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.

Web Sidesway Buckling

I noticed that there is a t_w^3 term in the numerator and denominator of AISC *Specification* Equation J10-7 for the web sidesway buckling case where the compression flange is not restrained against rotation. Since these terms cancel out, why are they included in the equation? Is it true that the web sidesway buckling strength is independent of web thickness?

You're right about the t_w^3 cancelling out. AISC *Specification* Equation J10-7 reduces to a function that is dependent on the flange stiffness couple (moment of inertia times *h*). What you have observed is that when the compression flange is braced, the web is able to provide flexural stiffness to prevent lateral deflection of the tension flange (Equation J10-6). However, when the compression flange is not braced, the web is no longer able to provide this restraint and thus the web thickness cancels out of the equation (Equation J10-7).

The formula is in the form it is in for consistency and to keep the calculation simple. This may seem counterintuitive, but note that the reduced formula would have an "*h*" in it. This value is not published in the AISC *Manual*, rather it is incorporated into h/t_w.

Heath Mitchell, P.E.

Weld Strength Calculation

On page 3-28 of the AISC *Seismic Design Manual*, the gusset-to-beam force was calculated assuming only a single line of weld. Is this a conservative assumption? Figure 3-7 on page 3-34 indicates a double-sided weld.

You are correct that the calculations starting on page 3-28 through the consideration of f_{peak}/f_{avg} treat the weld as a single line. However, two welds are assumed in the calculation for *D* (there is a 2 in the denominator).

If, instead, it were treated as two lines of weld, a factor of two would consistently apply to the weld force calculations. The result is that f_{peak}, f_{avg} and, therefore, f_r would each be half the value shown. Thus, the result is the same in the end.

This approach works because the welds are parallel, symmetrically loaded and have the same leg size. The leg size is removed as a factor in the calculation of f_a, f_v, and f_b. It is convenient for calculations because the number of welds assumed doesn't need to be tracked through the calculations; it can be determined at the end based on the calculated demand.

Heath Mitchell, P.E.

Stairs and Handrails

Where can I find information on the design of stairways and handrails?

You may wish to contact the National Association of Architectural Metals Manufacturers (NAAMM), which publishes a stair design manual under the auspices of its AMP subgroup. Much of the information is available electronically including materials on stair design, handrails, and so on, at **www.naam.org**. Select AMP, then Technical Literature.

Note that NAAMM has undertaken to revise and update its stair manual with the aid of the National Ornamental and Miscellaneous Metals Association (NOMMA). That revised stair manual is not yet available, but you may want to keep an eye out for it.

Martin Anderson

Composite Beam Design

AISC 360-05 Section I3-2a seems to imply that if h/t_w is less than or equal to $3.76\sqrt{(E/Fy)}$, then the condition of the stresses under construction loading need not be combined with stresses induced on the post-composite section. The 1989 AISC *Specification* required a check of $M_{DL}/S_{beam}+M_{LL}/S_{eff} \le F_a$. Is it correct that when using AISC 360-05 Section I3-2a, the superposition of construction stresses in the beam and post-composite stresses on the transformed section need not be checked?

The 2005 AISC *Specification* uses a strength-design approach in ASD and LRFD whenever possible and that's the case for bending of composite beams with compact webs. Consider how the stresses develop in an unshored steel beam all the way from the construction phase to the highest strength at ultimate loads. When the construction loads are added, the steel beam will develop bending stresses, most likely in the elastic range although there could be some yielding at the flange tips even at that stage. For the sake of this discussion, assume that the construction load stresses don't cause yielding. The stresses remain mostly constant while the concrete hardens. At that point, if loads are applied, then you're correct that the beam stresses are a superposition of the construction load stresses and the new ones. At some load level, the total stresses in the beam will be high enough to cause yielding. If additional load is applied, yielding will spread and eventually the entire steel beam will be at a stress of F_y and the concrete also will be stressed. Using the strength-design approach, we skip straight to this last step which is the "end of the line."

Consider the beam described above at the ultimate load. Cut a free body diagram of the left half of the beam and compute the moment M_u that would exist at midspan. It would be from all of the loads, regardless of whether they're applied during construction or superimposed. That's why we compare the fully plastic moment at the "end of the line" strength to the moment when all of the loads are applied, without concerning ourselves with what happens at lower load levels.

Don't forget to check the construction load case. Unshored steel beams are usually used to support the wet concrete, deck, beam self weight, and construction live load. The steel beam will resist these loads as a bare steel beam. Also, remember that deflection is cumulative and the construction deflection is the starting point for the deflection of the composite cross section.

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

Anchor Rods

I am trying to find the specific code section that requires a minimum of four anchor rods in a column base plate, but could not find anything in AISC 360. Where can I find this requirement?

The requirement for four anchor rods is an OSHA requirement. It is part of the Federal Register (29CFR Part 1926, Subpart R) and is available on the OSHA website (**www.osha.gov**). The section that contains the four-anchor rod requirement is 1926.755.

Heath Mitchell, P.E.

Combined Loading

I am designing a beam subject to combined strong-axis bending, weak-axis bending, axial compression and torsion. The shear is very small. What design guidance is available on this subject?

AISC *Steel Design Guide No. 9*, *Torsional Analysis of Structural Steel Members*, can be used for the situation that you describe. The design guide is available at **www.aisc.org/epubs** as a free download for AISC members for purchase by non-members. See Section 4.7.1, specifically equation 4.16a. AISC *Specification* section H3.3 also gives some guidance, although it does not go to the same depth as *Design Guide 9*. The general idea is to limit the applied stress to F_y for yielding due to normal stress, $0.6F_y$ for shear yielding or F_{cr} if buckling is the applicable limit state. For example, if your beam is unbraced against lateral-torsional buckling, then you'll need to determine F_{cr} per AISC *Specification* Chapter F. If there are several types of stress, such as axial, strong-axis bending, and warping normal stress, then an interaction equation approach is used.

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

Identifying Old Bolts

I am modifying a steel structure erected in 1968. The typical bolt assemblies include square bolt heads and hex nuts. There are no bolt head markings. Is it reasonable to assume that these bolts are ASTM A307?

Bolts conforming to the ASTM A307 standard have been used in steel structures at least since the mid-1930s and were available with square heads. A307 bolts are still available with square heads. Historically there was no head marking required for these bolts, however the heads of modern A307 bolts now have to bear an identification mark.

AISC 360 Appendix 5 states, "Representative samples of bolts shall be inspected to determine markings and classifications. Where bolts cannot be properly identified visually, representative samples shall be removed and tested to determine tensile strength in accordance with ASTM F606 or ASTM F606M and the bolt classified accordingly. Alternatively, the assumption that the bolts are A307 shall be permitted." Based on this, assuming the unidentified bolts are A307 is allowed.

Larry S. Muir, P.E.

Attaching Fill Plates

The 13th Edition AISC *Steel Construction Manual* Table 4-3 Case 1-C shows fill plates shop bolted to the upper shaft. Can fill plates be shop tack welded in lieu of bolting?

If the fill is no more than ¼ in. thick or there is no design load being transferred through the bolts, the fill has no structural significance and can be tack welded in place. If the bolts are being designed for a given load and the fill is greater than ¼ in. thick, the design must conform to the requirements of AISC *Specification* Section J5 and, depending on the design approach, the shop bolts may be required to develop the fill. Alternatively, the fill can be developed by welding.

Larry S. Muir, P.E.

Continuing Education

Do you have any recommendations on classes or seminars that would be useful for me to update my skills from ASD to LRFD?

AISC provides a number of continuing education opportunities. There are live seminars hosted around the country that can be found on the AISC website at **www.aisc.org/seminars**. One such seminar you may be interested in is *Design Steel Your Way II: Efficient Analysis for Steel Design using the 2005 AISC Specification*. AISC also offers online courses at **www.aisc.org/elearning**. For example, there is a free online presentation titled, *The Steel Specification: A Designer's Perspective*.

Plus, coming up in May 2011 is the North American Steel Construction Conference (**www.aisc.org/nascc**). This year it will be held in Pittsburgh and will offer a number of continuing education opportunities. The advance program with seminar and course descriptions is now available at the web link above. Previews of several NASCC presentations appear elsewhere in this issue of *MSC*.

There are also other sources such as AISC *Engineering Journal* papers and *Modern Steel Construction* articles available at **www.asic.org/ej** and **www.modernsteel.com**, respectively.

Erin Criste

Column Plumbness Requirements

What is the maximum plumbness tolerance for an interior column?

The tolerance on the plumbness of a column is given in Section 7.13.1.1 of the AISC *Code of Standard Practice*, which is available as a free download at **www.aisc.org/freepubs.** This states, "For an individual column shipping piece, the angular variation of the working line from a plumb line shall be equal to or less than 1⁄500 of the distance between working points, subject to the following additional limitations…" It goes on to list other limits for columns that are adjacent to elevators and exterior columns.

Larry S. Muir, P.E.

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.

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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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