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.

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If you have a question or problem that your fellow readers might help you to 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 at:

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OSHA Rules for Erection

I was reading through the AISC advisory on the new OSHA rules for steel erection [available at www.aisc.org]. There has been talk of requiring safety-type double connections for beams. This advisory only talks about needing safety double connections only for columns or beams framing over columns. It doesn't say safety connections are needed for ALL double connections. How this will be interpreted?

The new OSHA regulations regarding double connections only apply to those that occur through:

(1) column webs
(2) webs of girders that frame continuously over the tops of columns

In the latter case, the OSHA regulations only apply to the connection at the column over which the girder frames, but not to other connections of infill beams through that same girder web away from the location of the column. Their reasoning is that the ironworker is at risk in the above two cases because he or she is sitting on the beam that drops when the column fall-away hazard shifts from concern to reality. Because the ironworker sits on the girder when making the infill beam double-connections, and because the "box" is already built when the infill beams are being placed, the risk is not the same.

Incidentally, OSHA only requires that provision be made to support the framing members with two permanently installed bolts or the equivalent thereof. The clipped-end-plate-type safety connection they show is one such way to meet this requirement. Staggered clip angles, erection seats, and one-sided connections (like single plates, single angles and tees) can also be used successfully to mitigate the double connection concern and satisfy OSHA regulations. I'm sure there are other ways to do it, too.

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Shelf-Angle Design

What information is available to provide guidance in the design of a shelf-angle and its connection to a wall in particular, with the bolt in tension and the lower edge of the vertical leg of the angle in compressive contact with the wall. What is the stress distribution in the angle? Is only part of the angle effective in resisting the applied loads based on the spacing of the bolts?

Greg Michel, P.E.
Mani Muthiah, P.E.

Referring to Chapter 6, "Design of Connections" of the *PCI Design Handbook*, 4th ed., four equations are presented in subheading 6.5.9, "Connection Angles" (pp. 6-20 to 6-23) which may be of assistance. The following equations are taken from page 6-22 of this reference.

The minimum thickness of a non-gusseted angle, loaded in shear (or vertically) is:

$$t = \sqrt{\frac{4V_u e_v}{\phi F_y b_n}}$$

The tension on the bolt, subjected to this loading is:

$$P_u = V_u \frac{e_v}{e_i}$$

The minimum thickness of a non-gusseted angle, loaded axially (or horizontally) is:

$$t = \sqrt{\frac{4N_u g}{\phi F_y b_n}}$$

The tension on the bolt, subjected to this loading is:

$$P_u = N_u\left(1 + \frac{g}{e_i}\right)$$

The following notations are as follows:

b_n = net length of angle, in.
e_i = center of bolt to horizontal reaction, in.

e_v = eccentricity of vertical load, in.
F_y = yield strength of structural steel, ksi
g = gage of angle, in.
N_u = factored horizontal or axial force, kips
V_u = factored shear force, kips
ϕ = 0.9 = strength reduction factor

On page 6-74, this reference also gives both the shear and axial strength of angles having several thicknesses in Tables 6.20.13 and 6.20.14. One also may desire to consult several of the references cited on pages 6-55 and 6-56.

Timothy M. Young
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Snug-tightened Bolts Make Sense (revisited)

Regarding the March 2001 Steel Interchange, I have to say that I somewhat disagree with [Charles Carter's] conclusion that snug-tight bolts are cheaper than pretensioned bolts. I would certainly agree that allowing fabricators to use load values for bolts in bearing connections (N or X-type bolts) is cheaper slip critical (SC) connections as the allowable load in an N-type bolt is much higher than the allowable load in an SC type bolt (thus resulting in smaller, more economical connections).

However, the cost savings [may not be apparent] if the engineer forces the fabricator/erector to not pretension a bolt in a connection designed to use N-type bolts. In other words, if the fabricator/erector is told that he cannot use tension control (TC) type bolts and pretension the bolts in bearing connections. This requirement forces the detailer to note all connections with snug-tight only bolts and forces both the fabricator and erector to use two types of bolts on the project...since TC type bolts (with the splines) are not well suited for snug tight installation methods. Additionally, the erector's bolting crew must carry two types of bolts around in the field and carefully review the erection plans when installing the bolts (this is very time consuming and expensive).

In conclusion, I would certainly agree with Mr. Carter's response as long as the engineer does not care if the bolts in a bearing type connection are pretensioned. If he doesn't care, then the fabricator and erector can use TC type bolts (the generally preferred bolt type) regardless of how the connection is designed.

Your comments are well taken. It is very important to properly distinguish between snug-tightened joints, pretensioned joints and slip-critical joints. The new 2000 RCSC Specification, which is available for free download at www.boltcouncil.org, gives very good clarification of the differences between these three types of joints in Section 4.

I agree that bearing connections with N- or X-type bolts are cheaper than slip critical connections as the strength of a bolt in bearing is much higher, resulting in smaller, more economical connections when bolt bearing/shear strength is the controlling limit state. When permitted, bearing connections should be used.

I also agree that it is a needless and significant cost item when it is erroneously specified that bolts in bearing connections should not be pretensioned. The RCSC Specification specifically recognizes that the level of pretension present in a snug-tightened joint is not a consideration (see RCSC Specification Section 9.1). That is, just because a joint is specified as snug-tight does not mean that the bolts cannot be pretensioned; rather it means that they don't *have* to be pretendsioned. Erroneously specifying that bolts not be pretensioned often precludes the use of tension control (TC) type bolts, which commonly result in cost savings due to simplified detailing, procurement and installation.

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New Question

Uniform Force Method

This question concerns the design of horizontal brace connections using the Uniform Force Method (UFM), "bearing-type" bolts and single clip angles welded to the gusset plate.

The UFM defines both shear and axial forces for the gusset plate connectors to the supporting surfaces. The single clip's outstanding leg (OSL) bolts will need to be designed for eccentricity. Thus these OSL bolts will need to be designed for the combination of tension and shear forces.

When I use the tables in the 2nd ed. LRFD manual to calculate how many bolts I will need for the shear force eccentricity, I get an allowable "capacity" value which is based upon the instantaneous center of rotation method. How do I calculate the "actual" bolt shear stress for the reduction of the allowable tension stress per LRFD Table J3.3? My thought was to determine the number of bolts for the shear based upon the tables and then use the "elastic" method for determining the "actual" shear force/stress. Any thoughts?

Dan Hakes, P.E.