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

Through-bolting HSS

Can through bolts be used to transfer tension at a HSS column? Will the bolts require pretensioning, or can they be installed snug-tight?

Transferring tension with through bolts is not prohibited. However, neither the *Manual* nor the *Specification* provide guidance, so you must design such conditions based on your own engineering judgment and knowledge. Example 3.2 in AISC Design Guide 24: *Hollow Structural Section Connections* (a free download for members at www.aisc.org/dg) provides a similar example; it addresses threaded studs but may also be useful in evaluating your condition.

Through bolts can be neither pretensioned nor snug-tightened. Trying to produce the required pretension in the bolts will crush the walls of the HSS, something that seems to be more common than I would have thought until I began to see pictures of crushed HSS sent to the AISC Steel Solutions Center. Even the snug-tightened condition requires the plies to be brought into firm contact and this cannot be done for the condition you described. You'll have to specify the installation you want in the contract documents.

Long story short, a different approach might be better.

Larry S. Muir, PE

Web Openings

I am designing a lightly loaded steel beam with web penetrations exceeding the limits provided in AISC Design Guide 2: *Design of Steel and Composite Beams with Web Openings* (www.aisc.org/dg). The beam is part of a moment frame. Although I understand that the procedures provided in the design guide are not applicable, if these procedures are used to evaluate the condition, then the shear and flexural strength of the beam are significantly greater than the required loads. Is there no way to allow the larger opening?

Let me start off by saying that Design Guide 2 is simply a guide and not a mandatory document. As such, there may be times when you, as the engineer of record, choose to exercise your own judgment when interpreting the information presented in the guide—or you may choose to use a different method entirely for analysis of your condition. If your beam is not highly stressed and you do not believe the "larger" opening will adversely affect the beam performance, then the design may be perfectly adequate.

For instance, let's assume that your condition meets the recommended a_o/h_o limit but does not meet the recommended limit for p_o. The opening parameter, p_o, presented in design guide equation 3-24 is provided as a conservative means to ensure you will not have issues with web buckling localized around the opening or with the member shear strength. There is some discussion regarding the origin and intent of this parameter in Section 5.7, on page 48 of the design guide, which I suggest you review if you haven't already. The parameters in design guide equation 3-24 are all based on physical characteristics of the beam and do not consider the actual stress in the member. However, we can recognize that the likelihood of web buckling occurring in a member is greater when the stresses are greater. If your beam is lightly loaded or your opening occurs in an area of low stress, then it is reasonable to assume you could exceed the limit of p_o, or any of the other proportioning limits and not create a buckling condition. What is low stress and by how much can you exceed the recommended limit are matters of judgment that you'll need to assess for yourself.

As another example, the minimum tee dimensions indicated in Section 3.7.b.1, which were developed as "practical" guidelines due to a lack of test data for shallow tees, do not need to be strictly adhered to as long as you are exercising your engineering judgment. This is briefly explained in Section 5.7.b of the design guide (see page 49). The general intent was to limit the maximum opening size to less than 0.7*d*, leaving 0.3*d* intact. This was split to provide 0.15*d* top and bottom, which is where the 0.15*d* comes from. Again, as the author indicates these are "feel good" limits, and you have some leeway in how strictly you want to adhere to them.

Each condition needs to be evaluated on a case-by-case basis and resolved based on your personal engineering judgment relative to the anticipated loads.

Susan Burmeister, PE

Ungrouted Base Plates

I work in an industry that uses un-grouted baseplates for steel structures—electric substations, for example. The guidance provided in AISC Design Guide 1: *Base Plate and Anchor Rod Design* (www.aisc.org/dg) is directed toward the building industry. Although it is a well-written document, it does not address the situation where the base plate is ungrouted. Generally speaking, ungrouted base plates experience higher weak-axis bending stress as compared to a grouted base plate.

It seems there is much confusion regarding the location of the critical, theoretical bend line relative to the anchors as well as which sections of the current AISC *Code* apply. Are you aware of any published guidance to address this issue?

According to the scope, defined in Section A1, the 2010 AISC *Specification* (a free download at **www.aisc.org/2010spec**) applies to buildings and building-like structures. Therefore, substation structures are not addressed, and there are no plans to address them in the future. AISC Design Guide 1 is focused on typical building and building-like details, but AISC Design Guide 10: *Erection Bracing of Low-Rise Structural Steel Frames* (**www.aisc.org/dg**) has extensive information that you may find useful for these connections. The design recommendations therein are intended to address erection design, but the same principles can be used for permanently non-grouted base plates supported by leveling nuts. The following publications also contain design requirements and recommendations for non-grouted base plate connections:

- AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*
- ASCE *Substation Structure Design Guide*
- ASCE *Design of Steel Transmission Pole Structures*, ASCE/SEI 48

Base plate strength is typically limited by the flexural strength. The flexural strength of rectangular bars is addressed in *Specification* Section F11. The yielding limit state according to Equation F11-1 is applicable to plates bent about their weak-axis.

Bo Dowswell, PE, PhD

Shear Lag in Compression Members

Shear lag is addressed in Chapter D of the *Specification*, which addresses the design of a member for tension. It is usually not considered in Chapter E, which addresses the design of a member for compression. I understand that compression strength is generally governed by either flexural buckling or torsional flexural buckling; and also that stress levels are far below 0.9F_y; hence shear lag will generally not need to be considered. However, should shear lag be considered for short compression members, connected only at the web, with high compression forces?

The effect of uneven stress distribution should be considered but can often be neglected. In steel design, shear lag is almost always associated with the rupture of tension members.

Generally, shear lag is used to describe non-uniform stress conditions caused by localized load-transfer deformations. Such non-uniform stress can occur in elements subjected to both tension and compression. For example, the effective slab width for composite beams is often less than the spacing between beams due to the effect of shear lag at the compression flange.

In extreme cases, it is conceivable that the strength of a compression member could be affected by shear lag. However, I'm not aware of any research indicating that shear lag should be considered in the design of compression members. In practice, the connection detail typically restrains buckling and provides a ductile condition where the stresses can redistribute adequately without failure. Any deformation caused by compression yielding due to shear lag is likely to be negligible.

Hopefully, this will provide enough information for you to use your own judgment to determine what is appropriate for your situation.

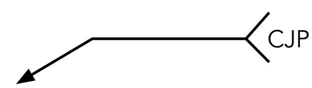
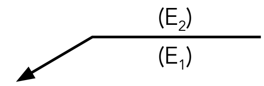
Bo Dowswell, PE, PhD

Flare-bevel Groove Welds

It is common to see flare groove welds shown in the contract documents without the throat provided in the weld symbol. I am under the impression that the welder must build up the weld at least flush and that the throats shown in Table J2.2 of the *Specification* can be assumed. Is this correct, or must the throat always be provided in the weld symbol?

Table J2.2 shows the effective throat of a flare groove weld that is sized to fill flush to the surface. Not all flare groove welds need do so, however.

Section 2.3.5.3 of AWS D1.1 addresses this and states the following:

> The contract documents shall show CJP or PJP groove weld requirements. Contract documents do not need to show groove type or groove dimensions. The welding symbol without dimensions and with "CJP" in the tail designates a CJP weld as follows:
>
> 
>
>
> The welding symbol without dimension and without CJP in the tail designates a weld that will develop the adjacent base metal strength in tension and shear. A welding symbol for a PJP groove weld shall show dimensions enclosed in parentheses below "(E_1)" and/or above "(E_2)" the reference line to indicate the groove weld sizes on the arrow and other sides of the weld joint, respectively, as shown below:
>
> 
>

Based on this requirement, the effective throat should be provided when calling out a flare-bevel groove weld.

Carlo Lini, PE

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**.

Larry Muir is director of technical assistance and Carlo Lini is staff engineer—technical assistance, both with AISC. Bo Dowswell and Susan Burmeister are consultants to AISC.

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:

866.ASK.AISC • **solutions@aisc.org**

Steel SolutionsCenter