shear amplification from the plastic hinge, which is located at the end of the shear plate, to the centerline of the column.

$$M_{uv}=V_{beam}\left(l_p+d_{col}/2\right) \tag{14.4-1}$$

where

V_{beam} = shear at the beam plastic hinge, kips (N), computed according to step 3 in Section 14.8.

d_{col} = depth of the column, in. (mm)

l_p = width of the shear plate, in. (mm)

14.5. BEAM FLANGE-TO-COLUMN FLANGE WELD LIMITATIONS

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

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

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

14.6. BEAM WEB AND SHEAR PLATE CONNECTION LIMITATIONS

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

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

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

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

(2) The minimum shear plate thickness shall be equal to at least 2/3 of the beam web thickness but not less than 3/8 in. (10 mm). To stabilize the beam web from lateral buckling at the column flange pretensioned bolts in standard holes with a maximum bolt spacing of 6 in. (150 mm) on center over the full height of the plate are required. The diameter of the bolts shall be equal to or greater than the thickness of the beam web.

14.7. FABRICATION OF BEAM WEB SLOTS

The beam web slots shall be made using thermal cutting or milling of the slots and holes or by drilling the holes to produce surface roughness in the slots or holes not exceeding 1,000 micro-inches (25 microns). Gouges and notches that may occur in the cut slots shall be repaired by grinding. The beam web slots shall terminate at thermally cut or drilled 1 $^{1}/_{16}$-in. (27 mm) diameter holes for beams nominal 24 in. (610 mm) deep or greater or 13/16-in. (21 mm) holes for beams less than nominal 24 in. (610 mm) deep. Punched holes are not permitted. The slot widths and tolerances are shown in Figure 14.3. The length of the 1/8-in. slot should be at least equal to the width of the shear plate, but need not exceed ½ the slot length, l_s. The transition from the 1/8-in. (3 mm) slot to the ¼-in. slot (6 mm) shall not have a slope greater than 1 vertical to 3 horizontal.

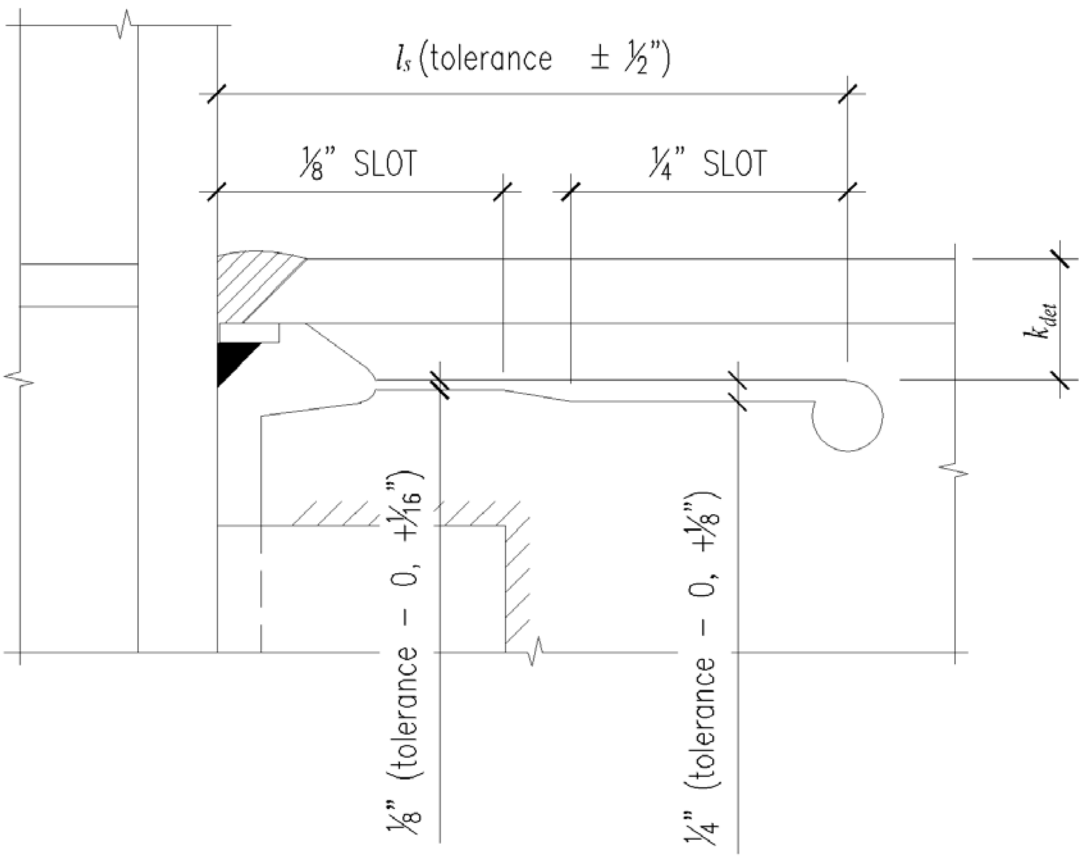


Fig. 14.3. Slot widths and tolerances.

14.8. DESIGN PROCEDURE

Step 1. Design the beam web slots. The beam slot length, l_s, shall be the least of the following within ± 10%:

$$l_s = 1.5b_f \tag{14.8-1}$$

$$l_s = 0.60t_{bf}\sqrt{\frac{E}{F_{ye}}} \tag{14.8-2}$$

$$l_s = \frac{d}{2} \tag{14.8-3}$$

$$l_s = l_p + \frac{(l_b - l_p)}{10} \tag{14.8-4}$$

where

E = steel elastic modulus, ksi (MPa)
F_{ye} = expected yield strength of steel beam, ksi (MPa)
= R_yF_y
R_y = ratio of the expected yield stress to the minimum yield stress, F_y
b_f = beam flange width, in. (mm)
d = nominal depth of the beam, in. (mm)
l_b = half the clear span length of beam, in. (mm)

l_p = width of the shear plate, in. (mm)
t_{bf} = beam flange thickness, in. (mm)

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

$$h = T - 2 \text{ in.} \pm 1 \text{ in.} \tag{14.8-5}$$
$$h = T - 50 \text{ mm} \pm 25 \text{ mm} \tag{14.8-5M}$$

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

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

$$t_p = C_{pr}\left(\frac{6}{h^2}\right)R_y\left(\frac{Z_{beam}l_p}{l_b - l_p}\right) \tag{14.8-6}$$

where

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

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

$$M_{weld} = C_{pr}\left(\frac{t_p}{t_p + t_{bw}}\right)\left(\frac{h}{T}\right)^2 Z_{web}R_yF_y \tag{14.8-7}$$

$$V_{weld} = V_{beam}\left(\frac{t_p}{t_p + t_w}\right) \tag{14.8-8}$$

$$e_x = \frac{M_{weld}}{V_{weld}} \tag{14.8-9}$$

where

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

$$= \frac{M_{pr}}{l_b - l_p} + V_{gravity} \tag{14.8-10}$$

and where

M_{pr} = $C_{pr}R_yF_yZ_{beam}$
$V_{gravity}$ = beam shear force resulting from the load combination $1.2D + f_1L + 0.2S$ (where f_1 is the load factor determined by the local building code for live loads, but not less than 0.5), kips (N)

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

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

$$= \frac{t_w T^2}{4} \qquad (14.8\text{-}11)$$

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

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

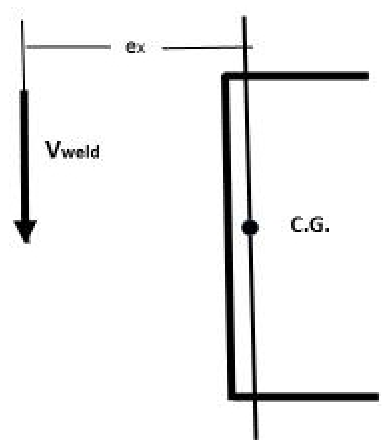


Fig. 14.4. Eccentrically loaded weld group.

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

The required strength of the weld connecting the shear plate to the column flange shall be equal to the nominal strength of the eccentrically loaded weld group as calculated according to Step 3.

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

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

$$M_f = M_{pr} + V_{beam} l_p \qquad (14.8\text{-}12)$$

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

Step 8. Check continuity plate requirements according to Section 2.4.4

Step 9. Check column panel zone according to Section 14.4

Step 10. Check column-beam moment ratio according to Section 14.4