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: Unless specifically noted, all AISC publications mentioned in the questions and/or answers reference the most recent edition and can be found at **www.aisc.org/specifications**.*

Shear Lag and Eccentricity

Does the shear lag factor, *U*, presented in Chapter D of the *Specification for Structural Steel Buildings* (ANSI/AISC 360) account for bending in the member when only a portion of the section is attached?

No. The Commentary states: "Shear lag is a concept used to account for uneven stress distribution in connected members where some but not all of their elements (flange, web, leg, etc.) are connected… The effect of connection eccentricity is a function of connection and member stiffness and may sometimes need to be considered in the design of the tension connection or member."

Shear lag and eccentric loading are different considerations. Shear lag can occur in both concentrically loaded and eccentrically loaded conditions. Shear lag is addressed in Chapter D. An eccentric tensile load may cause bending in the member, and a User Note in Chapter D points to Chapter H for "members subject to combined axial tension and flexure." The combined effects of bending and tension are addressed in Section H1.2.

The Commentary to Section D3 discusses the interaction of connection and member stiffness relative to the consideration of flexure due to eccentricity.

Larry S. Muir, PE

Weld Access Holes in Prequalified Moment Connections

Why is it not necessary to use access holes when welding beams to bolted extended end-plate moment connections that are prequalified in *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (ANSI/AISC 358), but weld access holes are required in welded unreinforced flange-welded web (WUF-W) moment connections and reduced beam section (RBS) moment connections?

Many of these requirements have been developed through testing. Regarding the end plates, the Commentary for Section 6.7 in AISC 358 states: "Cyclic testing has shown that use of weld access holes can cause premature fracture of the beam flange at extended end-plate moment connections (Meng and Murray, 1997). Short to long weld access holes were investigated with similar results. Therefore, weld access holes are not permitted for extended end-plate moment connections."

The RBS and WUF-W are different in that the flanges are directly connected to the column, and thus directly transfer moment through CJP welds. The weld access holes provide increased relief from concentrated weld shrinkage strains, avoid close juncture of welds in orthogonal directions, reduce stress concentrations and provide adequate clearance for high-quality welding and ease of inspection. The weld access hole requirements are also different for RBS and WUF-W.

The geometry of the weld access holes in RBS need only conform to the requirements of the *Specification*. The weld access holes in WUF-W must conform to the requirements of AWS D1.8 Clause 6.11.1.2. Weld access hole quality requirements shall conform to the requirements of AWS D1.8.

The Commentary to Section 5.5 discusses the RBS weld access hole and states: "Test specimens have employed a range of weld access-hole geometries, and results suggest that connection performance is not highly sensitive to the weld access-hole geometry. Consequently, prequalified RBS connections do not require specific access-hole geometry."

The Commentary for Section 8.5 discusses the weld access hole for the WUF-W system and states: "This special access hole was developed in research on the WUF-W moment connection (Ricles et al., 2000, 2002) and is intended to reduce stress concentrations introduced by the presence of the weld access hole."

See the January 2010 SteelWise "Prequalified Seismic Moment Connections (Revisited)" (**www.modernsteel.com**) for additional discussion of the requirements of prequalified seismic connections.

Jonathan Tavarez

Effect of Pretension on Tension Strength of Bolts

The footnote to *Specification* Table J3.1 indicates that the required minimum bolt pretension is "equal to 0.70 times the minimum tensile strength of bolts as specified in ASTM F3125/F3125M for Grade A325 and Grade A490 bolts with UNC threads, rounded off to nearest kip." However, the *Specification* applies a factor of safety, Ω, of two to the nominal strength, allowing only 0.50 times the minimum tensile strength of bolts to be applied to the bolt in service. If the bolt is already stressed to 0.70 times its minimum tensile strength, has the bolt not already "failed" (exceeded its design strength) before any additional load has been applied?

No. The bolt has not failed due to pretension. We recently received a similar question. The pretension does not reduce the available tensile strength of the bolt.

People often feel that a bolt pretensioned to 70% of its tensile strength has already violated the tension limit state for the bolt and

Larry Muir is director of technical assistance, **Jonathan Tavarez** is a staff engineer in the Steel Solutions Center and **Carlo Lini** is a senior staff engineer, all with AISC.

STEEL SOLUTIONS CENTER

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.

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**

any additional applied tension cannot be allowed. This is an incorrect interpretation and is addressed in a couple of places. The *RCSC Specification for Structural Joints Using High-Strength Bolts* addresses this in the Commentary for Section 5.1: "If pretensioned bolts are used in a joint that loads the bolts in tension, the question arises as to whether the pretension and the applied tension are additive. Because the compressed parts are being unloaded during the application of the external tensile force, the increase in bolt tension is minimal until the parts separate (Kulak et al., 1987; pp. 263-266). Thus, there will be little increase in bolt force above the pretension load under service loads. After the parts separate, the bolt acts as a tension member, as expected."

AISC Design Guide 17: *High Strength Bolts—A Primer for Structural Engineers* (available at **www.aisc.org/dg**) discusses this in more detail in Sections 4.2 and 6.2. You will find that the pretension in the bolt places the material within the bolt grip into compression. As an external tension load is applied to the bolt, this reduces the clamping force and slightly increases the tension in the bolt. This increase is negligible, as the Design Guide states: "Both theory and tests [6] show that the increase in bolt pretension up to the load level at which the connected parts separate is in the order of only 5 to 10%. This increase is small enough that it is neglected in practice. Thus, the assumption is that under service loads that apply tension to the connected parts a pretensioned bolt will not have any significant increase in internal load." The discussion in Chapter 6 includes more detail and free body diagrams to help illustrate this issue.

Carlo Lini, PE

Specifying Bolts

Given that ASTM F3125 has replaced the standards for A325, A325M, A490, A490M, F1852 and F2280, what is the proper way to specify high-strength bolts on contract drawings?

In order to minimize confusion, the "old" designations (A325, A490, etc.) in F3125 are used as grades. In order to ease the transition, the common designations (A325, A490, etc.) may be used when the more formal designation is not necessary. For example, Section J3.1 in the 2016 *Specification* refers to ASTM F3125/F3125M Grade A325, but Table J3.1 simply refers to A325 bolts. The Version 15.0 *AISC Design Examples* document (available at **www.aisc.org/manualresources**) simply refers to Group A or Group B bolts in figures. Section 3.1 of the *Code of Standard Practice for Steel Buildings and Bridges* (ANSI/AISC 303) states: "The structural design documents shall clearly show or note the work that is to be performed and shall give the following information with sufficient dimensions to accurately convey the quantity and complexity of the structural steel to be fabricated."

The way in which you specify the bolts should reflect your intent. If your goal is to minimize the cost, then there are advantages to not over-specifying materials or means and methods. If you specifically want the contractor to use twist-off-type tension-control (TC) bolts, then you need to specify ASTM F3125/F3125M Grade F1852 (or F2280). If you specifically do not want the contractor to use TC bolts, then you need to specify ASTM F3125/F3125M Grade A325 (or A490). If you want to give the bidding contractors the freedom to provide bolts and installation methods that provide the least cost to the project, then you might simply specify "Group A bolts as defined in Section J3.1 of the AISC *Specification*."

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**.