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 i2msany. Beginning with

ruction Manual,

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

K-Area Welding

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

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

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.

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

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:

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