LEGACY REPORT

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Legacy report on the 1997 *Uniform Building Code*™

DIVISION: 05—METALS
Section: 05120—Structural Steel

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

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1.0 SUBJECT

Slotted Web™ Beam-to-Column Steel Moment Frame Connection.

2.0 DESCRIPTION

2.1 General:

The Slotted Web™ Beam-to-Column Steel Moment Frame Connection provides a beam-to-column moment connection in structural steel. This connection satisfies the Special Moment-Resisting Frame (SMRF) requirements in Section 2213.7 of the 1997 *Uniform Building Code*™ (UBC). The connection features wide flange steel beams with a slot in the web, parallel and adjacent to each flange. These beams are welded at the web and flanges to wide flange steel columns with full-penetration welds. Continuity plates and shear doubler plates, as applicable, are provided at the column webs. Illustrative details for connections are provided in Figure 1.

2.2 Materials:

2.2.1 Structural Shapes: Structural steel shapes include wide flange shapes conforming to ASTM A 6, and Chapter 22, Divisions II and III, of the UBC. Specifications include ASTM A 36, ASTM A 572, Grade 50, ASTM A 992 or equivalent.

2.2.2 Plates: The steel plates must be fabricated from structural steel that complies with ASTM A 36, ASTM A 572, Grade 50, ASTM A 588 or equivalent.

2.2.3 Welds: All fillet welds are produced using E70 electrodes. The maximum effective throat thickness is $^1/_{16}$ inch (2 mm) less than the shear plate thickness. Weld filler metal and associated welding processes for all fillet welds shall comply with AWS specifications.

Weld metal shall achieve a minimum notch toughness of 20 ft.-lbs. at -20°F (27 N-m at -29°C) and 40 ft.-lbs. at 70°F (54 N-m at 21°C). Requirements are set forth in Annex III, American Welding Society (AWS) D1.1-98.

2.2.4 Bolts: Bolts must comply with ASTM A 325 or ASTM A 490.

2.3 Structural Design:

The structural design procedures shall be in accordance with Chapter 22, Divisions II, III, IV, and V, of the UBC as Load and Resistance Factor Design (LRFD). No lateral bracing of columns is required due to the absence of lateral torsional buckling in the slotted web beam design. In addition, compliance with AWS D1.1-98, Section 2, is required. The design shall also involve Sections 2.3.1 through 2.3.5 of this report.

2.3.1 Beam Slot Dimensions: The beam slots shall terminate at $1^1/_{16}$-inch-diameter (27 mm) holes for beams 24 inches (610 mm) deep and greater or $^{13}/_{16}$-inch (21 mm) holes for beams less than 24 inches (610 mm) deep. The beam slot length, l_s, shall be the least of the following:

(ENGLISH UNITS)	(SI UNITS)
$l_s = 1.5b_f$	$l_s = 1.5\,b_f$
$= 102t_f\,/\,(F_{ye})^{1/2}$	$= 268\,t_f\,/\,(F_{ye})^{1/2}$
$= 2/3d$	$= 2/3d$
$= l_p + (l_b - l_p)/10$	$= l_p + (l_b - l_p)/10$

where:

b_f = Beam flange width (inches or mm).

d = Beam depth (inches or mm).

F_{ye} = Expected yield strength of steel beams (ksi or MPa)

= $R_y F_y$

l_s = Beam slot length (inches or mm).

t_f = Beam flange thickness (inches or mm).

l_b = One-half clear span length of beam (inches or mm).

l_p = Length of shear plate (inches or mm).

R_y = 1.5 for ASTM A 36; 1.3 for ASTM A 572 Grade 42; 1.1 for all other grades.

Slot widths are $^1/_8$ inch (3.2 mm) along shear plate and ¼ inch (6.4 mm) for remainder of length.

2.3.2 Web Plastic Hinge Location:

The web plastic hinge length, l_{hinge}, in the beam or girder shall 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 or mm).

Z_b = Plastic section modulus of beam (in.^{3} or mm^3).

Z_f = Plastic section modulus of beam flanges (in.^{3} or mm^3).

The beam flange hinge location coincides with the end of the shear plate.

2.3.3 Moment Capacities: Plastic moment capacity at the plastic hinge location is calculated as follows:

$$M_p = Z_b F_{ye}$$

where:

M_p = Plastic Moment Capacity (lb.-in. or N-m).

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

$$h_p = T - 2 \qquad SI: h_p = T - 51$$

where:

h_p = Shear plate height (inches or mm).

T = Distance between web toes of fillets at top and bottom of web (inches or mm).

The shear plate thickness, t_p, is determined as follows:

$$t_p = (6/h_p^2)\,[Z_b l_p \,/\, (l_b - l_p)]$$

where:

t_p = Shear plate thickness (inches or mm).

2.3.5 Shear Plate Weld Design: The shear plate shall be welded to the beam web with a C-shaped fillet weld pattern. The weld shall be designed to resist M_{weld}, V_{weld}, and the resulting eccentricity, e. These values are determined as follows:

M_{weld} = Moment resisted by shear plate (lbf-in or N-mm)

= $1.1[t_p/(t_w + t_p)]Z_{web}F_{ye}$

V_{weld} = Shear resisted by shear plate (lbf or N)

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

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

= M_{weld}/V_{weld}

where:

D = Shear at column face due to dead loads (kips or kN).

L = Shear at column face due to live loads (kips or kN).

t_w = Thickness of beam web (inches or mm).

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

= $1.1M_p/(l_b\text{-}l_p)\text{+}(1.2D + 0.5L)/2$

Z_{web} = Plastic section modulus of beam web (in^3 or m^3).

= $t_w T^2/4$

2.3.6 Strength Demands at the Critical Sections: Strength demands at the critical sections shall be determined by calculation, applying statics that consider the effects of the moment and shear at the plastic hinge location.

2.4 Fabrication:

All components of the Slotted Web™ connection system shall be fabricated by an approved fabricator. The approved fabricator shall comply with Section 1701.7 of the UBC or be currently approved by ICBO ES. When approved by the design engineer and the building official, field fabrication may be permitted, with special inspections complying with Section 1701 of the UBC. Slot termination holes are made and slots are then flame-cut from web end to slot holes. Slot tolerances are as follows:

$^1/_8$-inch-wide (3.2 mm) slot: + $^1/_{16}$ inch, - 0 inch (+2 mm, -0 mm) to end of shear plate.

$^1/_4$-inch-wide (6.4 mm) slot: + $^1/_8$ inch - 0 inch (+3.2 mm, -0 mm) from end of shear plate to the slot determination hole.

Automatic flame-cutting equipment may be utilized in cutting both the slot and the termination hole. No grinding or surface finishing of the slot or hole is required.

No beam penetrations are permitted within the distance measured from beam end to slot termination hole location plus beam depth. No beam flange attachments, welded or bolted, are permitted within the distance measured from beam end to slot hole location plus one-half the beam width.

Full-penetration welds are used to connect beam flanges and webs to each column flange. The shear plate is connected to the beam web with fillet welds. Shear plates are attached to the beam typically with $^3/_4$-inch-diameter (20 mm) or $^7/_8$-inch-diameter (22 mm) ASTM A 325 or A 490 high-strength erection bolts at approximately 6 inches (150 mm) on center that are fully tensioned in the field. For example, two bolts are used for beam depths up to 24 inches (610 mm), with an additional bolt for nominal beam depths over 24 inches (610 mm). Partial penetration welds are used to connect shear plates to each column flange.

A Welding Procedure Specification (WPS) in conformance with Section 4 of AWS D1.1 shall 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 shall be in conformance with Section 3 of AWS D1.1. A WPS that is not in conformance with Section 3 of AWS D1.1 shall be based on a documented Procedure Qualification Record (PQR) in accordance with Section 4 of AWS D1.1. A previous PQR, done by the contractor's fabrication/erection subcontractor to qualify a WPS that complies with the provisions of Section 4 of AWS D1.1 and the construction documents, will satisfy this requirement on a given project application.

2.5 Erection:

Erection shall be in conformance with Section 7 of the Code of Standard Practice for Steel Buildings and Bridges, of the American Institute of Steel Construction (AISC), effective June 10, 1992, and shall be consistent with the requirements noted in Chapter 22, Divisions II or III, IV, and V, of the UBC for Load and Resistance Factor Design.

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

1. Shop drilling $1^1/_{16}$-inch (27 mm) holes in the web of the beam. Automatic flame-cutting may be utilized to produce the slot termination hole.
2. Partially flame-cutting slots starting at "Y" distance (as shown on the connection detail drawing) from the end of the beam and continue flame cut to the slot termination hole. No grinding of the flame-cut slots is required. Slot at k-line shall be tangent to top of slot termination hole.
3. Installing beam and fully tensioning all bolts in the field to the preinstalled shear plate.
4. Preheating and post-heating shall conform to the requirements of AWS D1.1.
5. Complete penetration welds at top and bottom flanges.

6. Welding beam web to column flange full height of shear plate.
7. Inspecting welds by ultrasonic testing per Section 2.6 of this report.
8. Welding shear plate to beam web and visually inspecting fillet welds.
9. Completing the slots by flame cutting from "Y" to the end of the beam web.
10. Providing fillet welds with size equal to $^1/_4$ the beam flange thickness, not less than $^1/_4$ inch (6 mm) nor more than $^3/_8$ inch (10 mm), and inspecting the welds.
11. If building erection can be accomplished by initially cutting the beam slot to full length, staged cutting of slot may be eliminated and cutting completed in shop only fabrication.

2.6 Special Inspection and Ultrasonic (Nondestructive) Testing:

Special inspection for the fabrication and erection of Slotted Web™ connection systems shall be in accordance with Section 1701 of the UBC. Special inspection shall verify compliance of steel with specifications, steel identification, qualification of welders, use of appropriate welding materials, storage conditions for welding materials, welded joint preparations, conformance of welding procedures with applicable AWS requirements and fabrication tolerances. As an alternate to special inspections as required by UBC Section 1701, fabrication may be completed on the premises of a fabricator registered and approved by the building official to do such work as outlined in Section 1701.7 of the UBC or a current ICBO ES evaluation report. Where necessary, project site fabrication may be completed following the special inspection requirements of the UBC.

In Seismic Zones 3 and 4, visual inspection and ultrasonic nondestructive testing (NDT) in accordance with Section 1703 of the UBC shall be performed. A visual inspection and testing program shall be established by the design engineer and shall include the following items, at a minimum:

This special inspection program shall include the following:

1. Visual inspection of the fillet welds. All complete-penetration groove welds contained in joints and splices shall be tested 100 percent by ultrasonic testing.

EXCEPTIONS:

a. For SSDA "Slotted Web"™ moment frame connections, the nondestructive testing rate for an individual welder or welding operator may be reduced to 25 percent, provided the reject rate is demonstrated to be 5 percent or less of the welds, tested for the welder or welding operator. A sampling of at least 40 completed welds for a job shall be made for such reduction evaluation. Reject rate is defined as the number of welds containing rejectable defects divided by the number of welds completed. For evaluating the reject rate of continuous welds over 3 feet (914) in length where the effective throat thickness is 1 inch (25 mm) or less, each 12-inch increment (305 mm) or fraction thereof shall be considered as one weld. For evaluating the reject rate on continuous welds over 3 feet (914 mm) in length where the effective throat thickness is greater than 1 inch (25 mm), each 6 inches (152 mm) or length of fraction thereof shall be considered one weld.

b. For complete penetration groove welds on materials less than $^5/_{16}$ inch (7.9 mm) thick, nondestructive testing is not required; for this welding, continuous special inspection is required.

2. All partial penetration groove welds used in column splices are tested by ultrasonic testing.
3. Base metal thicker than $1^1/_2$ inches (38 mm), when subjected to through-thickness weld shrinkage strains, is ultrasonically inspected for discontinuities prior to and after joint completion. Plates shall be evaluated per ASTM A 435; rolled steel shapes shall be evaluated per ASTM A 898.
4. Delayed hydrogen cracking of completed welds: 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:

   a. Flux core weld filler metal of low hydrogen percentage (less than 18 g per 100 g of in-place filler metal) from moisture-resistant sealed containers must be utilized.

   b. After flux core weld filler metal has been withdrawn from shipping wrapper and is 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, to eliminate flux absorbed moisture contamination.

   c. 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.

   d. If temperature is below 50°F (10°C), 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.

   e. Ultrasonic testing for weld volumetric flaws may proceed as soon as possible after welds are completed. A check for delayed cracking of welds from hydrogen embrittlement shall be made after the building structure is substantially loaded. Only the lower flanges need be tested, and at the rate of 25% of all SSDA "Slotted Web"™ connection welds. After concrete floors on the completed frame are in place, the ultrasonic recheck for delayed cracking may commence.

   f. Run-off 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.

   g. 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. This bottom flange weld back-up bar removal and repair is very important because only in this flange is the extreme fiber-in-bending at the root of the weld

5. Ultrasonic testing of other complete and partial penetration welds.
6. Qualification of NDT personnel is to be per ASNT SNT-TC-1A-01 for level I, II, III personnel.

7. Bolting: Bolts in the web of SSDA "Slotted Web"™ connections are erection bolts and do not require special inspection. Bolts must be installed in accordance with Section J3 of the AISC Load & Resistance Factor Design Specifications for Structural Steel Buildings (December 1, 1993), to support erection loads. Surfaces of connections must be clean and burr-free to allow for fit-up.

2.7 Identification:

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

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

3.0 EVIDENCE SUBMITTED

Data in accordance with the ICBO ES Acceptance Criteria for Qualification of Steel Moment Frame Connection Systems (AC129), dated July 1997.

4.0 FINDINGS

That the Slotted Web™ Beam-to-Column Steel Moment Frame Connection described in this report complies with the 1997 *Uniform Building Code*™ (UBC), subject to the following conditions:

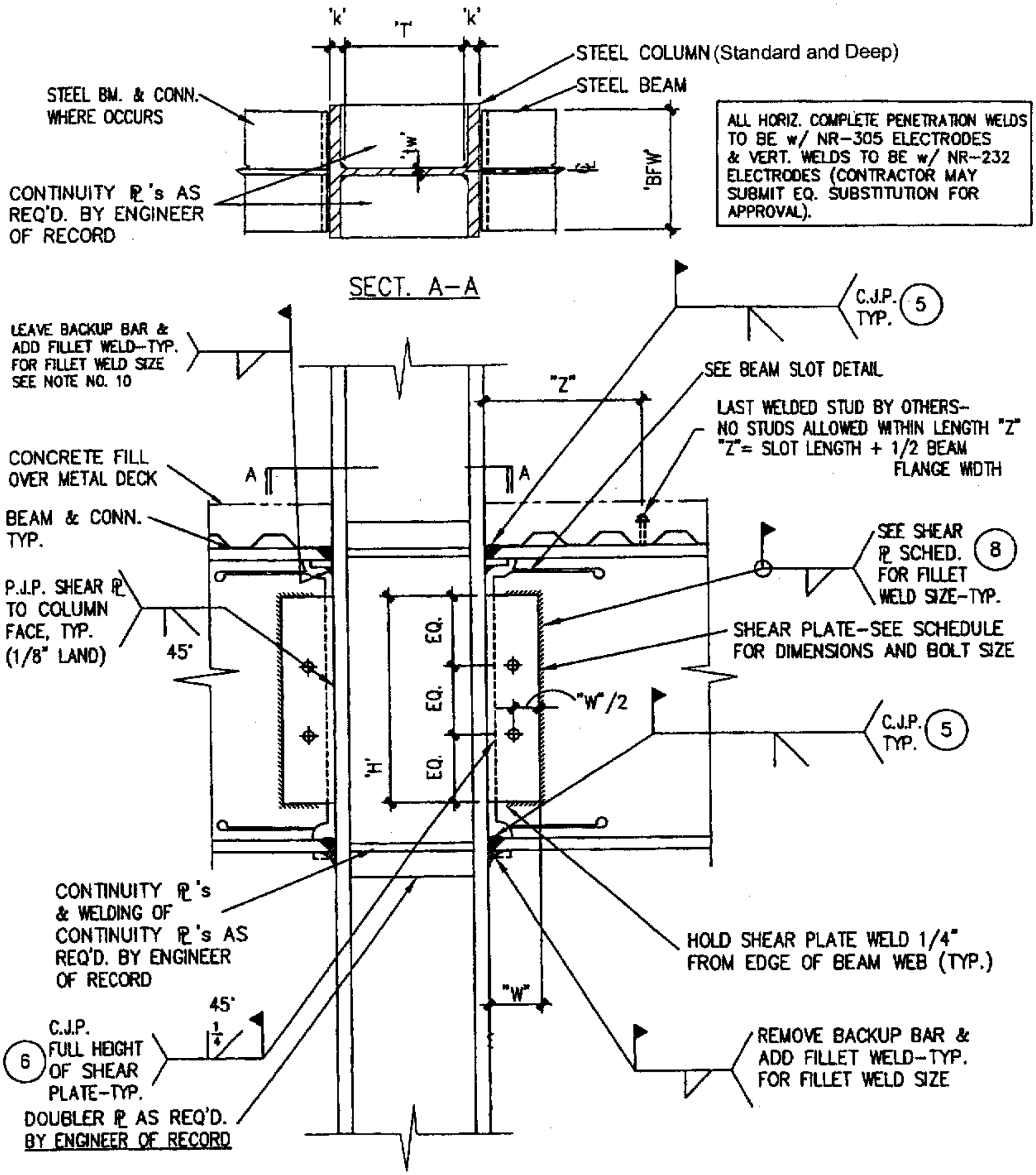
4.1 Design is in accordance with this report and the UBC.

4.2 Fabrication complies with Section 2.4 of this report and is done by a fabricator approved by the building official, as set forth in Section 1701.7 of the UBC; or fabrication is under special inspection as set forth in Section 1701.5 of the UBC.

4.3 Erection is in accordance with the UBC, the manufacturer's instructions, and Section 2.5 of this report and AISC erection standards.

4.4 Special inspection and nondestructive testing in accordance with Section 2.6 of this report are provided.

This report is subject to re-examination in one year.



SSDA BEAM SLOT CONNECTION

FIGURE 1

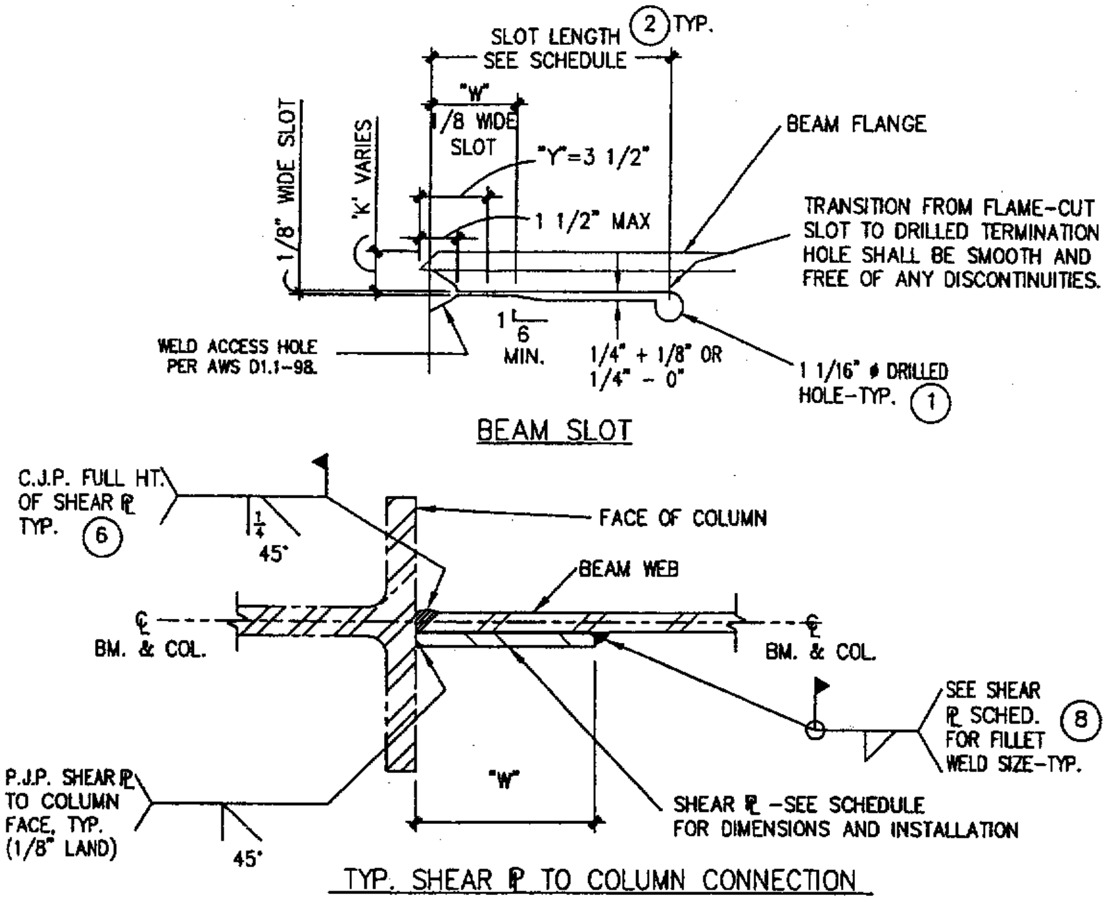


FIGURE 1—(Continued)