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.

Stiffened Seated Connection Strength

I am designing a stiffened seated connection to a wide-flange column web where the horizontal seat plate is welded to the column web and to both column flanges. The stiffener plate is also welded to the column web. Is the strength of this connection equal to the value in AISC *Steel Construction Manual* Table 10-8 plus the value of the welds connecting the seat plate to the column flanges on each side?

The AISC S*teel Construction Manual* procedure used in Table 10-8 puts the weld where it is best suited to carry the vertical shear load from the supported beam web to the column. Only 20% of the vertical weld length, L, on each side of the vertical stiffener plate is considered effective. If more of the seat plate is welded to the column web or flange, it is not considered effective in carrying vertical load due to the flexibility of the seat plate and the beam flange that sits on it. If more than the aforementioned 20% of seat plate weld is used, it is considered to carry no load in the *Manual* procedure. Likewise, and for the same reason, welding to the inside of the column flanges, as you have described, will not increase the stiffened seat capacity to carry vertical load.

Bill Thornton, P.E., Ph.D.

Base Plate Shear Transfer

AISC *Steel Design Guide No. 1*, *Base Plate and Anchor Rod Design*, 2nd Edition, discusses three methods of transferring shear to the concrete at a base plate: friction, shear lugs and anchor rods. However, I do not see a discussion on whether any of these methods can be used in combination with one another. Can the strengths from these mechanisms be combined?

I am not aware of a standard procedure for combining these resistances.

The load-deformation behavior of the three mechanisms is likely to be very different, so it could be expected that a great deal of deformation would be necessary to develop the strength of each. Also, we know these mechanisms do not behave in a perfectly plastic manner. One such example is the concrete breakout limit state for a shear lug. We also know that friction does not develop and then maintain that resistance indefinitely. Slip does not eliminate friction, but the friction is now based on a kinetic friction, which is lower than static friction.

This situation is similar to why we do not allow the full strengths of bolts and welds to be summed, and why we do not sum the strengths related to both bearing and slip resistance in pretensioned bolted joints. Surely some additive effects exist but we are not confident that we can accurately predict the behavior. Instead, we neglect one mechanism and base the strength solely on the other. That is the case here as well.

Larry S. Muir, P.E.

Eccentrically Loaded Bolt Groups

In the AISC 14th Edition *Steel Construction Manual*, formulas are provided for the analysis of eccentrically loaded bolt groups based on the instantaneous center of rotation method. In this analysis, the actual eccentricity is used; in earlier editions, the eccentricity was allowed to be reduced based on the number of fasteners in a vertical row using the equation,

$$e' = e - \frac{1-n}{2}$$

Was that method unsafe? I have designed many connections using this method and am wondering why it was changed.

The "Effective Eccentricity Method" you are referring to appeared first in the 6th Edition AISC *Steel Construction Manual* and continued through the 8th Edition *Manual*. In the 6th and 7th Editions, the eccentric bolt group tables were based on this method, which was developed in 1964 by T.R. Higgins. While this method still was given as an option in the 8th Edition, the "Ultimate Strength Method" of Crawford and Kulak was the preferred method, and the one used to produce the eccentric bolt group tables. Every edition of the *Manual* since the 8th Edition has used the ultimate strength method in these tables.

The "Effective Eccentricity Method" was not unsafe. It was eliminated because, as stated in the 8th Edition (see page 4-58), it did not provide a uniform factor of safety.

The AISC *Manual* is a compilation of guidance that follows the requirements in the AISC *Specification* and other related standards, and provides additional recommendations to cover aspects of design and construction that do not have code requirements associated with them. As the AISC Specification Committee has not adopted any particular method to design eccentrically loaded bolt groups, you are still free to use the "Effective Eccentricity Method" based upon your own engineering judgment. I would not use it for the aforementioned reasons, however, and reviewers of your work may have reservations about it because it is not in the *Manual*.

As an alternative, you might consider the basic "Elastic Method." It is still in the *Manual* because it is known to be conservative, satisfies all "first principles" of structural mechanics (equilibrium, compatibility, constitutive requirements), and can be used without a computer for any bolt group configuration not presented in a *Manual* table. It is what I call a "desert island" solution.

Bill Thornton, P.E., Ph.D.

steel interchange

Shape Properties

Where can I find the AISC steel shapes properties and dimensions?

There are two main sources for that information, and a third source you may find interesting.

- First, the AISC *Steel Construction Manual* contains information on dimensions and properties—see Part 1 of the *Manual*.
- Second, the dimensions and properties can be found in the AISC Shapes Database, which is an Excel file that is available for free download at www.aisc.org/shapesdatabase.
- Third, some independent individuals have used the AISC data and created their own applications. For example, one includes auto-generated dimensioned diagrams, whereas the AISC Shapes Database does not have any diagrams. One place where such things are posted for use by others is www.steeltools.org.

Martin Anderson

Welder Identification

Is there an AISC requirement that welders, welding operators or tack welders place a unique, hard-stenciled identification mark near the welds they have produced?

Prior to the 2010 edition of the AISC *Specification* (ANSI/AISC 360-10), there had been no requirement for welders to identify their welds. With the introduction of Chapter N in ANSI/AISC 360-10, a welder identification system is required (refer to Table N5.4-1, and Note 1 to the table).

This provision was not added to penalize contractors and welders, but recognizes that an ID is necessary in order to apply Section N5.5e, which permits reduction of ultrasonic tesing (UT) based on welder performance. It further offers contractors and inspectors a means of evaluating welder performance and repair rates.

While a hard stencil is acceptable, it is not mandatory. Contractors are permitted to use any tracking system they prefer in order to identify which welder welded a joint, including mapping or recording on drawings. Most commonly, welders use a marker such as a lumber crayon and simply put their initials in close proximity of the weld joint.

There is no requirement for tack welders, nor has there been in the past.

Keith Landwehr

Welding Electrode Storage

On one of our projects, the welding electrodes are not being stored in an oven. Our structural drawings specify the use of E7018 electrodes. Is it acceptable not to store the electrodes in an oven?

Yes, provided the exposure is limited. This topic is discussed in Section 2.2.6 of AISC *Steel Design Guide No. 21*, *Welded Connections—A Primer for Engineers*, which is available as a free download for AISC members at www.aisc.org/dg. Welding can be performed without electrodes being stored in an oven, but the electrodes must be properly handled. Specifically, low-hydrogen consumables must be kept dry in order to produce sound welds.

If the electrodes spend a significant amount of time exposed and out of the sealed packaging they must be placed in an oven to remove moisture. If they are removed from the sealed containers and then used within the prescribed time in AWS D1.1, then the oven may not be necessary.

Generally, the permissible exposures for E70 electrodes range from four hours to nine hours. However, AWS A5.5 allows testing to be used to establish an appropriate exposure time, up to the maximum exposure limit of 10 hours. Without the establishing tests, an E7018 electrode without the "R" designation would have an exposure limit of four hours. E7018 electrodes also are available with the "R" designation, which indicates they can be exposed for up to nine hours without additional testing.

Larry S. Muir, P.E.

Fire Design of Concrete-Filled HSS Columns

On page 28 of AISC *Steel Design Guide No. 19*, *Fire Resistance of Structural Steel Framing*, it states that two vent holes in opposite directions are needed at the top and bottom of concrete-filled HSS columns. Are these vent holes required by AISC or is this simply a recommendation?

The requirement for holes is stated in the ASCE 29 standard. *Design Guide 19* refers to this standard when dealing with unprotected concrete-filled HSS, and is the only place in the U.S. where the fire resistance of concrete-filled HSS is standardized. Holes are absolutely necessary to avoid pressure build-up inside the HSS due to internal water being heated and converted to steam. Absence of holes could result in an explosion of the heated concrete-filled HSS.

Erin Criste, LEED GA (with help from Farid Alfawakhiri of AISI)

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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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