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**.

steel interchange

After three years as AISC's director of technical assistance, Heath Mitchell has decided to move on to join the company his family owns, G.W.Y., Inc. The company serves the structural bolting industry, selling, repairing and renting a variety of installation tools for conventional hex-head and tension-control bolts, and Heath will be helping to manage day-to-day operations, expand the product line and introduce current products into new markets. He'll also continue as a consultant to the Steel Solutions Center.

We're sad to see Heath go but happy to announce that Larry Muir, who has already been working for us for many years as a part-time consultant in technical assistance activities, has taken over his role. Larry has also been a consultant with his own engineering practice for a number of years, and fefore that he was president of the engineering division of AISC member Cives Steel Company. All the while, Larry has been a very involved volunteer on AISC's Committees on Specifications and Manuals.

Astute readers will remember that Larry recently was announced as the 2014 recipient of the AISC T.R. Higgins Lectureship Award. (He made it into that distinct category just in time, as members of the AISC staff are not eligible to win it. Having been selected before joining the staff means Larry is quite unique as a Higgins recipient!)

In celebration of Larry's new role at AISC, this month's SteelQuiz features a selection of questions Larry has answered.

Charles J. Carter

Plate Bending

A debate is raging in our office. For years, the allowable bending stress in base plates was $0.75F_y$. Beginning with the 13th Edition AISC *Steel Construction Manual*, however, AISC appears to stipulate $0.60F_y$ for ASD. 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 and 2010 AISC *Specifications* include a slight gain in strength over the 9th Edition ASD.

In the 9th Edition, 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 $0.6F_y$. The quotient 0.66/0.6 equals 1.1. This approximates the ratio of Z_x/S_x using the lower bound value for wide-flange beams.

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

Eccentrically Loaded Bolts

I am analyzing an eccentric bolt group and cannot use the eccentrically loaded bolt group tables in the AISC *Manual* due to a non-standard spacing. How can I calculate the coefficient *C*?

The Instantaneous Center of Rotation Method used to determine this value is described in Part 7 of the AISC *Manual*. It is an iterative process. Some engineers have written a program to do this, but you can also use a spreadsheet. Some structural engineering textbooks also present the calculations, which are readily adaptable to a spreadsheet.

Once you have built the spreadsheet or program, you can proceed in one of two ways. First, you can simply guess at a location of the instantaneous center until you satisfy equilibrium. This is not as bad as it may sound if you are only doing this occasionally. Second, you can use the spreadsheet goal seek function to find the location of the instantaneous center. For a symmetrical bolt group with a vertical load, the instantaneous center of rotation will be located along the line perpendicular to the load passing through the bolt group centroid. Knowing this makes the job easier.

There are a number of programs available online (many free) to do this work for you. Some will supply the location of instantaneous center of rotation. Once the center of rotation is known, the results can be easily verified.

If you would rather write a program, there is also an AISC *Engineering Journal* article by Brandt that presents a program in FORTRAN that can be used to find the coefficient C of the bolt group (this is in the 2nd Quarter 1982 issue).

Application of Q_f

When the variable Q_f is used in AISC 360-10 Section K1, it is typically applied as a multiplier outside of the bracketed portion of the equation. As such, it has an effect on the entire nominal strength calculation. However, Equations K1-12 and K1-13 for the limit state of wall plastification have Q_f inside the bracketed portion of the equation. Is it correct for these two limit states that Q_f only applies to a portion of the nominal strength equation?

I actually had the same question when I was reviewing this section of the AISC *Specification*. Q_f is in the correct location (inside the brackets) in 2010 AISC *Specification* Equations K1-12 and K1-13. The equation is based on a yield line approach. Since the force in the member has a greater effect on the strength of the yield lines transverse to the chord axis and little effect on the yield lines parallel, Q_f is applied to only those portions where it has a significant effect. This approach predicts a strength consistent with test results.

K-Area Welding

Does AISC 360 prohibit welding in the *k*-area of a wide-flange shape?

The AISC *Specification* does not prohibit welding in the *k*-area. There have been some reported problems with welds made in the *k*-area, so it is generally avoided, when possible. However, there are times where welding in this area is required. For more information on this topic you can refer to the *MSC* article "AISC Advisory Statement on Mechanical Properties Near the Fillet of Wide Flange Shapes and Interim Recommendations, January 10, 1997" (02/97).

AISC 358 Section 3.6 (and its associated Commentary) describes requirements for continuity plate corner clips. Although this is not a direct prohibition of welding in the *k*-area, the resulting corner clip geometry is intended to avoid welding in the *k*-area.

When welding in the *k*-area is performed, it should be noted that AISC 360-10 Chapter N Table N5.4-3 requires visual inspection: "When welding of doubler plates, continuity plates or stiffeners has been performed in the *k*-area, visually inspect the web *k*-area for cracks within 3 in. (75 mm) of the weld."

Base Plate Shear Transfer

AISC Design Guide 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 summing 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 or why we do not sum the strengths related to both bearing and slip resistance in pretensioned bolted joints. Surely some additive effect exists but we are not confident that we can accurately predict the behavior so we instead neglect one mechanism and base the strength solely on the other.

Moment Connection to HSS Column

I am working on the design of a moment connection between a wide-flange beam and an HSS column. The beam flange is wider than the HSS column it connects to. According to AISC 360-10 Section K1.3b, B_p/B must be less than or equal to 1. Do we need to taper the flanges of the beam to be the same width of the column at the joint or can we keep the normal flange width with no taper and use $B_p/B = 1.0$?

If fatigue is not a concern for your connection, there is no need to taper the flange. The flange width should be assumed equal to the width of the HSS for calculation purposes. In AISC 360-10 Chapter K, β will then be equal to 1.0.

AISC Design Guide 24 Chapter 4 provides guidance related to these connections. Example 4.3 addresses the directly welded connection and treats the flange as a transverse plate. However, this example is configured such that the beam flange is narrower than the HSS width.

For this type of connection with a beam flange width greater than or equal to the HSS column width, the applicable checks are Equations K1-7, K1-9 and K1-10 or K1-11. Equations K1-9, K1-10 and K1-11 are similar to the local web yielding and crippling checks for wide-flange beams in Section J10. Equation K1-7 incorporates an effective width concept. If a CJP groove weld between the flange and the HSS wall is not used, this effective width concept also should be incorporated into the design of the weld, as shown in Equation K4-4.

Fatigue applications may require tapering.

Larry Muir is AISC's new director of technical assistance.