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.

Slip-Critical Bolts?

TC bolts were used in a bolted splice for W8s. The beams have corrugated deck on top of them that will receive 6 to 12 in. of concrete, and the assembly forms a ramp for foot traffic. The engineer wants us to remove the TC bolts and replace them with slip-critical bolts. I am under the impression that slip-critical is a connection and not a type of bolt. Is this a valid point from the inspector? Do we have to replace the bolts, or is the use of TC bolts acceptable for this condition?

You are correct that slip-critical (SC) refers to a type of connection, while TC refers to a type of bolt. Furthermore, TC bolts often are used in SC connections.

TC bolts are a product that is central to one of the four methods permitted by the RCSC *Specification* to achieve pretension in a bolt, as is required in SC connections. As long as the requirements in the RCSC *Specification* for this method are followed, TC bolts are acceptable in SC connections.

Amanuel Gebremeskel, P.E.

Charpy V-Notch Requirements

AISC 341-05 Section 6.3 specifies a minimum Charpy V-notch value for structural steel in the SLRS with flanges 1½ in. and thicker. Commentary Section C6.3, however, states that steel with flanges exceeding 2 in. is subject to the same requirement. I assume the Standard is correct and the Commentary is incorrect? Please verify.

There does not appear to be an inconsistency between the *Seismic Provisions* Section 6.3 and the Commentary C6.3. The first part of Section C6.3 (where the 2 in. is mentioned) is in reference to the AISC *Specification* (AISC 360-05), while the following statement (where 1½ in. is mentioned) is referring to the requirements of the AISC *Seismic Provisions* (AISC 341-05).

The Commentary is pointing out the difference between AISC 360 and AISC 341 in this regard.

Kurt Gustafson S.E., P.E

Flare-Bevel Groove Welds

I am detailing an HSS-to-HSS weld and have a flare-bevel groove weld, which is shown in the latest edition of the AISC Manual (page 8-61). The effective weld size (E) is shown as $\frac{5}{8}T_1$. In Table J2.2 of the *Specification*, the effective weld size of a flare-bevel groove weld is given as $\frac{5}{8}R$ for the GMAW process. Which one is correct?

The Effective weld size shown in Table 8-2 (page 8-61) of the 13th edition *Steel Construction Manual* was based on the 2004 AWS D1.1 requirements. This effective size for flare-bevel groove welds has been revised in the 2006 AWS D1.1 and will vary based on the weld process used. The current draft of the 2010 AISC *Specification* includes a revision to update the provisions for flare-bevel groove welds to be consistent with the current AWS D1.1 requirements for these welds. Look for Manual Table 8-2 and the Specification Table J2.2 to both be based on a function of R, rather than the wall thickness in the next editions. For the GMAW process, the proposed draft lists the effective weld size as $\frac{5}{8}R$ for the flare-bevel groove weld.

Kurt Gustafson S.E., P.E

Lessons Learned from the AISC Seminar

In discussions with several engineers, I am hearing it said that the 13th edition is forcing engineers to abandon the ASD method, and to conform to the LRFD method.

Two things that I remember from this year's AISC seminar on the 13th edition are:

1. **The LRFD strength is equal to 1.5 times the ASD strength, and**
2. **Either approach can be used, and the designer just has to remain consistent with the chosen method during the calculation.**

Unfortunately, I am not knowledgeable enough with the 13th edition to convince them that ASD is still permitted. Can you explain this a little more convincingly?

It remains completely viable—and familiar—to use ASD with the 13th edition. You may remember from the seminar you attended that all examples were worked in both ASD and LRFD, and many comparisons showed where provisions were identical or improved in the 13th edition ASD compared to the 9th edition ASD.

The two points that you list form the basis of the "unified" specification. The levels of safety are essentially equivalent regardless of which load combinations in ASCE 7 the designer chooses to use. These were good points to bring away from the seminar.

The 2005 AISC *Specification* was developed in a format that permits the engineer to use either ASD or LRFD for structural steel design. Section B3.3 covers the use of LRFD load combinations and design, while Section B3.4 covers the use of ASD load combinations and design. That is why the AISC lecture series on the 2005 *Specification* and *Manual* bears the title *Design Steel Your Way*.

The R_n used in the Specification is the nominal strength of the member or component, which is the same capacity regardless of whether ASD or LRFD load combinations are used in the analysis. This R_n value is calculated the same way and then the φ (LRFD) or Ω (ASD) factor is applied based on which approach you choose. The factor that is applied must be consistent with the load approach used in the analysis. Thus, if the LRFD load approach were used in the analysis, the R_n must be multiplied by the φ factor for the applicable limit state to determine the Design Strength (LRFD). Similarly, if the ASD load approach were used in the analysis, the R_n must be divided by the Ω factor for the applicable limit state to determine the Allowable Strength (ASD).

Although the 2005 *Specification* is presented in a strength format, the stress format is usually just lurking right there in the equation; use your thumb to cover the section property (A, Z, S, etc.) if you still can't see it. Also, there have been changes that have occurred in the Specification since the development of the 18-year-old 1989 ASD *Specification*. While some of the capacities may be similar, others have changed. There are limit states to consider that may have not been addressed in past specifications. One would need to look at the controlling limit state for the member or component under consideration in order to make a valid comparison.

In the end, though, ASD has not been abandoned.

Kurt Gustafson S.E., P.E

Welding in the K-Area?

Section 3.9.6 of Design Guide 21 discusses the potential effect of welding in the k-area for column doubler and stiffener (continuity) plates. Does this same concern apply to welding of beam stiffener plates in the k-area?

Yes. AISC recommends that you avoid the k-area when attaching stiffeners to beams. This area is avoided by clipping the stiffener plate. Please see Fig. C-J10.7 of the 2005 AISC *Specification* Commentary (a free download at **www.aisc.org/2005spec**) for how to detail this.

Amanuel Gebremeskel, P.E.

Turning the Bolt Head

A contractor told us that applying the pretensioning by turning the head of an ASTM A325 bolt does not produce as good a result as turning the nut. I never heard of that. Is there any merit in his contention?

It likely is most common to turn the nut, rather than the bolt head, during the installation process; however, the RCSC *Specification* explicitly permits the turning of the head (see Section 8.2). The procedures used in construction must be the same as those demonstrated during the pre-installation verification process, as required by the RCSC *Specification*.

Kurt Gustafson S.E., P.E

Block Shear Strength

I need some help understanding the Block Shear Equation (J4-5) in the 2005 *Specification*. For block shear strength:

$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$$

The left side of the equation must be less than or equal to the right side. For ASTM A36 steel, F_u = 58 ksi and F_y = 36 ksi, which means that $0.6F_u$ will always be greater than $0.6F_y$. The net area in shear, A_{nv}, is smaller than the gross area in shear, A_{gv}, but not enough to overcome the difference between $0.6F_u$ and $0.6F_y$.

As an example, consider a ⅜ in. plate with 1½ in. edge distance and 6 rows of ¾-in. bolts in STD holes at 3-in. spacing.

$$\begin{aligned} A_{gv} &= (\tfrac{3}{8}\text{ in.})(5 \times 3\text{ in.} + 1\tfrac{1}{2}\text{ in.}) \\ &= 6.19\text{ in.}^2 \\ A_{nv} &= A_{gv} - \text{hole area deduction} \\ &= 6.19\text{ in.}^2 - (\tfrac{3}{8}\text{ in.})[5.5\text{ holes} \times (\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})] \\ &= 4.39\text{ in.}^2 \end{aligned}$$

From this, $0.6F_uA_{nv}$ = 153 kips, and $0.6F_yA_{gv}$ = 134. That is, this shows $0.6F_uA_{nv} > 0.6F_yA_{gv}$.

So where did I go wrong? When will $0.6F_uA_{nv}$ ever be less than $0.6F_yA_{gv}$?

I believe you went wrong in your preliminary assumption that the net area is smaller than the gross area, but not enough to overcome the difference between $0.6F_u$ and $0.6F_y$.

Remember that the Specification is not written for a specific case, but for a multitude of geometries and material types that may be used in steel construction. Therefore, both parameters of the equation must be checked to find which controls.

Try some other examples and see what happens. One simple extrapolation of your stated problem would be to use ASTM A572 Grade 50 plate instead, substituting F_y = 50 and F_u = 65 into your results as follows:

$$0.6F_uA_{nv} = 153\text{ kips} \times \tfrac{65}{58} = 171\text{ kips}$$
$$0.6F_yA_{gv} = 134\text{ kips} \times \tfrac{50}{36} = 186\text{ kips}$$

In contrast to your example, $0.6F_uA_{nv} < 0.6F_yA_{gv}$.

Kurt Gustafson S.E., P.E

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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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