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.

Reuse of Bolts

We are working on a bolted through plate girder bridge that is currently being dismantled and transported to be used at another location. The bridge was in service for roughly 5-7 years prior to dismantling. Can the bolts that have been removed for dismantling be reused? Would I need to look for certain types of damage before approving the reuse and/or any other items of concern that I should be aware of?

I am assuming that these are ASTM A325 or A490 pretensioned bolted connections.

Section 2.3.3 of the RCSC *Specification* (a free download at www.boltcouncil.org) discusses the reuse of high strength bolts as follows: "Reuse: ASTM A490 bolts and galvanized ASTM A325 bolts shall not be reused. When approved by the Engineer of Record, black ASTM A325 bolts are permitted to be reused."

Generally, ASTM A325 bolts that are not galvanized can be reused if they have only been retightened once or twice, and the nut can fairly easily be reinstalled on the bolt. For background and guidance in making this decision, you may want to look at page 62 of the *Guide to Design Criteria for Bolted and Riveted Joints*, which is also available as a free download from RCSC. There also is a discussion on page 47 of AISC Design Guide No. 17 *High-Strength Bolts: A Primer for Structural Engineers*, which is a free download for AISC members at www.aisc.org/epubs.

Kurt Gustafson, S.E., P.E.

Slip-Critical Connections

When using slip-critical connections, is it common to exclude the threads from the shear plane, or does thread location matter since the design assumption is not based on load transfer by shear/bearing?

Section J3.8 of the AISC *Specification* requires that slip-critical connections must also be checked for the bearing condition. In most cases, the slip strength is less than the shear strength of the bolt and it does not matter whether the threads are excluded or not. In some instances, when the slip-critical connection is designed to the serviceability limit state, and employs standard holes and Class B faying surfaces, the design slip resistance can be slightly higher than the shear strength of the bolt with the threads included. In these instances excluding the threads can provide a slightly higher (about 1%-4%) design capacity.

Larry S. Muir, P.E.

UT of CJP Welds

Can UT be done on a complete-joint-penetration groove weld if the thickness of the steel is less than 5⁄16 in.? If not, can a visual inspection be performed, or can another type of NDT be done?

UT is not well suited to distinguishing defects in the first 5⁄16 in., so that is considered a lower limit. If visual is not considered acceptable, which it should be in most cases, MT is often selected as the next best alternative.

Tom Schlafly

K-Brace for OCBF System

I am designing an ordinary concentrically braced frame using a K-brace. AISC 341 Section 14.3 states that the column for K-type bracing is to be designed for the unbalanced loading. Is that accurate? Designing the column for the forces specified in Section 14.3 seems very high. Is there any alternative, such as designing the column for the amplified seismic load?

While K-type bracing is not strictly prohibited for OCBF (except for OCBF above seismic isolation systems), it is generally not considered desirable in concentrically braced frames, and is prohibited entirely for SCBF. It is considered undesirable to have columns subjected to unbalanced lateral forces from the braces, as these forces may lead to column failure. If one chooses to use K-type bracing, the *Seismic Provisions* requires that the unbalanced force be considered as defined.

Kurt Gustafson, S.E., P.E.

Beam Bracing

I am designing a one-story open-framed (no decking) building and must provide lateral stability bracing of the beams. Working with the 2005 AISC *Specification*, Appendix 6.3 addresses the force required for both nodal and relative bracing in beams. I have a situation where nodal bracing is desired for architectural reasons. I am aware that this bracing force must be delivered to a rigid support at bracing ends. Does the bracing force act in an additive manner? For example, I have four parallel beams restrained from rotation via nodal bracing; does the bracing have to be proportioned to resist four times the force computed from Eq. A-6-7?

Appendix 6 requires that a minimum stiffness and strength be available to brace a beam to prevent lateral-torsional buckling. The implication of this is that if a member that has exhausted its axial strength due to other loads [$P_u/\phi P_n$ or $\Omega P_a/P_n = 1.0$] were to be used to brace a beam, there would be no brace strength remaining to prevent LTB. Therefore, the bracing member does have to be designed with the bracing force added to other loads on the brace. This is in addition to the stiffness requirement.

For your second question, if it is possible for all beams to buckle at the same time, the bracing forces will accumulate from the first beam through the last, and the brace at that point must provide the strength and stiffness required in Appendix 6 for all beams.

Amanuel Gebremeskel, P.E.

Channel Columns

What Section of the AISC *Specification* covers channel columns?

If there are no local slenderness issues, the provisions of Section E3 in the 2005 AISC *Specification* can be used to design the channel as a compression member. If there are local slenderness issues [as per Table B4.1] the requirements in Section E7 also apply.

Amanuel Gebremeskel, P.E.

Bolt Hole Alternatives for Fit-Up

In many instances bolt holes are required to be enlarged at the site because the size of bolt holes does not accommodate allowable deviations in the erection of the structure. Which of the following alternatives do you recommend to address this issue?

1. **Provide oversized holes during fabrication.**
2. **Limit the allowable dimensional deviations during erection.**
3. **Enlarge holes in the field for only joints where the misfit occurs.**

Common practice is usually to supply short-slotted holes for shear connections. This combined with the flexibility of typical shear connections allows some additional fit-up tolerance in the field. The joints can be designed as bearing connections as long as the slots are perpendicular to the direction of the load.

Option 1: It is not uncommon to use oversized holes designed as slip-critical connections for certain applications such as vertical bracing and bolted flange plate moment connections. It is rare to see oversized holes used in beam connections.

Option 2: The AISC *Code of Standard Practice* provides fabrication and erection tolerances that are in accordance with current construction and design practices. Additional limits might come at a high cost or scare bidders away.

Option 3: This option is problematic if the same bolt size is to be used, since the connection would have to be designed as slip critical at the outset. Slip-critical connections usually will have less strength than a bearing-type connection with an equal number of bolts. This will lead to uneconomical connections.

A common approach is to ream the hole to the next larger size and use a bolt with the corresponding larger diameter. Since the connection was originally designed based on the strength of the smaller bolt and the larger bolt will have both greater strength and deformational capacity, mixing the differing bolts does not present a problem.

Usually things fit up pretty well. In my experience misfits are the exception not the rule, so it is best to assume that things will fit as intended, unless your experiences with certain contractors or locations prove otherwise. I have listed some of the common ways these problems are dealt with, but each situation must be approached differently, taking into account cost, schedule, and most of all safety.

Larry S. Muir, P.E.

Use of Overstrength Factor

The AISC *Seismic Design Manual* (Page 3-43) does not use the overstrength factor in the design of the column in that example. However, other references design both columns and beams considering the overstrength factor. Which is correct?

The column check in Design Example 3.8 in the AISC *Seismic Design Manual* reflects a column where the axial load ratio $P_u/\phi P_n \leq 0.4$. Therefore, use of the overstrength factor is not required. This is covered in Section 8.3 of the AISC *Seismic Provisions*. If the axial load ratio is greater than 0.4, the overstrength factor must be considered. This may be the case in the other reference to which you refer.

Kurt Gustafson, S.E., P.E.

Number of Washers

What is the maximum number of washers allowed on a bolt under the nut? What if it is for an anchor rod embedded in concrete for a column base?

The AISC and RCSC *Specifications* address installation requirements for high-strength bolts used in steel-to-steel connections where the steel faying surfaces are being clamped together. Even in such connections, there is nothing in the AISC or RCSC *Specifications* that limits the number of washers under the nut or the head of a bolt. Placing additional washers under the nut is a common practice used to exclude the threads from the shear plane. In steel-to-steel connections, it would be unusual for more than two washers to be required to exclude the threads using material thicknesses usually seen in structural work. The EOR needs to exercise some judgment as to when it is preferable to add washers or change out the bolt.

Anchor rods are used for a different purpose than high-strength bolts, and the evaluation of the restraint details is quite different. The EOR needs to assess the purpose of the anchor rod in the particular application, and the forces that the anchor rods are assumed to resist. Again, the engineer needs to use some judgment as to how the forces are transferred from the column to the foundation, and the effect that adding additional washers may have on that assumed force transfer mechanism. If the anchor rod is being used to resist base shear (which is not recommended by AISC) the evaluation will likely be quite different than if the anchor is only assumed to resist tension.

Kurt Gustafson, S.E., P.E.

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