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.

ASD and LRFD

What are the major differences between the ASD and the LRFD design methods in the current AISC specification?

With the 2005 AISC *Specification*, the difference between ASD and LRFD amounts to little more than which set of ASCE 7 load combinations you choose to use in the design. It basically boils down to which side of the design equation you place the safety factor on: in the combined form of load factors on the load side and resistance factors on the strength side (LRFD), or in the form of safety factors only on the strength side (ASD).

The *Specification* stipulates nominal strengths for the various limit states that are applicable for a particular design situation. This nominal strength is the same whether the ASD or LRFD approach is used for the design. This nominal strength is then either divided by a safety factor Ω if ASD load combinations are used, or multiplied by a resistance factor ϕ if the LRFD load combinations are used. You just want to remember to stay consistent throughout the analysis and design process, and not to mix approaches.

The name of the AISC seminar developed as an introduction to the new *Specification* reflects this design approach option: "Design Steel Your Way with the 2005 AISC *Specification*." In this way you can use the load approach with which you feel more comfortable, with the end results reflecting a similar level of safety or reliability. If you'd like to attend this seminar, view the listing of upcoming dates at **www.aisc.org/seminars**.

—Kurt Gustafson, S.E., P.E.

WT in Flexure

Section F9 of the AISC *Specification* covers tees loaded in the plane of symmetry. Equations F9-4 and F9-5 cover the stem being in tension or compression. I'm designing a member that is loaded in its axis of symmetry (the y-axis for a tee or double angle), which produces tension or compression in the stem. However, the formulas use I_y, the moment of inertia of the weak axis. Using the weak-axis moment of inertia for bending about the strong axis is counterintuitive to me. Why is the weak-axis moment of inertia used in these equations?

It may seem counterintuitive, until you realize that you are checking lateral-torsional buckling (LTB), which is about ensuring that the required level of lateral stability is present to enable the needed strength in the vertical direction. If the beam were fully braced, yielding and local buckling would suffice for the strength checks. With an unbraced length, however, LTB tends to cause the tee to translate laterally and rotate torsionally (hence the name). The LTB check is made to ensure that the cross section can resist the lateral torsional effects, and is based in part on the weak-axis moment of inertia I_y.

—Sergio Zoruba, Ph.D., P.E.

ASTM F1554 Grade 36 vs. ASTM A36

What's the difference between ASTM A36 anchor rods and ASTM F1554 Grade 36 anchor rods? They both have the same tensile and yield properties.

The difference is that ASTM F1554 is a standard developed specifically for anchor rods. ASTM A36 is a material specification that applies to mild carbon structural steel in general. The ASTM F1554 standard is a complete product specification for anchor rods, and includes three grades of rod, one of which is Grade 36, which is essentially A36 (or A307) rod material. Each of the rod strengths in F1554 requires a color coding on the end to facilitate easy identification in the field. The standard also stipulates requirements for threading and other dimensional characteristics. Because of these unique features pertaining to anchor rods, it is the preferred standard for anchor rods.

The AISC *Specification* permits the use of other materials for anchor rods as defined in Section A3 of the AISC *Specification*. Note that other specifications provide very little of the anchor-rod-specific value found in ASTM F1554.

—Kurt Gustafson, S.E., P.E.

Flare Bevel Groove Welds in Tension and Compression

I'm designing a flare bevel groove weld between a rectangular HSS16×8×½ and an A36 plate. The load is parallel to the weld axis (tension and/or compression). Table J2.5 states "Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts." So, how do I design the weld?

We assume that you are referring to a face of an axially loaded tube being welded to an end connection plate.

The stipulation in Table J2.5 is specific to a joint similar to the longitudinal seams in a built-up box column made from four plates. When such a column is loaded under a concentric axial load in compression or tension, the welds between the plates will not be exposed to shear. The welds will be exposed to compression or tension, the same as the plates. Hence from a strength perspective, the welds are not resisting the forces between two adjacent plates.

However, for your case, the HSS axial load is transferred in shear to the connection plate (there is a net difference in force). Hence the weld must be designed for shear strength (and the base metal also checked for strength by way of Section J4).

—Sergio Zoruba, Ph.D., P.E.

What is the Difference Between r_t and r_{ts}?

In the 9th edition AISC ASD manual, the value of r_t was tabulated, and one can refine it by using the St. Venant equation as given in Commentary. The 13th edition AISC manual

tabulates values of r_{ts}. Are both values same? If not does one need to calculate the r_t?

The r_{ts} published in the 13th edition AISC manual is the effective radius of gyration used in the determination of L_r for the lateral-torsional buckling limit state for W-shapes and similar shapes (doubly symmetric compact I-shaped members and channels). This parameter is used in Equation F2-6 of the 2005 AISC *Specification*. This is similar to that used in the 1989 AISC ASD *Specification*, which is still permitted as a conservative approximation by Equation F2.2. See the user note in Section F2.2 of the 2005 AISC Specification for further discussion.

—Kurt Gustafson, S.E., P.E.

End Panel

My questions relate to panel zone shear and tension field action. The 2005 AISC *Specification*, Section G3 states that tension field action cannot be used in the "end panel." There is no definition given for end panel.

Is the end panel not intended to be a support zone from the bearing stiffener, if run up from the center of the support to a cantilevered or fixed end, where the deformations are constrained? Who originally contrived the end panel notion, and what reference could we use to determine the intent?

The end panel is defined in the Glossary of the 2005 AISC *Specification* (at the beginning of the *Specification*) as the "web panel with an adjacent panel on one side only." Hence it is the last panel located at each end of a plate-girder containing transverse stiffeners.

Per Section G3.1, tension-field action cannot be considered in such panels. This provision is not new and a good explanation can be found in the Commentary to Section G3.1 of the 2005 AISC Specification. As stated in the Commentary:

"The key point is the development of tension field action in the web of plate girders is the ability of the stiffeners to support the compression from the two panels on either side of the stiffener. In the case of end panels there is a panel only on one side."

—Sergio Zoruba, Ph.D., P.E.

Technical Bulletin No. 3

In a recent project, a reference to "Technical Bulletin No. 3" was made when specifying steel material. Can you please advise where this bulletin can be found?

You are likely looking at an old project, or someone used old references to develop their project specifications. Technical Bulletin No. 3 was an interim advisory issued in 1997 to address a new steel material being produced at the time, ASTM A572 Gr. 50 with Special Requirements. This material was the forerunner to the current ASTM A992 material prior to the introduction of the ASTM A992 standard (initially adopted in 1998). The bulletin is outdated and rather meaningless, since you can get everything it required simply by specifying ASTM A992 for W-shapes.

—Kurt Gustafson, S.E., P.E.

Historic Bolt and Rivet Strengths

I am working on design modifications to an existing power plant built between 1967 and 1970. I can reference back the steel member designations to the AISC *Steel Construction Manual*, fifth edition. Based on field observations, the building was constructed using standard beam connections. The general connection notes required the fabricator to design the bearing or friction connections (appears to be fabricator's choice) for the forces indicated on the drawings or for the full member strength if no forces were given. The fabricator was to use rivets or high-strength bolts.

I have an old copy of the 5th edition AISC manual. I noticed connection strength tables for standard beam connections, but they are for rivets. Based on my field observations, the connections use ¾-in.-diameter bolts. What were the shear and tension properties of high-strength bolts at that time?

If the structure was constructed during the 1967 to 1970 era, it is likely that you should be looking in the sixth edition AISC manual rather than the fifth edition. The sixth edition AISC manual was based on the 1963 AISC specification, which followed the 1961 major reorganization of earlier specifications. Many aspects of steel design and construction that are commonplace today first appeared at this time, including high-strength bolts. The sixth edition of the manual was published in 1964 and included design values for various types of rivets and bolts. The values for bolts were given for both bearing-type and friction-type joints.

You can also find this information in AISC's *Design Guide 15: Rehabilitation and Retrofit*, which is a free download to AISC members at **www.aisc.org/epubs**.

—Kurt Gustafson, S.E., P.E.

Kurt Gustafson is the director of technical assistance, and Sergio Zoruba is a senior engineer in AISC's Steel Solutions Center. Charlie Carter is AISC's chief structural engineer, and Lou Geschwindner is AISC's vice president of engineering and research.

Steel Solutions Center
One East Wacker Dr., Suite 700
Chicago, IL 60601
tel: 866.ASK.AISC • fax: 312.670.9032
solutions@aisc.org