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.

Changes to Delegated Connection Designs

I have a contract in which the connections have been specified as Option 3 connections per the 2010 AISC *Code of Standard Practice*. That is, the connections are to be designed by our engineer with sealed calculations provided as the substantiating connection information. Shears and moments (for moment connections) were provided in the design drawings.

We submitted representative samples of the substantiating connection information, which showed the types of connections (shear tabs and bolted flange-plate moment connections) and the calculations we planned to submit to justify our connections. The final substantiating connection information was consistent with the representative samples, and we submitted it to the Structural Engineer of Record (SER) with the shop and erection drawings as a part of the approval process as given in Section 4.

The returned shop drawings were marked up to require one additional row of bolts in each shear connection and two additional bolts in each moment connection flange plate, even though the calculations we submitted demonstrate compliance with the requirements on the design drawings and in the AISC *Specification*, and recommendations in the AISC *Manual*.

When we asked why the additional bolts were necessary, the SER responded that they wanted more strength in the connections. We objected because of the additional costs of revising our shop drawings and changing the connections. The SER responded that the 2010 AISC *Code of Standard Practice* gives them final authority to require this, per Section 4, and additional compensation is not required. Is this opinion consistent with the intent of the *Code*?

No. The sentence in Section 4 acknowledges the SER's final authority for the completed structure. In the case of a disagreement about connection design, Section 4 is intended to ensure that all parties are aware that the SER maintains responsibility for the entire completed structure, including connections. It is not intended as a means for the SER to require changes to connections that conform to the applicable requirements without consideration of the costs involved in the changes. While the additional rows of bolts may be well-intentioned additions on the part of the SER, they are essentially changes to the contract requirements, and as such are subject to equitable adjustment by change order.

In specifying Option 3 the SER must also provide the appropriate criteria to which the connection design must conform. If this is accomplished by reference to AISC standards and the building code, then any connection that complies with these requirements must be accepted. If there are special requirements that exceed the AISC standards, they must be stated in the contract documents and the connection design must comply with these alternative standards. However, if there is a disagreement with regard to the interpretation of the specified criteria, it is the SER's authority to settle such a disagreement at his/her discretion. Note that the *Code* discusses approval of representative samples of the substantiating connection information early in the process. This is the time to raise and discuss disagreements about interpretations of the specified criteria.

Specifying Option 3 for the connections, and subsequently requiring changes to connection designs that conform to the specified requirements, is not compatible with this option for connection design delivery. If the SER requires changes to conforming connections designed by the delegated engineer, the process now follows Option 1 for connections. That is, the SER is taking back the design of those connections from the delegated engineer. The SER who does this should identify the specific changes required, but must recognize that there may be cost and schedule adjustments involved to make these changes.

Charles J. Carter, S.E., P.E., Ph.D.

Fillet Weld to Skewed Plate

I am trying to determine the equivalent fillet weld size for a skewed shear plate connection. AWS D1.1 Table B.1 only lists dihedral angles over 60°. In my case the dihedral angle is 54°. How is the equivalent fillet weld leg size determined for angles other than those listed in the table?

Section 2.4.3 of AWS D1.1 covers skewed T-joints. Section 2.4.3.3 covers dihedral angles between 60° and 30°. These joints require the use of the Z-loss dimension in order to calculate the effective throat.

This also is discussed in Parts 8 and 10 of the 13th Edition AISC *Steel Construction Manual*. There is a discussion of skewed shear connections starting on page 10-149. Table 10-13 summarizes welding requirements for skewed shear tab connections. Table 8-2 (on page 8-36) is a reproduction of a figure in AWS D1.1 that shows the effective throats of fillet welds with various dihedral angles.

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

Bracing Connection Design

The 13th Edition AISC *Steel Construction Manual* discusses the analysis of existing diagonal bracing connections using the Uniform Force Method. How is objective function given on pages 13-10 and 13-11 used in distributing the moment in the bracing connection and how is the Lagrange multiplier calculated?

You don't actually have to calculate the Lagrange multiplier. The solutions for α and β given in the *Manual* on page 13-11 are all you need to calculate the force distribution in the connection.

If you still want to calculate it, differentiate the objective function ξ with respect to α, β and λ, and set the result of each differentiation to zero. This gives you three equations in three unknowns, α, β and λ, which can then be solved. The solution on page 13-11 was obtained in this way.

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

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Bolting Cost Comparison Article

There was a *Modern Steel Construction* article that discussed the relative costs of snug tight, pretensioned and slip-critical connections. In which issue of *MSC* did it appear?

Perhaps you are thinking of an article by David Ruby that appeared in the May 2003 issue. This article has been reprinted/summarized from time to time, particularly the table on the second page. Back issues of *Modern Steel Construction* magazine are available in electronic form at **www.modernsteel.com/backissues**.

Martin Anderson

Maximum Fillet Weld Size

AISC *Specification* Section J2.2b has a requirement that the maximum size of a fillet weld on material ¼ in. or more in thickness shall be 1⁄16 in. less than the material thickness. Would this requirement apply to the case of a column-to-base-plate connection?

The criteria for maximum size of fillet welds found in AISC *Specification* Section J2.2b is not intended to apply to the base-plate-to-column connection that you describe. The language in the *Specification* refers to welds "along edges." This is intended to describe a fillet weld with a leg that is applied on the thickness of the connected part, such as for a lap joint with a plate. For additional information on this topic, see the short discussion and figure illustrating this provision in the Commentary to Section J2.2b on page 16.1-331.

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

IMF Panel Zone Strength

AISC 341 Section 9.3a requires that, as a minimum, the panel zones in special moment frames be designed based on the expected moments at the column faces due to plastic hinge formation in the beam. However, Section 10.3 does not have a similar requirement for intermediate moment frames. Why is the same capacity design philosophy not noted for the special and intermediate resisting frame?

The Commentary to AISC 341-10 specifically addresses this question. Although it is currently being typeset and is not yet available, it should be soon. I have copied the relevant commentary, below, for your convenience.

> "The panel zone for IMF is required to be designed according to Section J10.6 of the *Specification*, with no further requirements in the *Provisions*. As noted in the Commentary to Section E2.2, panel zone yielding is permitted as part of the inelastic action contributing to the drift capacity of the IMF and the requirements of the *Specification* are considered adequate for the expected performance."

The Commentary to Section E2.2 states, in part:

> "While the design for SMF is intended to limit the majority of the inelastic deformation to the beams, the inelastic drift capability of IMF is permitted to be derived from inelastic deformations of beams, columns and panel zones."

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

Evaluation of Existing Structures

Do you have any publication for evaluation of existing steel structures?

AISC Steel Design Guide No. 15, *Rehabilitation and Retrofit Guide—A Reference for Historic Shapes and Specifications* (available as a free download for AISC members at **www.aisc.org/dg**) was developed for this purpose.

Additionally, Appendix 5 of the AISC *Specification* covers evaluation of existing structures. The *Specification* is available as a free download to both members and non-members at **www.aisc.org/2010spec**.

Aside from those two main references, there are a number of relevant articles in *Modern Steel Construction* magazine, such as the SteelWise article from the February 2007 issue (available as a free download at **www.modernsteel.com/backissues**).

Lastly, the AISC Shapes Database exists in a historic form, listing the various dimensions and properties of historic structural steel members produced in the U.S. It is available at **www.aisc.org/shapesdatabase** and is also a free download.

Martin Anderson

Existing Column Out-of-Plumb

We have been asked to evaluate an existing structure where a number of the columns exceed the AISC *Code of Standard Practice* out-of-plumb tolerance limit. Does AISC have design recommendations for when columns exceed the erection tolerances established by the *Code*?

If all parties are in agreement that the columns being out-of-tolerance is acceptable, the existing structure can be modeled accounting for the existing eccentricities. The Direct Analysis Method in AISC 360 Appendix 7 can be used to analyze the structure for stability effects. Section 7.3 (2) allows for the existing geometry to be used in lieu of notional loads.

Charles J. Carter, S.E., P.E., Ph.D.

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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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