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

Web Openings

AISC Design Guide 2: *Steel and Composite Beams with Web Openings* limits the opening parameter, $p_o = a_o/h_o + 6h_o/d$, to a maximum value of 5.6 for steel sections and 6.0 for composite sections.

Our openings conform to the opening parameter limit for the composite section. However, we are using unshored construction and do not satisfy the limit for steel sections.

This limit does not consider the magnitude of the load in the web, so it must be conservative. In our case, the precomposite load is only about 40% of total load.

Is it possible to exceed the opening parameter and still have an acceptable design?

Yes. AISC design guides (available at **www.aisc.org/dg**) provide guidance, not requirements. The guidance is intended to be useful to practicing engineers during typical designs. In order to provide simple and practical guidance, the procedures are sometimes simplified and conservative, as you note. Other approaches are possible. The references in the design guides often provide more in-depth discussions of the issues and can be helpful when addressing unusual conditions.

In this case, Section 5.7.2 of Design Guide 2 provides the following additional information regarding web buckling:

> "The criteria to prevent web buckling are based on the work of Redwood and Uenoya (1979) in which they developed conservative criteria based on the opening size and shape and the slenderness of the web of the member…
>
> Their recommendations are adopted in whole for steel members and relaxed slightly for composite sections to account for the portion of the shear carried by the concrete slab, V_c, The higher limit on the opening parameter, p_o, of 6.0 for composite sections versus 5.6 for steel sections coincides with successful tests (Donahey and Darwin 1988). Failure in composite sections is normally governed by failure of the concrete slab, and adequate strength has been obtained even when local buckling has been observed (Clawson and Darwin 1980, Clawson and Darwin 1982, Donahey and Darwin 1986). As discussed in section 5.6 (after Eq. 5-20), the limits on also serve to ensure that the design equations provide conservative predictions for member shear strength, even if web buckling is not a factor.
>
> …The guidelines limiting the maximum values of V_m can be quite conservative for sections with web width-thickness ratios below the maximum limits. Redwood and Uenoya (1979) provide guidance for members which lie outside the limits of this section."

In addition, a reference to Lucas and Darwin 1990 in the design guide summarizes the results of a number of physical tests. At least a couple of these had opening parameters in excess of the limit provided in the design guide and still resulted in test-to-predicted strengths in excess of one.

Carlo Lini, PE

Strength Reductions for Members with Holes in the Tension Flange

Section E3.6g.5 of the 2010 *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341-10) allows bolted column splices in special moment frames. It states: "Bolted column splices shall have a required flexural strength that is at least equal to $R_y F_y Z_x / \alpha_s$ of the smaller column, where Z_x is the plastic section modulus about the x-axis." However, it seems that any holes placed in the section will reduce its strength below the expected strength. Increasing the size of the column or reinforcing it with plates is not helpful since Z_x will be increased, resulting in an even higher demand. How can a bolted splice be used in a special moment frame?

Your question contains an incorrect assumption. It has been shown through physical tests that within certain limits, a member with holes can still develop its gross flexural strength. Section F13.1 of the *Specification for Structural Steel Buildings* (ANSI/AISC 360, available at **www.aisc.org/specifications**) addresses "Strength Reductions for Members with Holes in the Tension Flange." In this section, when $F_u A_{fn} \geq Y_t F_y A_{fg}$ the limit state of tensile rupture does not apply. For capacity-based design, this relationship would have to be adjusted to account for R_y and R_t. For ASTM A992, both R_y and R_t equal 1.1, so the relationship remains unchanged.

AISC's *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (ANSI/AISC 358-10 with ANSI/AISC 358s1-11, available at **www.aisc.org/seismic**) also addresses bolted flange plate connections, which will force yielding in the beam even though there are holes in the beam flanges.

There are several factors that might help explain this observed behavior. The tensile strength is greater than the yield strength; this offsets the presence of the holes to some degree. The holes on the compression side have little impact on the strength, and the maximum compression strength, though commonly assumed to be equal to the yield strength, is likely greater than we assume.

Larry S. Muir, PE

Pretensioned vs. Snug-Tight Installation

Many of the fabricators we work with regularly use twist-off-type tension-control (TC) bolts regardless of the connection type. Based on this, would it make sense to specify all of our connections as pretensioned, since we are presumably getting pretensioned connections anyway?

No. We are not surprised that some fabricators prefer TC bolts. TC bolts can be used in slip-critical, snug-tight and pretensioned connections. The bolts also have other potential advantages that have made them attractive to fabricators and erectors. The 2014 RCSC *Specification for Structural joints Using High-Strength Bolts* (available at **www.aisc.org/specifications**) does not limit the amount of pretension in a bolt. Therefore, a TC bolt can be tensioned up to the point where the spline is severed, even in a connection specified as snug-tight and designed as a bearing connection. However, simply severing the spline does not ensure that the joint has been properly pretensioned.

The installation of pretensioned joints involves the following considerations beyond those for a snug-tight joint:

- **Pre-installation verification.** Section 8.2 of the RCSC *Specification* states: "Pre-installation testing shall be performed for each fastener assembly lot prior to the use of that assembly lot in the work. The testing shall be done at the start of the work."

  This is an extra step that must be performed during the bolt installation for pretenstioned connections but is not required for snug-tight conditions. Though pre-installation verification is a relatively straightforward process, we do occasionally hear of issues. At the very least, it involves having a properly calibrated tension calibrator (though not all fabricators, erectors and inspectors have one on hand), scheduling the testing and ordering bolts in sufficient quantities to accommodate both the installation and the required testing.
- **Bolt storage and installation conditions.** Section 2.2 of the RCSC *Specification* addresses the storage of fastener components. These requirements apply equally to snug-tight and pretensioned installations. However, the condition of the bolt and the lubricant is more of a concern for pretensioned joints. TC bolts can be particularly sensitive to the condition of the lubricant. If the bolts are not properly stored or the final tensioning is delayed, then the bolts may need to be cleaned and relubricated. For heavy hex head bolts (Grade A325 and A490) this can be done by the user in the field. TC bolts "shall not be relubricated, except by the manufacturer."
- **Installation procedures.** You cannot simply put a TC bolt in a hole and engage the wrench until the spline breaks and expect to have a properly pretensioned joint. First, the bolts must be installed in accordance with the requirements in Section 8.1 of the RCSC *Specification*, which lists the installation requirements for snug-tightened joints. For large, heavy joints, you may actually end up breaking bolts or the splice before you bring the plies into firm contact, and the RCSC *Specification* addresses this, stating: "If a splined end is severed during this operation, the fastener assembly shall be removed and replaced." Once firm contact is achieved, the installation must progress "systematically from the most rigid part of the joint in a manner that will minimize relaxation of previously pretensioned bolts."
- **Inspection.** There are inspection tasks required for pretensioned connections that are not required for snug-tight connections. Section N5.6.(a) of the AISC *Specification* states: "For snug-tight joints, pre-installation verification testing as specified in Table N5.6-1 and monitoring of the installation procedures as specified in Table N5.6-2 are not applicable.
- **Arbitration.** Arbitration is addressed in Section 10 of the RCSC *Specification* and ideally, will rarely be required. Allowing snug-tight installation eliminates the possibility that it will be required.

Each of the above items involves logistical considerations and present potential impacts to the cost and schedule of the project. Some of these items are less of a concern in the shop than they are in field. However, many bolted joints are installed in the field. Therefore, the preferences of the erector must also be considered.

Larry S. Muir, PE, and 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 senior staff engineer, both with AISC.