If you've ever asked yourself "Why?" about something related to structural steel design or construction, *Modern Steel*'s monthly Steel Interchange is for you! Send your questions or comments to **solutions@aisc.org**.

steel interchange

Note: Except where specifically noted, any mention of AISC documents applies to both the 2010 and 2016 editions. All AISC documents referenced can be found at www.aisc.org/publications.

Local Buckling of Round HSS

Section F8 of the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360) addresses the flexural strength of round hollow structural sections (HSS). In Equations F8-2 and F8-4, the ratio, *D/t*, is not squared. Should it be?

Also, these equations can result in nominal flexural strengths higher than formula F8-1. How can the nominal strength be greater than the plastic flexural strength of the section?

The answer to your first question is no. Although the local buckling strength of flat elements is dependent on $(b/t)^2$, Equations F8-2 and F8-4 for the local buckling of round HSS are correctly based on a linear variation in D/t.

As for your second question, the nominal strength cannot exceed the flexural strength. Section F8 states: "The nominal flexural strength, M_n, shall be the lower value obtained according to the limit states of yielding (plastic moment) and local buckling." Because the shape factor, Z/S, varies with D/t, the Z/S ratios for the HSS shapes listed in Part 1 of the AISC *Steel Construction Manual* vary from 1.29 to 1.40. Equation F8-2 was developed using $Z/S = 1.3$, which is the approximate value (depending on E and F_y) at the transition point between compact and noncompact behavior (when $D/t = 0.07E/F_y$).

Bo Dowswell, PE, PhD

Bolted Connections in the Seismic Force Resisting System

I am an engineer performing connection design for a fabricator. We are having an argument in our office about whether the bolted connections at drag struts must be designed as slip-critical. Can you provide clarification?

Yes. Clarification is needed on several points.

Bolted connections required to meet the AISC *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341) can generally be designed as bearing joints as indicated in Section D2.2(a). There is only one condition where the *Seismic Provisions* require connections to be designed as slip critical, and that is for vertical brace connections using oversized holes. This obviously does not apply to drag struts.

The other issue is that you are asking if a connection needs to be designed as slip critical, when I think it would be more appropriate to ask whether or not the connection falls within the seismic force resisting system (SFRS). Since you are working for the fabricator and are not the engineer of record (EOR), you should not be deciding that some particular member is a drag strut, then whether it needs to be considered part of the SFRS system and then whether the connection needs to be designed as slip critical. Only the EOR knows what part of the structural system has been considered in the design to provide the required resistance to the seismic forces. Section A4.1 of the *Seismic Provisions* requires "identification of members and connections that are part of the SFRS" in the structural design drawings and specifications. A member is part of the SFRS if the EOR says it is. This determination by the EOR requires engineering judgment and intimate knowledge of the structure and its design. If the connection is identified as being part of the SFRS in the contract documents, then the connection must satisfy Section D2.2(d), which indicates that the bolted connections need to be detailed and fabricated as slip critical but can be designed as bearing as indicated in Section D2.2(a). If it is not clear whether the connection is part of the SFRS, then you will need to seek guidance from the EOR.

Carlo Lini, PE

Which Edition of the *Manual*?

How long can engineers continue to use the 14th Edition of the *Manual* now that the 15th Edition is available?

The *Manual* is never referenced in building codes. There is no requirement to use any edition of the *Manual*. Which edition of the *Manual* to use is up to the engineer to decide based on the requirements for their project. The *Specification* is referenced in the building codes, but it is unlikely that any jurisdictions have adopted a building code that references the 2016 *Specification* yet.

The 14th Edition *Manual* can be viewed as a tool to make using the 2010 *Specification* easier in practice. The 15th Edition *Manual* is an updated reference that reflects the contents of and changes in the 2016 *Specification*.

Keith Grubb, SE, PE

*Bonus: Back in April 2012, Keith wrote a SteelWise article describing the AISC Manual Resource Page. This page can now be accessed at **www.aisc.org/publications/steel-construction-manual-resources**. The Manual Resources for the 15th Edition* Manual *are expected to be updated and accessible by the end of 2017.*

*Keith's original article, which is still relevant half a decade later, can be found in the Archives section at **www.modernsteel.com**.*

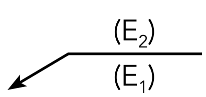
PJP Groove Weld Symbols

I have several questions about partial joint penetration (PJP) groove welds:

1. **What is the proper callout for a PJP groove on the contract drawings?**
2. **Must the effective throat be shown in parentheses?**
3. **As the engineer, would I ever specify a value for *S*, the groove depth?**
4. **Should the weld symbols for PJP groove welds look different on engineering drawings and shop drawings?**

I have provided answers to each of your questions, below.

1. Clause 2.3.5.3 of AWS D1.1 requires the contract documents to specify the required size. It provides the following figure:



2. Yes. The effective throat must be in parentheses.
3. The short answer is: Generally, no.

   Now for a longer answer: AWS A2.4: *Standard Symbols for Welding, Brazing, and Nondestructive Examination* allows a PJP groove weld to be designated with: the groove depth alone, the effective throat alone or both the groove depth and the effective throat. Since the effective throat can depend on both the groove depth and the process, specifying only the depth of preparation does not directly govern the effective throat, which is likely your primary concern as an engineer working on structures within the scope of the AISC *Specification*. Over-specifying a weld in the contract documents can lead to higher fabrication costs since it can preclude—unless a change to the contract is requested and approved—the use of welds that may be more economical while still providing the required strength.

   Neither AWS D1.1 nor the *Specification* prohibit the engineer from providing the weld groove depth, but doing so is not required and is generally not necessary.
4. Typically, yes, they will look different. The figure above from AWS D1.1 provides the proper symbol for design documents (engineer drawings). Generally, the welding symbols on approval documents (shop drawings) should look like the symbols in Table 8-2 of the *Manual*. As discussed above, AWS A2.4 provides several options.

One final point: Your above questions use the terms *weld symbol* and *callout*. AWS D.1.1 uses only the term *welding symbol*. This is consistent with clause 4.1 of AWS A2.4, which makes a distinction between the terms *weld symbol* and *welding symbol*.

Larry S. Muir, PE

Welds that are Too Big

I have specified a ½-in. fillet weld between a column and a base plate. The fabricator has provided a ⅞-in. fillet weld. Though there are no visually apparent detrimental effects from the larger weld, Section J2.2b of the *Specification* indicates that the maximum fillet weld size cannot be "greater than the thickness of the material minus 1⁄16 in." Must the ⅞-in. fillet weld be removed and the parts re-welded with a ½-in. fillet weld?

No. The AISC *Specification* does not place a maximum fillet weld size on a T-joint. It is generally recommended that welds be sized based on demand. The Commentary to Section J2.2b indicates that that the t – 1⁄16 in. requirement is imposed to prevent the upper corner of the welded plate from being melted away, and thus not providing the full required weld throat dimension. Figure C-J2.1 illustrates this condition and illustrates why the requirement does not apply to a column-to-base plate weld.

There can be detrimental effects caused by an oversized weld such as warping of welded parts and of course the economic impact of placing a larger weld. However, for most structures falling within the scope of the *Specification*, providing a larger weld is generally not detrimental.

Jonathan Tavarez

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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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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