Steel Interchange

Steel Interchange is an open forum for *Modern Steel Construction* readers 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 to solve, please forward it to *Modern Steel Construction*. At the same time, feel free to respond to any of the questions that you have read here. Please send them to:

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Answers and/or questions should be clearly presented. E-mail submittals and/or e-mail attachments are welcome.

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****** Questions and answers can now be e-mailed to: grubb@aiscmail.com ******

The following responses from previous Steel Interchange columns have been received:

Question from April 1999:

For calculating the allowable bending stress on beams (ASD 9th edition, Chapter F), can I assume a pair of brackets welded from the top flange to bottom flange on both sides of the beam serves as a lateral support to reduce the laterally unsupported length of the compression flange?

Emha Antariksa

Lateral bracing must prevent both twisting and lateral deflection of the member's cross-section at lateral brace points. Stiffness, strength, and spacing of the lateral bracing must be adequately chosen to prevent lateral torsional buckling (LTB) before the design bending strength requirement is satisfied.

With this in mind, does the lateral bracing prevent both twisting and lateral deflection of the member's cross-section? In addition, are the stiffness, strength, and spacing of the brace(s) adequate? With these concerns in mind, the bracing arrangement suggested by the question above appears inadequate because twisting and lateral deflection of the member's cross-section may still be of concern.

Several references for the design of lateral bracing are:

· *Structural Steel Design: LRFD Approach*, 2nd edition, J. C. Smith. (Chapter 7, Bracing Requirements)

· *Steel Structures: Design and Behavior*, 4th edition, C. G. Salmon and J. E. Johnson. (Section 9.13, Lateral Bracing Design)

· *Fundamentals of Beam Bracing*, J. A. Yura, in Is Your Structure Suitably Braced, Structural Stability Research Council.

Timothy M. Young
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What are "banging bolts" and how do they affect structural steel framing?

Periodically, I've answered a call where an engineer architect, building owner, or some other related entity has a client or some tenants that reported hearing sharp sounds like rifle shots coming from "the building". Some heard one. Some have heard them a handful of times over a number of years.

What they are all describing is something called "banging bolts". Bolt-banging results when bolted connections slip into bearing under load. In the majority of buildings, bolted connections settle in during construction and the occupants never hear a thing. In other cases, the slip into bearing occurs after occupancy. It's an instantaneous occurrence that makes a loud sound...just like a rifle shot. Fortunately, there is no structural significance to this as the connections can carry the load in bearing by design. That is, there's no threat to life safety here. It just scares the heck out of everyone who hears it from the president of the company down to the summer intern.

To minimize the occurrence (I don't think you could hope to eliminate it entirely), I suggest that engineers specify their bolted connections as snug-tight bearing connections whenever possible. This minimizes the potential for the intentional or incidental slip resistance that eventually might lead to a connection that doesn't slip into bearing until after occupancy.

Also, I suggest that erectors tell their crews not to tighten any bolts until after the drift pins have been removed from the connections. Leaving the drift pin in as bolts are installed and tightened tends to center the holes in the connected plies and increase the potential for slip in the connection.

Actually, the latter recommendation (about drift pin usage) is probably more important than the first. Unfortunately, some decision makers take more extreme measures, like unilaterally requiring field-welded connections, which may do nothing more than eliminate steel as an option.

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I've noticed that the properties for HSS sections are slightly different in the *HSS Connections Manual* (1997) than those in the *LRFD Manual*, 2nd edition (1994). Most properties have lower values in the newer manual. Why?

While preparing the HSS Connections Manual, AISC discovered that HSS producers routinely produce their shapes to a wall thickness that is near the minus tolerance in ASTM A500 (minus 10%). From a production standpoint, they can do this with regularity because they order their raw materials (plate) from plate producers who can meet tolerances that are much tighter than those to which they produce HSS. From a commercial standpoint, HSS producers do this because they are then able to sell a product by weight for 100% of the theoretical but only need to provide 90% of it per ASTM A500. This tolerance goes back to the old days of production when a more distributed variation in thickness was common.

AISC supported an ASTM ballot revision to change the tolerance in the ASTM specification to bring the wall thickness tolerance to within the normal variations that are present in other structural shapes and eliminate this problem. The measure was defeated in that ballot, however, as the practice is apparently standard in the HSS industry. Nonetheless, this left us with a design issue.

To ensure safety, our *Specification for the Design of Steel HSS* requires that the wall thickness used for design be taken as **0.93 times the nominal wall thickness** in electric-resistance-welded (ERW) HSS. This brings the design value in line with the expected variation in other structural shapes.

From a design perspective, the thickness issue is a more serious problem for connections, where the wall thickness gets squared and cubed in some of the equations for design strength. For example, taking 0.93^3 gets you down to about 0.8 or about 80% of the strength based upon nominal wall thickness when the thickness term is cubed. In design strength equations for member design, however, the thickness term is normally not raised to a power and the effect is much less than for connections, although not negligible.

As a first step upon discovery of the issue, AISC published an advisory to designers. This advisory first appeared in *Modern Steel Construction*. The issue has also been addressed at various AISC functions, including the North American Steel Construction Conference, the HSS Connections short course, and the AISC national lecture series. The *HSS Connections Manual* thoroughly addresses the change, and the next LRFD manual will include the new HSS section properties.

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New Questions

Stair stringers are typically constructed from channels. Frequently, the stair stringers must be mitered to accommodate landings and connections to floor levels. The flange forces cause a prying action on the web of the channel in the mitered joint. Stiffener plates between the flanges are often required to resist these prying forces. Is there some point where the web of the channel can resist these prying forces thereby eliminating the need for a stiffener plate?

Allan T. Goffe
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In designing a partially restrained connection consisting of top and bottom angles and a simple shear connection, the point of inflection is between the beam flange bolt gage line and the face of the connection angle. In connections of this type the driving of bolts is difficult and it may be impossible to tighten the bolts even if the gage lines are staggered. For this reason, instead of using bolts in the top flange of the beam, can welds that meet the strength requirements be used if the weld does not encroach on the inflection area?

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