IF YOU'VE EVER ASKED YOURSELF "WHY?" about something related to structural steel design or construction, *Modern Steel Construction*'s monthly Steel Interchange column is for you! Send your questions or comments to solutions@aisc.org.

Beveled Transitions

A ½-in. steel plate transitions from 10 in. wide to 18 in. wide over a length of 6 in. (a transition slope of 1.5 to 1 along each side of the plate). Is this transition acceptable?

The requirements for transitions depend on the application. Is the piece subject to static load with R = 3 used in the design, or is it a high-seismic or cyclic loading case? Also, are the plates welded or cut?

If it is statically loaded with R = 3 used in the design: the requirements recently changed and no transition is now necessary for welded plates. The change was made in the 2008 version of AWS D1.1 where the provision requiring a 2.5 to 1 transition has been eliminated. If it were a cut profile, I would say the corner needed to have some radius but that would not be required of a welded transition. If there is a chance of uneven stress distribution, you may wish to require some sloped transition but it is no longer a code requirement.

If it is a case of high-seismic or cyclic loading: the 2.5 to 1 transition is still an AWS D1.1 requirement (see Clause 2.16.1.2 and Figure 2.12). When the provision was removed from the static requirements in AWS D1.1, it was added to the AWS D1.8 *Seismic Supplement* (see AWS D1.8 Clause 4.2). Also, if the load is cyclic and you are designing using the fatigue categories in Appendix 3, there are details that require the use of a sloped transition or defined radius, e.g., cases 5.2 and 6.1.

Erin Criste and Tom Schlafly

AESS Expectations

We had architecturally exposed structural steel on a previous project, and a difference in paint appearance resulted at parts where welding or grinding occurred—the steel "looked" different. Are there guidelines or standards as to the finishes where grinding and/or welding has occurred, so that we can avoid the problems of differing expectations in the future?

The surface variations from rolled to ground to welded will always appear different through paint, and especially when the paint or coating is not a flat sheen. AESS is specified in Section 10 of the AISC *Code of Standard Practice*, and this contains basic requirements as defaults. Special finish requirements beyond those in Section 10 are to be specified in the contract documents.

There are other documents that are available relevant to this from the Rocky Mountain Steel Construction Association. That information was published as an AESS supplement to the May 2003 issue of *Modern Steel Construction* and includes a sample AESS specification. It also discusses some of the aspects of specifying AESS that may add significant costs to the steel package. For a free online version of that supplement, go to **www.modernsteel.com/backissues**. Perhaps that will help you resolve the issue.

Charles J. Carter, S.E., P.E., Ph.D.

Reinforcing an Existing Beam

I'm designing for additional load in an existing structure and plan to reinforce the floor framing. Do I have to account for the existing stresses in the unreinforced shapes as I design the reinforced cross-section?

If our methods were truly elastic in nature, the loading history might be important. However, beam design involves inelasticity, and the only impact that the loading history has is on the deflection. That is, your deflection of the reinforced member starts at the position of the unreinforced member and progresses from there under the new loading. Strength is unaffected.

For the reinforcement of these members, you might want to look at two papers from AISC *Engineering Journal*: "Reinforcing Steel Members and the Effects of Welding," by Raymond H.R. Tide (4th Quarter 1990) and "Field Welding to Existing Steel Structures," by David T. Ricker (1st Quarter 1988). They are available to AISC members as free downloads at **www.aisc.org/epubs**. Non-members pay a nominal purchase price.

Charles J. Carter, S.E., P.E., Ph.D.

Windows Compatibility

I just upgraded to Windows 7 and am trying to use the CD companion that came with my 13th Edition AISC *Manual*. It doesn't seem to work. How can I make it run?

Backward compatibility seems to be more and more of a problem these days. Microsoft offers various compatibility modes to run programs and resources that were created before Windows 7. They have a page outlining the process at:

http://windows.microsoft.com/en-us/windows7/Make-older-programs-run-in-this-version-of-Windows

Alternatively, we have put the content of the CD companion online at **www.aisc.org/epubs**. You should be able to use the various features that way as it does not require the interface program that ran the CD companion. It is a free download for AISC members; the price for non-members is $70.

Martin Anderson

Shear Stud Spacing in Composite Design

For the design of composite flexural members, Section I3.2d(6) in the 2005 AISC *Specification* limits the maximum center-to-center spacing of shear connectors to eight times the total slab thickness or 36 in. Does "total slab thickness" refer to the total thickness of slab and deck (for composite steel deck) or the concrete thickness above the deck?

For a composite metal deck with concrete topping, the total slab thickness is the thickness of the concrete topping plus the thickness of the composite metal deck. For example, if you have 3 in. of concrete topping on a 3-in. composite metal deck, the total slab thickness is 6 in.

Heath Mitchell, P.E.

Proprietary Connection?

I have been told by a provider of one proprietary seismic moment connection that they have a patent on the WUF-W connection. Is this connection subject to a patent?

No, the WUF-W connection is and always has been in the public domain. This abbreviation covers a detail with a welded unreinforced flange (the WUF part) with a welded web (the W part) as illustrated below with the special seismic weld access hole also illustrated below. Like its close cousin with a bolted web (the WUF-B), it was developed by the FEMA-funded SAC Joint Venture and published in FEMA 350 and related documents. Both of these details have been incorporated into AISC documents, including the AISC *Seismic Provisions* (AISC 341) and the AISC *Seismic Design Manual*. Additionally, the WUF-W detail recently was added to the list of details that are prequalified for use in Special Moment Frames (SMF) and Intermediate Moment Frames (IMF) in AISC 358 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* according to AISC 341 Appendix P.

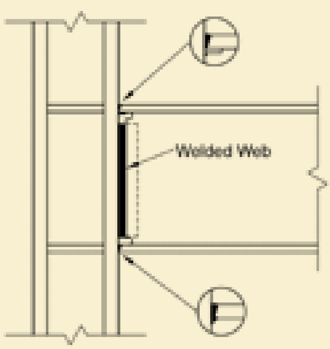
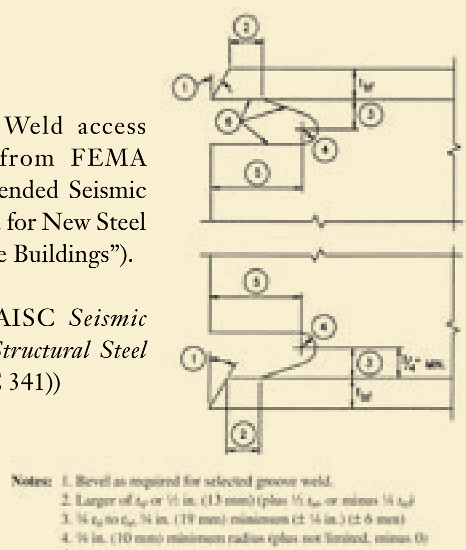


Figure C-I-11.1 Schematic illustration of strong-axis moment connection: directly welded. See Kaufmann, Xue, Lu and Fisher (1996).

(From 2005 *Commentary on the Seismic Provisions for Structural Steel Buildings*)



Tolerances shall not accumulate to the extent that the angle of the access hole cut to the flange surface exceeds 25°.

Figure 11-1. Weld access hole detail (from FEMA 350, "Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings").

(From 2005 AISC *Seismic Provisions for Structural Steel Buildings* (AISC 341))

In a letter dated March 8, 2010, AISC contacted the proprietary connection vendor, stating our position, asking for clarification of their claims, and requesting that they cease and desist from making threats and claims that we consider false and unsubstantiated. We have received no response.

Charles J. Carter, S.E., P.E., Ph.D.

Flexural Strength at Bolt Holes

When using Equation F13-1 to determine the flexural strength, φM_n, of a W-shape beam with holes in the flange, is S_x simply the value in the property tables from Part 1 of the AISC *Manual* or do I have to calculate it considering the holes that are present in the tension flange?

The gross elastic section modulus, S_x, from Part 1 of the AISC *Manual* is used in Equation F13-1 without consideration of bolt hole reductions. The ratio in the rest of that equation accounts for the reduction due to holes in the form of a critical stress.

Brad Davis, S.E., Ph.D.

Axial Loads in Shear Connection

I would like to have clarifications from AISC regarding the use of double-angle connections and single-plate connections. Is it true that a double-angle connection provides little resistance to axial loads? If so, should I use a single-plate connection in this case?

In general, both double angles and shear tabs can transfer significant axial loads. However, double-angle connections using the "Welds B" detail shown on page 10-46 of the 13th Edition AISC *Manual* are not well suited to transferring axial loads. There is no standard procedure for calculating the axial strength of these welds, since the axial load will subject the vertical weld to shear, tension and torsion tending to open the root of the weld. Where the connection is to be welded to the support, the single-plate connection is a better option than the double-angle connection when axial loads must be transferred.

Larry S. Muir, P.E.

The complete collection of Steel Interchange questions and answers is available online. Find questions and answers related to just about any topic by using our full-text search capability. Visit Steel Interchange online at **www.modernsteel.com**.

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Steel Interchange is a forum to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Opinions and suggestions are welcome on any subject covered in this magazine.

The opinions expressed in Steel Interchange do not necessarily represent an official position of the American Institute of Steel Construction and have not been reviewed. It is recognized that the design of structures is within the scope and expertise of a competent licensed structural engineer, architect or other licensed professional for the application of principles to a particular structure.

If you have a question or problem that your fellow readers might help you solve, please forward it to us. At the same time, feel free to respond to any of the questions that you have read here. Contact Steel Interchange via AISC's Steel Solutions Center:

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