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DIVISION: 05 00 00—METALS

SECTION: 05 12 00—STRUCTURAL STEEL FRAMING

REPORT HOLDER:

SEISMIC STRUCTURAL DESIGN ASSOCIATES, INC.

**791 E. WASHINGTON BOULEVARD
LOS ANGELES, CALIFORNIA 90021**

EVALUATION SUBJECT:

SLOTTED WEB™ BEAM-TO-COLUMN STEEL MOMENT FRAME CONNECTION



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DIVISION: 05 00 00—METALS
Section: 05 12 00—Structural Steel Framing

REPORT HOLDER:

SEISMIC STRUCTURAL DESIGN ASSOCIATES, INC.
791 EAST WASHINGTON BOULEVARD
LOS ANGELES, CALIFORNIA 90021
(213) 494-0446
www.slottedweb.com

EVALUATION SUBJECT:

SLOTTED WEB™ BEAM-TO-COLUMN STEEL MOMENT FRAME CONNECTION

1.0 EVALUATION SCOPE

Compliance with the following codes:

2012 and 2009 *International Building Code*® (IBC)

Property evaluated:

Structural design

2.0 USES

The Seismic Structural Design Associates (SSDA) Slotted Web™ Beam-to-Column Special Steel Moment Frame (SMF) Connection (referred to hereafter as the SSDA Slotted Web™ SMF connection) provides a beam-to-column connection in structural steel moment frames.

3.0 DESCRIPTION

3.1 General:

The SSDA Slotted Web™ SMF connection provides beam-to-column moment resisting connections for use in steel Special Moment Frame (SMF) systems. This proprietary moment connection satisfies all applicable requirements of the IBC and of ANSI/AISC 341-10, including Sections E3 and K1 for the 2012 IBC (ANSI/AISC 341-05, including Section 9 and Appendix P for the 2009 IBC). The system also meets the prequalification requirements of Sections 3.1 and 3.4 of the ICC-ES Acceptance Criteria for Steel Moment Frame Connection Systems (AC129). The connection system features wide flange (W-shape) steel beams with two slots in the web, parallel and adjacent to each flange. The beams are welded at the web and flanges to flanges of wide flange (W-shape) steel columns with complete-joint-penetration (CJP) groove welds. The single shear plate is welded to the column flange with a partial-joint-penetration or a CJP groove weld, as required for strength, and to the beam web with a fillet weld.

Continuity plates and doubler plates, as applicable in accordance with Section 4.0, are provided at the column flanges and web as illustrated in Figure 1.

3.2 Material:

3.2.1 Structural Steel Beams and Columns: Structural steel for beams and columns must conform to ASTM A992 Grade 50 for beams, and ASTM A913 Grade 65 or ASTM A992 Grade 50 for columns.

3.2.2 Plates: The steel plates must be fabricated from structural steel complying with ASTM A572 Grade 50.

3.2.3 Welds: All CJP groove welds are designated as *demand critical welds*, and must be made with a filler metal complying with requirements of AWS D1.8: 2009, Clause 6.3, and capable of providing a minimum Charpy V-Notch (CVN) toughness of 20 ft-lb (27 J) at -20°F (-29°C) as determined by the AWS A5.20 classification test method, and 40 ft-lb (54 J) at 70°F (21°C) as determined by Annex A of AWS D1.8:2009 for the 2012 IBC (Appendix X of AISC 341-05 for the 2009 IBC). Weld filler metal must be E70-T6 H8 for welds at beam top and bottom flanges, and E71-T8 H8 for other CJP groove welds and fillet welds. Fillet welds must be made with a filler metal complying with requirements of AWS D1.8: 2009, Clause 6, capable of providing a minimum Charpy V-Notch (CVN) toughness of 20 ft-lb (27 J) at -20°F (-29°C) as determined by the AWS A5.20 classification test method.

3.2.4 Bolts, Washers and Nuts: Bolts, washers and nuts must conform to Section A 3.3 of AISC 360 and AISC 348. Bolts must comply with ASTM A325 or ASTM A490.

4.0 DESIGN AND INSTALLATION

4.1 Structural Design and Prequalification Limits:

The SSDA Slotted Web™ SMF connections are prequalified for use in steel Special Moment Frames (SMFs), within the limits noted in this report.

The axial loading in beams is not part of the connection prequalification requirements, which is consistent with AISC 341 and AISC 358. However, all steel moment frame members and their connections must be designed to resist all applicable load combinations prescribed in IBC Section 1605, including those load combinations involving axial tension and compression loading.

The structural design procedures must be in accordance with Chapters 16 and 22 of the IBC as Load and Resistance Factor Design (LRFD). Steel moment frames using the SSDA Slotted Web™ SMF connection must be designed using a capacity design approach as identified in

Commentary to Section A3.2 of AISC 341-10 for the 2012 IBC (Section 6.2 of AISC 341-05 and its commentary for the 2009 IBC). Strength demands at the critical sections must be determined by calculation, applying statics that consider the effects of the probable maximum bending moment and corresponding shear at the plastic hinge location. CJP groove welds must be used between beam flanges and column flange, and between beam web and column flange. The shear plate must be connected to the column flange using either a PJP or CJP welds, depending on the strength requirements, and connected to the beam web with fillet welds as illustrated in Figure 1. For determining seismic loads, the system seismic performance coefficients and factors for the IBC are to be as follows:

SEISMIC SYSTEM*	RESPONSE MODIFICATION COEFFICIENT	SYSTEM OVERSTRENGTH FACTOR, Ω_0	DEFLECTION AMPLIFICATION FACTOR, C_d
Special Steel Moment Frames	8	3	5 ^{1/2}

*Seismic force-resisting system as defined in ASCE 7-10 and ASCE 7-05, Table 12.2-1.

In addition, compliance with the American Welding Society (AWS) Structural Welding Code–Steel (ANSI/AWS D1.1:2010 for the 2012 IBC and D1.1:2004 for the 2009 IBC), with modifications as set forth in AISC 360 Section J2, and Section 3.2.3 of this report, is required. The design must also take into account requirements set forth in Sections 4.1.1 through 4.1.17 of this report.

4.1.1 Beam Slot Dimensions: The beam slots must terminate at 1^{1/16}-inch-diameter (27 mm) holes for beams 24 inches (610 mm) deep and greater or 1^{3/16}-inch-diameter (21 mm) holes for beams less than 25 inches (610 mm) deep. The beam slot length, l_s , must meet the least of the following equations (within ±10%):

ENGLISH UNITS	SI UNITS
$l_s = 1.5b_f$	$l_s = 1.5b_f$
$= 102t_f / (F_{ye})^{1/2}$	$= 268t_f / (F_{ye})^{1/2}$
$= \frac{d}{2}$	$= \frac{d}{2}$
$= l_p + (l_b - l_p) / 10$	$= l_p + (l_b - l_p) / 10$

where:

- b_f = Beam flange width, inches (mm).
- d = Beam depth, inches (mm).
- F_{ye} = Expected yield strength of steel beams, ksi (MPa).
 $= R_y F_y$
- l_s = Beam slot length, inches (mm).
- t_f = Beam flange thickness, inches (mm).
- l_b = One-half clear span (between column flanges) length of beam, inches (mm).
- l_p = Width (along the beam span direction) of shear plate, inches (mm).
- R_y = Ratio of the expected yield stress to the specified minimum yield stress, in accordance with Table A3.1 of AISC 341-10 for the 2012 IBC (Table I-6-1 of AISC 341-05 for the 2009 IBC), ksi (MPa).
- F_y = Specified minimum yield stress of the type of the steel to be used, ksi (MPa).

Slot widths are 1/8 inch (3.2 mm) along the shear plate and 1/4 inch (6.4 mm) for the remainder of the length, as illustrated in Figure 1.

4.1.2 Plastic Hinge Location: For calculation purposes, the web plastic hinge location, l_{hinge} , on the beam or girder, must be determined as follows:

$$l_{hinge} = (l_b - l_p) [(Z_b - Z_f) / (3Z_b)]$$

where:

- l_{hinge} = Web plastic hinge length, measured from end of shear plate toward beam center, inches (mm).
- Z_b = Plastic section modulus of beam cross section, in.³ (mm³).
- Z_f = Plastic section modulus of beam flanges, in.³ (mm³).

Other terms are defined in Section 4.1.1 of this report.

The centroid of the beam plastic hinge region, that consists of beam flanges from end of slots to column face, the shear plate, and the beam web region from end of slots to the column face, shall be taken at the end of the shear plate.

4.1.3 Plastic Moment Capacity at Hinge(s): Plastic moment capacity at the plastic hinge location, which does not include the effects of strain hardening, must be calculated as follows:

$$M_p = Z_b F_{ye}$$

where:

- M_p = Plastic moment capacity at hinge, kip-in. (N-mm).
- Z_b = as defined in Section 4.1.2 of this report.
- F_{ye} = $R_y F_y$

where:

- R_y = ratio of the expected yield stress to the specified minimum yield stress F_y .
- F_y = specified minimum yield stress of the type of steel to be used in the yield element, ksi (MPa).

4.1.4 Probable Maximum Moment at Hinge(s): Probable maximum moment at the beam plastic hinge location, which includes the effects of strain hardening, must be calculated using the following equation:

$$M_{pr} = C_{pr} Z_b F_{ye}$$

where:

- M_{pr} = Probable maximum moment at hinge, kip-in. (N-mm).
- C_{pr} = Factor to account for the peak connection strength, including strain hardening effect, must equal to 1.1.
- Z_b = as defined in Section 4.1.2 of this report.
- F_{ye} = as defined in Section 4.1.3 of this report.

4.1.5 Moment at Colum Center Line(s): The moment must be calculated as follows:

$$M_c = M_{pr} + V_u \cdot \left(\frac{d_{col}}{2} + l_p \right)$$

$\left(\frac{d_{col}}{2} + l_p \right)$ is the distance between the centroid of beam plastic hinge and column center line.

4.1.6 Adequacy of Connection Flexural Capacity: The connection must develop the full ductile capacity (M_{pr}) of the beam.

4.1.7 Required Shear Strength at Connection: The required shear strength V_u of beam and beam web-to-column connection must be determined using the following equation:

$$V_u = \frac{2M_{pr}}{L'} + V_{gravity}$$

where:

- V_u = required shear strength of beam and beam-to-column connection, kips (N).
- L' = distance between beam web plastic hinges, inches (mm).
- M_{pr} = maximum probable moment at hinge, kip-in (N-mm). See Section 4.1.4.
- $V_{gravity}$ = beam shear force resulting from the following factors $(1.2D + f_1L + f_2S)$, as defined in Equation 16-5 of the IBC, kips (N).

In addition, the design shear strength of the beam must be checked according to Chapter G of AISC 360, $\phi V_n \geq V_u$.

4.1.8 Column Panel Zone: The column panel zone must comply with Section E3.6e of AISC 341-10 and Section J10.6(b) of AISC 360-10 for the 2012 IBC (Section 9.3b of AISC 341-05 and Section J10.6(b) of AISC 360-05 for the 2009 IBC). The contribution of panel zone deformation to overall story drift must be considered in accordance with ASCE 7 Section 12.7.3.

4.1.9 Shear Plate Design: The width l_p is 4 to 6 inches (102 to 152 mm). The height is determined as follows:

$$h_p = T - 2 \text{ inches} \quad \text{SI: } h_p = T - 51 \text{ mm}$$

where:

- h_p = Required shear plate height, inches (mm).
- T = Distance between web toes of fillets at top and bottom of the beam web, inches (mm).

The thickness, t_p , must be determined using the following equation:

$$t_p = \left(\frac{6}{h_p^2} \right) \left[\frac{Z_b l_p}{(l_b - l_p)} \right]$$

where:

- t_p = Required shear plate thickness, inches (mm).
- Z_b = Plastic section modulus of the beam, in.³ (mm³)

The thickness of the plate must be at least $\frac{3}{8}$ inches (10 mm) or at least $\frac{2}{3}$ of the beam web thickness, whichever is greater. For other terms, see Section 4.1.1 of this report.

4.1.10 Shear Plate Welding Design: The shear plate must be welded to the beam web with a C-shaped fillet weld that has been designed to resist M_{weld} , V_{weld} , and the resulting eccentricity, e . These values are determined using the following equations:

$$M_{weld} = \text{moment resisted by shear plate, lbf-in (N-mm)}$$

$$= 1.1 \left[\frac{t_p}{(t_w + t_p)} \right] Z_{web} F_{ye}$$

$$V_{weld} = \text{shear resisted by shear plate, lbf (N)}$$

$$= \left[t_p / (t_w + t_p) \right] V_{beam}$$

e = eccentricity of shear plate (in. or mm)

$$e = M_{weld} / V_{weld}$$

V_{beam} = Shear at face of column, kips (kN)

$$= \frac{M_p}{l_b - l_p} + \frac{1.2D + f_1L + f_2S}{2}$$

where:

- t_p = as defined in Section 4.1.9 of this report.
- F_{ye} = as defined in Section 4.1.3 of this report.
- D = Shear at column face resulting from unfactored dead loads, kips (kN).
- L = Shear at column face resulting from unfactored live loads, kips (kN).
- S = See Section 1605.2 of the IBC.
- t_w = Thickness of beam web, inches (mm).
- f_1 & f_2 = As defined in Equation (16-5) of the IBC.
- M_p = as defined in Section 4.1.3 of this report.
- l_b = as defined in Section 4.1.1 of this report.
- l_p = as defined in Section 4.1.1 of this report.
- Z_{web} = Plastic section modulus of beam web, in³ (m³).
- = $t_w T^2 / 4$

where:

- T = as defined in Section 4.1.9 of this report.

4.1.11 Continuity Plate Requirements: Design of the continuity plates is the responsibility of the registered design professional and must meet the requirements of Section E3.6f of AISC 341-10 for the 2012 IBC (Section 7.5 of AISC 341-05 and Section 2.4.4 of AISC 358-05s1-09 for the 2009 IBC).

4.1.12 Beam Limitations: In accordance with requirements in AISC 341-10 for the 2012 IBC (AISC 341-05 for the 2009 IBC), beams must satisfy the following limitations:

1. Beams must be rolled wide-flange shapes or built-up sections prescribed in Section 2.3 of ANSI/AISC 358-10 including AISC 358s1-11. Material for plates and welds for built-up beams must conform to Section 3.2 of this report.
2. Beam depth is limited to the maximum allowed for a W36.
3. Beam weight is limited to a maximum of 400 lbs/ft (600 kg/m).
4. Beam flange width and thickness must comply with Sections E3.5 and D1.1 of AISC 341-10 for the 2012 IBC (Section 9.4 of AISC 341-05 for the 2009 IBC). Beam flange thickness is limited a maximum of $2\frac{1}{4}$ inches (57 mm).
5. Beams must be full length between Slotted Web™ Steel Moment Frame Connections.
6. The ratio of the distance between beam hinge centerlines over the beam depth must be limited to 6.5 or greater.
7. Lateral bracing of beams must be provided as follows: The beam must be braced in conformance with Sections E3.5 and D1.2 of AISC 341-10 for the 2012 IBC (Section 9.8 of AISC 341-05 for the 2009 IBC), where the length of the beam is defined as the distance between the ends of the Slotted Web™ connections.

As tested, and in conformance with AISC 341, no supplemental lateral bracing is required at or near the plastic hinge.

4.1.13 Lateral Bracing Requirements: Lateral bracing must comply with Sections E3.4b and E3.4c of AISC 341-10 for the 2012 IBC (Sections 9.7 and 9.8 of AISC 341-05 for the 2009 IBC), except that no beam end lateral bracing (at or near the plastic hinge) is required.

4.1.14 Panel Zone Deformation: The contribution of panel zone deformations to overall story drift must be determined so as to comply with ASCE 7, Section 12.7.3, and AISC 360, Section J10.6.(b).

4.1.15 Column Limitations: Columns must satisfy the following limitations:

1. Columns must be rolled wide-flange shapes or built-up sections prescribed in Section 2.3 of AISC 358-10 including AISC 358s1-11. Materials for plates and welds for built-up columns must conform to Section 3.2 of this report.
2. The column depth is limited to a maximum W36 (W920).
3. The column is limited to a maximum weight of 710 lb/ft (1057 kg/m).
4. Column orientation: beam is connected to column flange.
5. Width-thickness ratios for the flanges and web of columns must conform to the limits of Sections E3.5a and D1.1 of AISC 341-10 for the 2012 IBC (Section 9.4 of AISC 341-05 for the 2009 IBC).
6. Lateral bracing of the columns must be provided in accordance with Section E3.4c of AISC 341-10 for the 2012 IBC (Section 9.7 of AISC 341-05 for the 2009 IBC).

4.1.16 Column-Beam Relationship Limitations:

1. The design shear strength of the column web must be determined in accordance with Section E3.6e of AISC 341-10 and Section J10.6b of AISC 360-10 for the 2012 IBC (Section 9.3 of AISC 341-05 and Section J10.6b of AISC 360-05 for the 2009 IBC).
2. Column-beam ratios must be limited as follows:

The column-beam ratio must be satisfied by applying Eq-2:

$$\frac{\sum M_{pc}^*}{\sum M_{pb}^*} > 1.0 \quad (\text{Eq-2})$$

where:

$\sum M_{pc}^*$ = The sum of the projections of the nominal flexural strengths (M_{pc}) of the column above and below the connection joint, at the intersection of the beam and column centerline with a reduction for the axial force in the column. The nominal flexural strength of the column may be computed in accordance with Eq-3:

$$\sum M_{pc}^* = \sum Z_c (F_{yc} - P_u/A_g) \quad (\text{Eq-3})$$

where:

Z_c = The plastic section modulus of the column (in.³ or m³).

F_{yc} = The minimum specified yield strength of the column at the connection (psi or Pa).

P_u/A_g = Ratio of column axial compressive load, computed in accordance with load and

resistance factor provisions, to gross area of the column (psi or Pa).

$\sum M_{pb}^*$ = The sum of the projections of the expected flexural strengths of the beams at the web plastic hinge locations to the column centerline. The location of the web plastic hinges in the beam must be determined in accordance with Section 4.1.2 of the report. The expected flexural strength of the beam may be computed in accordance with Eq-4:

$$\sum M_{pb}^* = \sum (1.1R_y F_{yb} Z_b + M_v) \quad (\text{Eq-4})$$

R_y = Adjustment coefficient for material over-strength, in accordance with Table A3.1 of AISC 341-10 for the 2012 IBC (Table I-6-1 of ANSI/AISC 341-05 for the 2009 IBC).

F_{yb} = Specified minimum yield strength of the beam (psi or Pa).

Z_b = Plastic modulus of the beam section (in.³ or m³).

M_v = Additional moment due to shear amplification from the centroid of the beam hinge to the column centerline (lb-in. or N-m). See Section E3.4a of AISC 341-10 for the 2012 IBC (Section 9.6 of AISC 341-05 for the 2009 IBC).

Strength demands at the critical load transfer locations through the Slotted Web™ beam-to-column connection and column must be determined by superimposing M_{pb}^* computed based on the known beam hinge centroid location, and then ramping up the moment demand at each critical section, based upon the span geometry.

4.1.17 Protected Zones: The protected Zones consist of (1) the portion of the beam web between the face of the column to the end of the slots plus one half the nominal depth of the beam beyond the slot end, and (2) the beam flange from the face to the column to the end of the slot plus one-half the nominal beam flange width. The Protected Zones must meet the requirements set forth in Section E3.5c of AISC 341-10 for the 2012 IBC (Section 7.4 of AISC 341-05 for the 2009 IBC). Field modification of the moment connection within this zone is not permitted.

4.2 Welding:

Welding must be in accordance with Section E3.6a of AISC 341-10 for the 2012 IBC (Section 7.3 and Appendix W of AISC 341-05 for the 2009 IBC). Welding must be performed in accordance with a welding procedure specification (WPS) as required in AWS D1.1 and approved by the engineer of record. A WPS in conformance with Clause 4 of AWS D1.1 must be developed for fabrication of each and every different welding application, with respect to position, welding process, electrode manufacturer, trade name of the filler metal for the selected electrode type, and welding parameters required to complete the fabrication. To apply prequalified processes, the WPS must be in conformance with Clause 3 of AWS D1.1 and Clause 6.1 of AWS D1.8:2010 for the 2012 IBC (AWS D1.8:2005 for the 2009 IBC). The WPS variables must be within the parameters established by the filler metal manufacturer. A WPS that is not in conformance with Clause 3 of AWS D1.1 must be based on a documented Procedure Qualification Record (PQR), in accordance with Clause 4 of AWS D1.1, which is subject to the approval of the engineer of record.

4.3 Fabrication:

All components of the SSDA Slotted Web™ SMF connection must be manufactured by an approved fabricator complying with Section 1704.2.5.2 of the 2012

IBC (Section 1704.2.2 of the 2009 IBC). Compliance with the IAS Accreditation Criteria for Fabricator Inspection Programs for Structural Steel (AC172) is deemed to equal compliance with Section 1704.2.5.2 of the 2012 IBC (Section 1704.2.2 of the 2009 IBC). In addition to complying with applicable codes such as the IBC, AISC 360, AISC 303, AISC 341, AWS D1.1 and AWS D1.8, the approved fabricator must demonstrate that the following slot tolerances can be consistently maintained:

$\frac{1}{8}$ -inch-wide (3.2 mm) slot: + $\frac{1}{16}$ inch, - 0 inch (+ 2 mm, - 0 mm) to end of shear plate;

$\frac{1}{4}$ -inch-wide (6.4 mm) slot: + $\frac{1}{8}$ inch - 0 inch (+ 3.2 mm, 0 mm) from end of shear plate to the slot determination hole; $\frac{1}{16}$ -inch-diameter (27 mm) termination hole: + $\frac{1}{16}$ inch, - 0 inch (+ 1.6 mm, - 1.6 mm); $\frac{3}{16}$ -inch (21 mm) termination hole: + $\frac{1}{16}$ inch, - 0 inch (+ 1.6 mm).

Welding must be in accordance with Section 4.2 of this report.

The beam web slots shall be made using *thermal cutting* to produce a surface roughness not exceeding 1000 micro-inches. Gouges and notches that may occur in the thermally cut slots may be repaired by grinding. The beam slots terminate at $\frac{1}{16}$ -inch-diameter (27 mm) holes for beams 24 inches (610 mm) deep or greater, or $\frac{13}{16}$ -inch diameter (21 mm) holes for beams less than 24 inches (610 mm) deep. Slot widths are nominally $\frac{1}{8}$ inch (3.2 mm) along the shear plate and $\frac{1}{4}$ inch (6.4 mm) for the remainder of the length, with a 1:6 slope to make the transition.

Refer to Figure 1 for an illustration of a typical SSSA Slotted Web™ SMF Connection detail.

4.4 Erection:

Erection of the SSSA Slotted Web™ SMF connection must conform with AISC 360-10 including Chapter N, AISC 303-10, AISC 341-10 including Chapters I and J, AWS D1.1:2010, and AWS D1.8:2009 for the 2012 IBC (AISC 360-05 including Chapter M, AISC 303-05, AISC 341-05 including Appendices Q and W, and AWS D1.1:2004, and AWS D1.8:2005 for the 2009 IBC); and must be consistent with the requirements noted in Chapters 17 and 22 of the IBC.

Shear plates are attached to the beam web typically with $\frac{3}{4}$ -inch-diameter (20 mm) or $\frac{7}{8}$ -inch-diameter (22 mm) A 325 or A 490 high-strength erection bolts at approximately 6 inches (150 mm) on center that are fully tensioned in the field as determined by the erector/contractor and then welded using fillet welds. All welding must be in accordance with Section 4.2 of this report.

Shop preparation and field erection of the system includes the following installation procedures:

1. Termination holes are shop-drilled in the web of the beam. Refer to Section 4.1.1 for dimensions. Automatic flame cutting may be utilized to produce the slot termination hole.
2. Slots are partially flame-cut in an approved fabricator's shop starting at "Y" distance (as shown on the connection detail Figure 1) from the end of the beam, and the flame cut is continued to the slot termination hole. No grinding of the flame-cut slots is required. The slot at k-line must be tangent to the top of the slot termination hole.
3. The beam is installed and all bolts are fully tensioned in the field to the preinstalled shear plate.
4. Preheating and post-heating must conform to the requirements of AWS D1.1.

5. The beam top and bottom flanges are welded to the column flange with CJP groove welds.
6. The beam web is welded to the column flange along the full height of the shear plate with a CJP groove weld.
7. Welds are inspected by Nondestructive Testing (NDT) in accordance with Section 4.5 of this report.
8. The shear plate is welded to the beam web with a fillet weld. The fillet weld is visually inspected in accordance with Section 4.5 of this report.
9. The slots are completed by flame-cutting from "Y" to the end of the beam web.
10. The backup bar is removed at the bottom beam flange, and fillet weld reinforcement is provided with size equal to $\frac{1}{4}$ the beam flange thickness not less than $\frac{1}{4}$ inch (6 mm) nor more than $\frac{3}{8}$ inch (10 mm). The weld is inspected in accordance with Section 4.5 of this report.
11. If building erection can be accomplished by initially cutting the beam slot to full length, the staged cutting of the slot is eliminated and fabrication is completed as shop-only.
12. Visual welding inspection must be performed before welding, during welding, and after welding, and must be in compliance with Section J6.1 of AISC 341-10 for the 2012 IBC (Appendix Q, Section Q5.1, of AISC 341-05 for the 2009 IBC).
13. Nondestructive Testing (NDT) of welds must comply with Section J6.2 of AISC 341-10 for the 2012 IBC (Appendix Q, Section Q5.2, of AISC 341-05 for the 2009 IBC).

Refer to Figure 1 for a typical SSSA Slotted Web™ SMF Connection detail.

4.5 Quality Assurance:

A plan for quality assurance conforming to Sections 1704 and 1705 of the 2012 IBC (sections 1704 through 1707 of the 2009 IBC) must be included in the structural design/construction document prepared by a registered design professional and approved by the code official.

Special inspection for steel construction must conform to Sections 1704.2.5, 1705.2, and 1705.11.1 of the 2012 IBC, and Chapter J of the AISC 341-10 for the 2012 IBC (Sections 1704.2, 1704.3, and 1707.2 of the 2009 IBC, and Part 1 Section 18 and Appendix Q of AISC 341-05 for the 2009 IBC), and must be included in the approved quality assurance plan. Special inspection must verify compliance of steel with specifications; steel identification; material verifications and identifications of high-strength bolts, nuts and washers; qualification of welders; use of appropriate welding materials; storage conditions for welding materials, bolts, washers and nuts; welded joint preparation; conformance of welding procedures with approved WPS and applicable provisions of AWS D1.1, AISC 360 and AISC 341; fabrication tolerances; and steel frame joint details in conformance with approved construction documents. Inspections must include compliance with Sections J6 through J8 of AISC 341-10 for the 2012 IBC (Section Q5, Appendix Q of AISC 341-05 for the 2009 IBC); with the exception that those requirements for Reduced Beam Section (RBS) must be replaced with corresponding requirements for the slots/termination holes.

In Seismic Design Categories C, D, E and F, visual inspection and nondestructive testing (NDT) in accordance with Section 1705.12.2 of the 2012 IBC; Clause 4.9.2 of AWS D1.1:2010 and Clause 7 of AWS D1.8:2009 for the

2012 IBC (Section 1708.3 of the 2009 IBC; Clause 4.8.2 of AWS D1.1:2004 and Clause 7 of AWS D1.8:2009 for the 2009 IBC) must be performed. Personnel for visual inspection and NDT testing must be qualified in accordance with Section J4 of AISC 341-10.

In addition to the requirements set forth above, precautions for preventing the occurrence of delayed hydrogen cracking of completed welds, set forth below, must be followed.

The phenomenon of delayed cracking has been known to occur infrequently and is due to the presence of free hydrogen in the weld zone. This hydrogen can have its source in the base metal or weld filler metal, or can be introduced into the weld from atmospheric moisture (rainwater) or organic contaminants. In order to limit the frequency of these missteps, the following precautions must be taken:

1. Flux core weld filler metal of low hydrogen percentage, as required by Section 3.2.3 of this report, from moisture-resistant sealed containers, must be utilized.
2. After flux core weld filler metal has been withdrawn from the shipping wrapper and left exposed to the atmosphere for more than four hours, at least 4 feet (1219 mm) of wire prior to initiation of welding must be withdrawn and disposed of, to eliminate flux absorbed moisture contamination.
3. Prior to initiation of welding, the connection elements must be preheated per AWS D1.1 Table 3.2. To accomplish a proper preheat, the column flange backside and the girder flange underside are heated. After preheat is accomplished, welding commences. Interpass temperature must be maintained per AWS D1.1 while welding.
4. If temperature is below 50°F (10°C) or wind speed is more than 5 mph, the completed weld must be wrapped with an insulating blanket to provide for a slow cooling of the completed weldment. Heat application at a lower intensity than preheating methods may be used to effect a slow cooling of the weld.
5. Ultrasonic testing (UT) for weld volumetric flaws may proceed as soon as possible after welds are completed. A check for delayed cracking of welds from hydrogen embrittlement must be made after the building structure is substantially loaded. Only the lower flanges need be tested and at the rate of 25 percent of all SSDA Slotted Web™ connection welds, provided that this reduced percent of UT test is not in conflict with Chapter J of AISC 341-10 for the 2012 IBC (Appendix Q of AISC 341-05 for the 2009 IBC). After concrete floors on the completed frame are in place, the ultrasonic recheck for delayed cracking may commence.
6. Run-off Tabs (Weld Tabs) and Back-up Bars: Run-off tab may be left in place so long as the welds are slag-free and the weld profile forms a smooth transition from the end of the tab to the beginning of the design weld profile.
7. Only the back-up bar of the bottom flange of the SSDA Slotted Web™ Beam Connection need be removed and the weld refinished per accompanying design

detail drawings. The bottom flange weld back-up bar removal and repair must be in accordance with Section 3.3 of AISC 358-10 including AISC 358s1-11.

Bolts in the Web of SSDA Slotted Web™ connections are erection bolts and do not require special inspection. Bolts should be installed in accordance with Section J3 of AISC 360 so as to support erection loads. Surfaces of connections should be clean and burr-free to allow for fit-up.

5.0 CONDITIONS OF USE

The SSDA Slotted Web™ SMF Connection described in this report complies with, or is a suitable alternative to what is specified in, the code indicated in Section 1.0 of this report, subject to the following conditions:

- 5.1 The SSDA Slotted Web™ SMF Connection must be designed by a registered design professional with design procedure identified in Section 4.1 of this report, and must be approved by the code official.
- 5.2 The steel SMF, utilizing the SSDA Slotted Web™ SMF Connection, must be normally enclosed and maintained at a temperature of 50°F (10°C) or higher, as prescribed in Section A3.4 of AISC 341-10 and AWS D1.8 subclause 6.3.6 for the 2012 IBC (Section 7.3b of AISC 341-05 for the 2009 IBC).
- 5.3 The scope of prequalification of the SSDA Slotted Web™ SMF Connection must be limited to the details provided in Sections 4.1.12, 4.1.15 and 4.1.16 of this report.
- 5.4 Structural design drawings and specifications, shop drawings and erection drawings must comply with Sections A4 and I1 of AISC 341-10 for the 2012 IBC (Section 5 of AISC 341-05 for the 2009 IBC).
- 5.5 Fabrication must comply with Sections 4.2 and 4.3 of this report, and must be performed by an approved fabricator, as described in Section 1704.2.5.2 of the 2012 IBC (1704.2.2 of the 2009 IBC), in the approved shop facility.
- 5.6 Erection must be in accordance with Section 4.2 and 4.4 of this report and the approved engineering plan, prepared by a registered design professional and approved by the code official.
- 5.7 Quality assurance must be provided in accordance with Section 4.5 of this report.

6.0 EVIDENCE SUBMITTED

Data in accordance with the ICC-ES Acceptance Criteria for Steel Moment Frame Connection Systems (AC129), dated October 2012.

7.0 IDENTIFICATION

Material Identification must be in accordance with Section 2203 of the IBC, and mill certificates must be provided for all steel plates, steel shapes, weld filler material and fasteners.

Steel must be marked or indent-stamped to demonstrate compliance with design requirements.

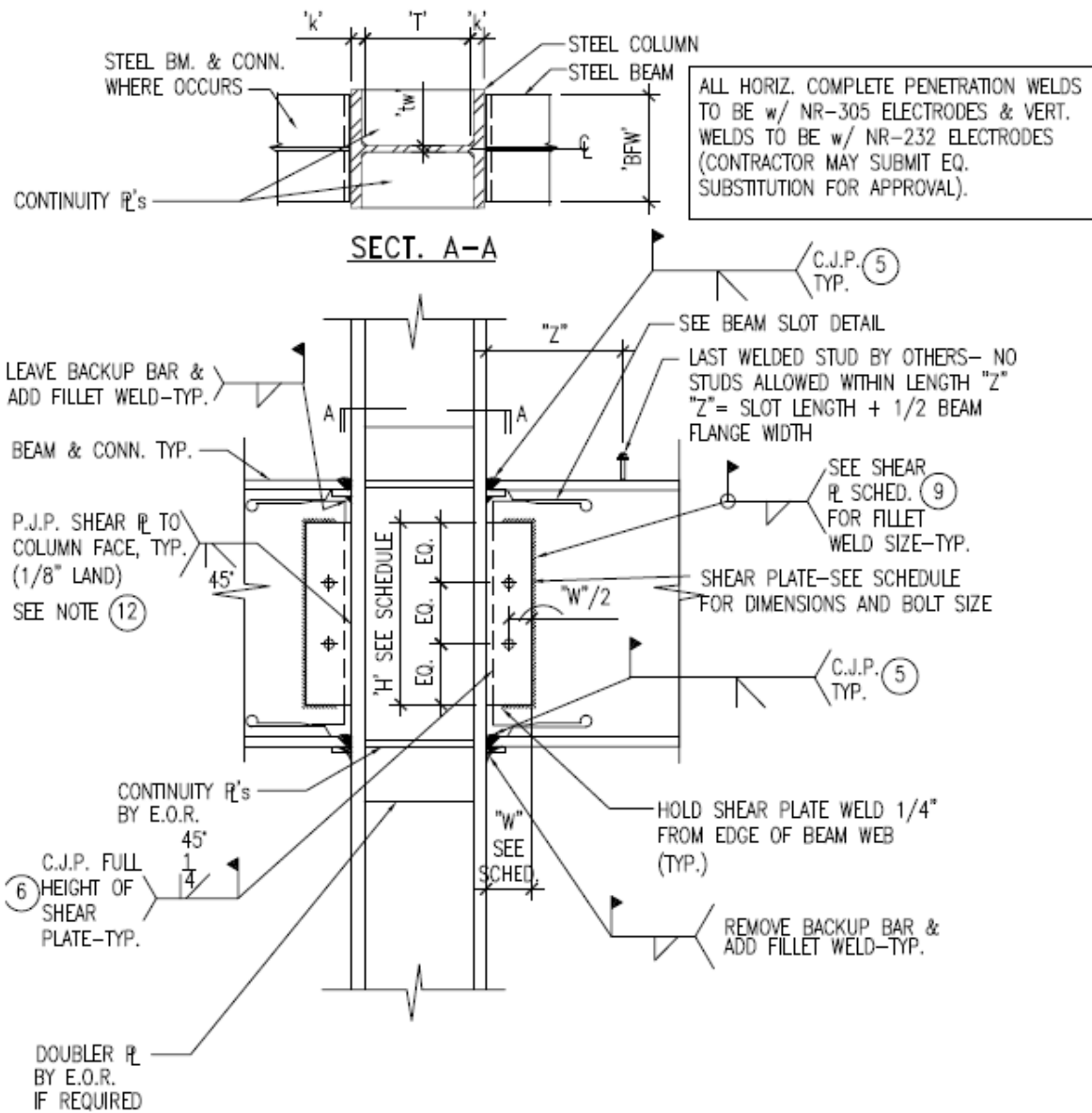


FIGURE 1—SSDA SLOTTED WEB™ SMF CONNECTION

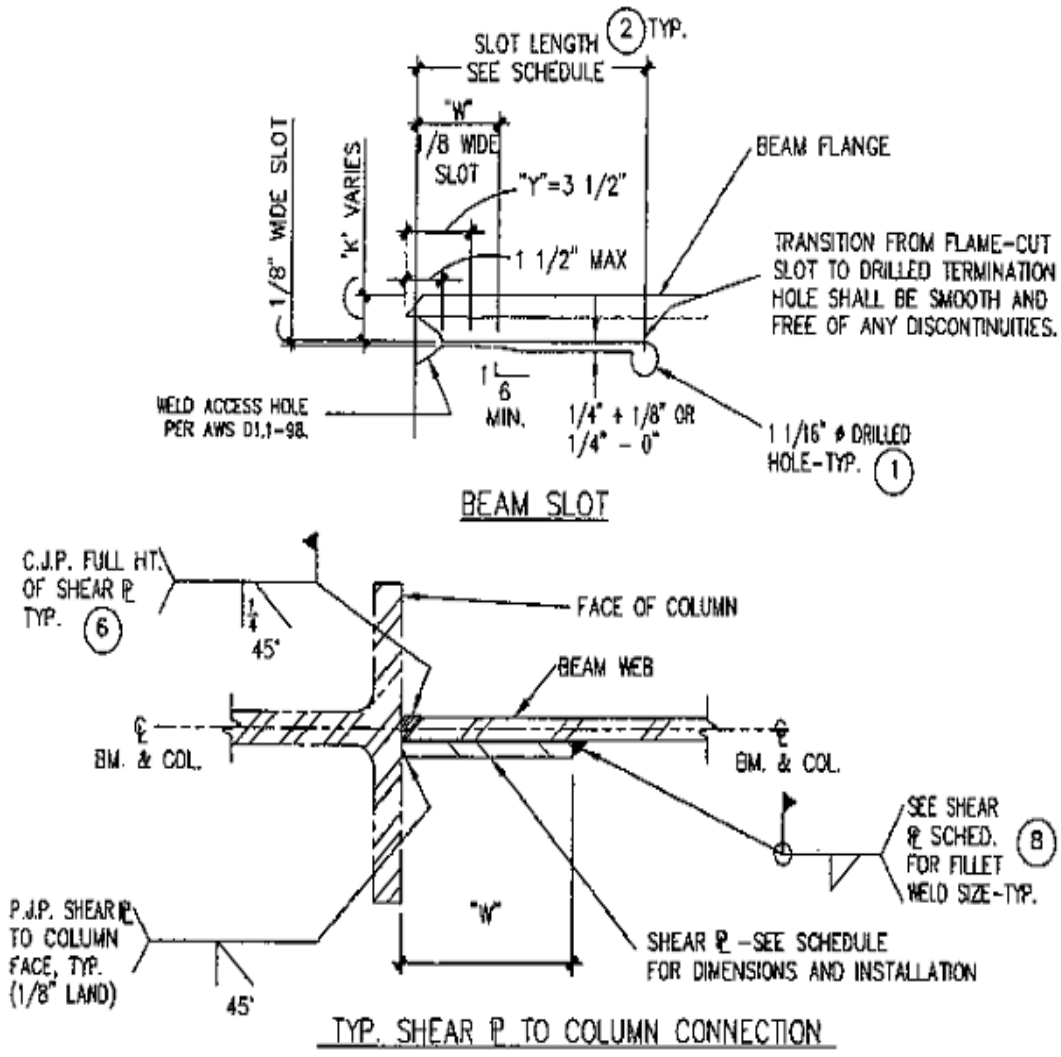


FIGURE 1—SSSA SLOTTED WEB™ SMF CONNECTION (Continued)