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.

Nuts and Washers for Anchor Rods

What is the proper material specification for anchor rod nuts and washers?

ASTM F1554 has a table of recommended nuts for use with various grades and diameters of anchor rods. Typically these nut recommendations are nuts that develop the tensile capacity of the rod. In other words, the nut is sized so that the rod will fail in tension before nut or thread failure. I recommend that you obtain a copy of ASTM F1554 and use Section 6.6.1 to select the appropriate nuts. Common structural steel-related ASTM standards are available in *Selected ASTM Standards for Structural Steel Fabrication*, which has just been revised for 2011 and can be purchased at **www.aisc.org/astm**. (See page 17 for additional information.) ASTM standards also are available for purchase at **www.astm.org**.

ASTM F436 washers likely will be too small to use with oversized holes in base plates. Table 14-2 in the 13th Edition AISC *Steel Construction Manual* has recommendations for maximum base plate hole size and minimum washer dimensions such that the washer will completely cover the hole, regardless of anchor rod position. For base plates in compression and where shear is not transferred through the rods, there are cases where ASTM F844 washers will be of sufficient size to meet the table requirements. Otherwise, a structural-grade material (such as ASTM A36 or A572 Grade 50) is selected with a thickness that is sufficient to transfer the required shear and/or uplift.

Heath Mitchell, P.E.

Conflicting Requirements Between Contract Documents

The project specification calls for shop-primed steel, but the drawings say in the notes section to not prime the steel that is concealed. Which directive governs?

Absent of any contract provisions, Section 3.3 of the 2005 AISC *Code of Standard Practice* contains two provisions that relate to your question. I will explain them in reverse order.

The second paragraph details what is done when a discrepancy is discovered between two (or more) parts of the contract documents before work is performed. It requires reporting of the discrepancy by the fabricator or erector so that it can be resolved by the engineer, architect, owner, etc. It is not required that the fabricator or erector perform a review of the contract documents to discover conflicts, as the quality and coordination of the drawings, specification, and similar contract documents is rightly the responsibility of the design team.

The first paragraph covers the case of a discrepancy that is discovered after work has been performed. It establishes that a requirement stated in the design drawings governs over a conflicting statement in the specification.

I'm not sure from your description if this is a case of before or after. However, a clear statement on the drawings not to prime concealed steel is sufficient reason to expect when bidding that priming is not required. If it is now required, a contract price adjustment to pay for the priming may be appropriate.

I'd also like to point out that it is well known—and stated as such in AISC *Specification* Section M3.1 and its Commentary—that steel to be enclosed by building finish need not be primed or painted. With this in mind, perhaps it will be agreeable for your project that there need be no financial consequence to the owner in spite of the conflict in the contract documents.

Charles J. Carter, S.E., P.E., Ph.D.

Paint Under Bolt Heads

I have a field issue where paint is on the outer plies (under the bolt head and under the washer) in new pretensioned joints in an existing structure. The inspector is rejecting the bolts because the paint exists and it is squeezing out under the bolt head and washer. Can the paint remain? Is it a problem that it is squeezing out?

Paint is permitted under bolt heads and washers, but the RCSC *Specification* does have a caution in the Commentary about thick coatings (see the Commentary to Section 8.2). This Commentary specifically says galvanized coatings, but if the paint is squeezing out, it is probably thick enough that the Commentary information in the RCSC *Specification* about thick coatings applies.

The question here is whether the coating is causing a reduction in the pretension below the minimum required. The answer to that question tells you whether the coating can remain or must be removed. You may not have to remove the coatings, but you might have to allow for the loss of pretension or re-pretension, as suggested in that Commentary. Alternatively, you can remove the paint.

Charles J. Carter, S.E., P.E., Ph.D.

Finding an AISC Member Fabricator or Erector

I'm trying to find an AISC member fabricator. Does AISC provide such a list?

Yes. There is a tab at the top of the AISC website that says "Find a Company/Person" or you can use the URL **www.aisc.org/members**.

This is a directory search of AISC member fabricators and erectors. There is a drop-down menu on this page where you can select a fabricator or an erector. Also, you can search by city and/or state for a person or company.

Erin Criste

Plate Bending

A debate is raging in our office. For years, the allowable bending stress in base plates was 0.75F_y. The 13th Edition AISC *Steel Construction Manual* appears to stipulate 0.60F_y for ASD design methodology. Is this an error? If not, can you explain why the change is necessary?

Previously, when checking weak-axis bending the allowable stress was 0.75F_y. However, the check was made using S_y. Currently the allowable stress is 0.6F_y, but the check is made using Z_y. For a rectangular section Z_y/S_y = 1.5. Since 0.75/0.6 = 1.25, the 2005 AISC *Specification* includes a slight gain in strength over the 1989 ASD *Specification*. (The 2010 AISC *Specification* continues this practice.).

In the 1989 ASD, you were essentially using the plastic section modulus for both weak and strong axis bending. For fully braced strong axis bending of a compact member, the allowable stress used to be 0.66F_y instead of 0.6F_y. 0.66/0.6 = 1.1. This approximates the ratio of Z_x/S_x for a wide flange beam.

So in the end there really has not been much change at all, though the calculations look somewhat different.

Larry S. Muir, P.E.

steel interchange

Using Phi and Omega

On page 2-10 in the 13th Edition AISC *Steel Construction Manual* it states, "The general relationship between the safety factor (Ω) and the resistance factor (φ) is Ω = 1.5/φ." Does this relationship also extend to the loading, meaning if factored LRFD loads are provided can an engineer use the LRFD loads divided by 1.5 with ASD resistance factors?

Considering dead and live loading only, if the LL/DL ratio is exactly 3, this is an identical design. For higher ratios it is conservative and for lower ratios you get a lighter design load than using ASD load combinations. Throw in wind load or another load and the permutations possible make it hard to say whether it is conservative. I have heard of a conversion ratio of 1.4 (LL/DL ratio of 1) to add a little conservatism when taking LRFD loads back to ASD design levels. I think this would be appropriate as most common applications have LL/DL ratios less than 3. However, as I already noted, the conservatism is dependent on the actual load types and magnitudes being considered.

Charles J. Carter, S.E., P.E., Ph.D.

Torsion in HSS

How do you determine the Warping Constant (C_w) for HSS?

There is no need for C_w to be defined for HSS. C_w is used for open shapes such as I-shapes. Imagine the case of a simply supported beam subject to torsion. The top flange laterally displaces in one direction while the bottom displaces in the other. If you look at a plan view of these, you will see that the section warps, which means that initially plane sections don't remain plane. The section is more resistant to this bending if the flanges have large lateral moments of inertia and are far removed from the centroid. That is why C_w is related to I_y and h_o. For an HSS, flange bending is only part of the resistance to such warping. The in-plane shear stiffness and strength of the side walls also resists it, and this mechanism is much stiffer than the flange bending strength, so the problem comes back to a pure torsion shear stress type of problem.

Brad Davis, S.E., Ph.D.

Braced Frame Beam Design

On pages 3-47 and 3-48 of the AISC *Seismic Design Manual* a procedure is outlined for determining the axial force in a chevron braced frame beam. On page 3-48, the axial force is calculated as the average of the tension and compression resultants. Could you please explain the justification for this average value? From statics, it seems that it should instead be treated as a straight sum of the two resultants.

At the top of page 3-48 it states, "Assuming that the unbalanced force is shared equally." The intent is that an equal amount of load is dragged in from the left side and the right side. In other words, half of the load is dragged in from each side of the frame. This is a simplifying assumption for the design example and applies to the specific building being evaluated, but depends on the building configuration and frame layout. For example, some frames may drag all of the load in from one side and some braced frame beams may be used to transfer loads to other frames along the same line. These are just a few of the possible load paths that may apply.

Heath Mitchell, P.E.

Galvanized Joint Preparation

RCSC *Specification* Section 3.2.2 (c) requires galvanized surfaces in slip-critical joints to be roughened by hand wire brushing. Is the timing of brushing important?

I am not aware of any requirement on the timing of the hand wire brushing or any research that would support such a requirement. For non-galvanized, bare (not painted) surfaces, the proper faying surface can be maintained for at least a year. It is likely that the galvanized roughened surface will be okay for at least this long.

I personally have had success with having the hand-wire brushing done by the galvanizer. I believe there is more control over the process there than in the field. I did not have a concern about the delay between the roughening and the assembly of the joint.

Larry S. Muir, P.E.

Weld Access Holes

We use CJP groove welds for the column flange to base plate connection in Special Moment Frames. Are we allowed to use weld access holes at column base plate connection where the connection is similar to Extended End-Plate beam-to-column connection?

The prohibition on use of weld access holes is only for the prequalified, extended end-plate beam to end-plate detail specified in AISC 358. The weld access hole is eliminated there because the presence of a weld access hole interrupts the flow of force from the beam flange to the bolts inside the beam flange. The strain pattern that results tends to promote a fracture of the flange at some point in the loading prior to significant yielding. It is acceptable to not use a weld access hole in this case because the peak demand on the flange weld is out at the bolt lines, not at the center of the web.

This is the only detail where we recommend omission of weld access holes. A column base plate usually doesn't have anchor rods inside the flanges like a moment end plate, so I don't see the need to omit weld access holes in that case.

Charles J. Carter, S.E., P.E., Ph.D.

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**.

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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:

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