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.

Silicon Steel

I am analyzing a structure that was built in the late 1920s. The structure consists of steel space frame arches. The existing drawings note the steel typically as structural grade steel, except for the chords of the main arches, which are noted to be silicon steel. What is silicon steel and why would it be singled out for use in the chords of a truss? Is it recommended to test coupons of steel from this era or was steel fairly standardized by then where I could just use typical specification properties?

ASTM A94 steel, also historically known as silicon steel, was one of the first high-strength steels. Silicon contributes to the strength and hardness, but also reduces weldability. Silicon steel typically had a yield strength of 45 ksi, a tensile strength of 80-95 ksi and contained very little carbon. It was used in steel bridges and incorporated into the lower portions of built-up columns in buildings back in the 1910s and 1920s.

Material testing may be required. Appendix 5 in AISC 360-10, *Specification for Structural Steel Buildings*, provides information on the evaluation of existing structures, and provides provisions you might use to evaluate your structure. The document is available as free download at www.aisc.org/freepubs.

Erin Criste

PJP Groove Weld Strength

AISC *Specification* Table J2.5, Available Strength of Welded Joints, lists different PJP groove weld strengths in compression for "connections of members designed to bear other than columns" and "connections not finished-to-bear." Please explain what is meant by that and why the "connections not finished-to-bear" have a higher strength.

AISC *Specification* Table J2.5 covers three types of joints for PJP groove welds that transfer compression forces perpendicular to the weld axis: those in joints designed to bear and those in joints not designed to bear.

The first two types listed in the table are for joints designed to bear and refer to Section J1.4. Joints designed to bear are designed to transfer forces through steel-to-steel bearing. In other words, the PJP groove weld is not necessary to transfer compression forces. In the case of column splices or bases (J1.4a) the welds are there simply to hold the parts in place and compression in the weld does not need to be considered. In the case of joints in compression members other than columns (J1.4b), there are minimum lateral force or tensile requirements on the joint, thus the typical PJP groove weld strength is given equal to $0.6F_{exx}$.

The final case is for joints that are not designed to bear. In other words, the weld is designed to transfer the compression load. In this case, the compression limit states in PJP groove welds justify the higher strength of $0.9F_{exx}$.

For a more detailed discussion of this, please refer to the Commentary to Section J2.4.

Thomas J. Schlafly

Coatings for Faying Surfaces

I have not been able to find information on which manufacturers make an appropriate paint for either a Class A or a Class B faying surface for slip-critical bolted connections. Where can I find this information?

You may wish to check NEPCOAT, which is the North East Protective Coating Committee, an organization of state DOTs in the northeast United States. They have developed a list of qualified protective coatings for bridges, and their list includes slip coefficient values for a number of paints. The website for NEPCOAT is www.maine.gov/mdot/nepcoat/.

You may also want to contact the companies that produced the paint systems found in the NEPCOAT list. There may be products meeting your requirements that are not on their list.

Erin Criste

Seismic End-Plate Width

I am designing a Four-Bolt Stiffened (4ES) Extended End-Plate Moment Connection in a Special Moment Frame. I have been designing according to AISC 358 Chapter 6. Table 6.1 has a limitation for the width of the end-plate of 10¾ in. maximum to 10¾ in. minimum. Am I only allowed to use a 10¾ in. wide end-plate?

This was the case when AISC 358 was first introduced. However, the prequalification limits have been modified in "Supplement No. 1 to ANSI/AISC 358-05 Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications." The minimum width for this configuration has been changed to 7 in. The supplement can be downloaded for free from www.aisc.org/epubs.

Larry S. Muir, P.E.

OMF Connection Design

I am trying to determine the design moment for an FR beam-to-column moment connection in an ordinary moment frame. AISC 341-05 Section 11.2a specifies the required flexural strength as the lesser of $1.1R_yM_p$ or the maximum moment that can be developed by the system. How do I determine the maximum moment that can be developed by the system?

The Commentary to AISC 341-05 Section 11.2a provides guidance for determining the maximum force that can be developed by the system. It states, "Factors that may limit the maximum moment that can be developed in the beam include the following:

(1) The strength of the columns;
(2) The strength of the foundations to resist uplift;
(3) The limiting earthquake force determined using $R = 1$."

Larry S. Muir, P.E.

steel interchange

Round HSS Biaxial Bending

The 3rd Edition AISC LRFD *Manual* contained the *Design Specification for Steel Hollow Structural Sections*. This HSS *Specification* had an equation indicating that, for biaxial flexure of round HSS, the required flexural strength, M_u, can be calculated as the square root of ($M_{ux}^2 + M_{uy}^2$). Since the 13th Edition AISC *Steel Construction Manual* does not contain the HSS Specification, is this condition now governed by equation H1-1b and is a linear combination required?

Formerly a stand-alone document, the AISC HSS *Specification* was fully incorporated into AISC 360-05, *Specification for Structural Steel Buildings*. The section you are referring to in the HSS *Specification* is included in AISC 360-10 Section H1.3. Considering flexure alone, there is no need to evaluate X- and Y-axis flexure separately for a round shape. Because the flexural strength is not axis-dependent, the resultant moment can be calculated and directly compared to the flexural strength.

There was a caveat when combining this with compression loads in the HSS *Specification*. In order to be able to use the resultant flexural stress, the effective length of the column in compression must be the same in all directions.

There is a similar approach given in AISC 360. It considers flexure and buckling about the same axis and separate limit state for buckling out-of-plane.

Heath Mitchell, S.E., P.E.

HSS Connection Design

I am trying to design a fully restrained moment connection for a structure consisting of rectangular HSS. My situation involves a Cross-Connection as described in Chapter K3 of the AISC *Specification*. AISC *Specification* Section K3.3a contains a limit on the aspect ratio of the member, but doesn't say to which member it is referring—the branch or the chord. Please explain why the aspect ratio is a limiting factor and to which member it applies.

The aspect ratio restriction in AISC *Specification* Section K3.3a(6) is referring to both the branch and the chord. This is clarified in the 2010 AISC *Specification* (a free download at **www.aisc.org/2010spec**). AISC *Steel Design Guide No. 24*, *Hollow Structural Section Connections* (**www.aisc.org/epubs**) also addresses this and does have design examples for HSS moment connections.

The Commentary to Chapter K explains that the limits of applicability generally represent the parameter range over which the design provisions have been verified in experiments. They have also been set to eliminate the occurrence of certain failure modes for particular connection types. They set the scope of connections for which the AISC *Specification* is intended to be used. Designs outside of these limits are allowed, but because they are outside of the scope of the specification, the design needs to be based on the experience and judgment of the engineer.

Heath Mitchell, S.E., P.E.

Impact Loading

The 9th Edition AISC *Steel Construction Manual* contained information on impact loads for elevators. Where can I find this information in the AISC *Manual*?

This information used to be in the AISC *Specification*. The *Specification* no longer specifies loading criteria, but instead refers to ASCE 7. In ASCE 7-10 Chapter 4, Live Loads, the impact loads for elevators are addressed in Section 4.6.2. This section refers the reader to ASME A17.1.

Erin Criste

Single Angle LTB

I am trying to calculate the capacity of a L6×6×¾ bent about its z-axis. The provisions in AISC *Specification* Section F10 do not seem to cover lateral-torsional buckling about the minor principal axis. Why is this?

Lateral-torsional buckling isn't possible for a member bent about its minor principal axis. In general, yielding and leg local bucking apply for minor axis bending of angles. There is a user note to this effect in the 2010 *Specification*, which can be downloaded at **www.aisc.org/2010spec**.

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

Double Angle Connections

What angle leg sizes are valid for use with Table 10-1 in the 14th Edition AISC *Steel Construction Manual*?

The leg sizes are largely immaterial in Table 10-1, except that the assumption of L_{eh}=1¼ in. described on page 10-10 must be maintained.

In contrast, the eccentricity on the weld group in Table 10-2 is dependent on the leg size, so this dimension is specifically addressed in Table 10-2.

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**.

Heath Mitchell is director of technical assistance, Erin Criste is staff engineer, technical assistant and Tom Schlafly is director of research at AISC. Larry Muir and Brad Davis are consultants to AISC.

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:

Steel Solutions Center
One East Wacker Dr., Suite 700
Chicago, IL 60601
tel: 866.ASK.AISC • fax: 312.803.4709
solutions@aisc.org