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.

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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:

Steel Interchange
Attn: Keith A. Grubb, S.E., P.E.
One East Wacker Dr., Suite 3100
Chicago, IL 60601-2001
fax: 312/670-5403
email: grubb@aiscmail.com

Question from February 2000:

Are we dangerously kidding ourselves when we specify "X" high-strength bolts? The allowable shear stresses are increased more than 40% for threads excluded from the shear plane. The installer of these bolts is an ironworker working many feet above the ground with a bolt bag filled with possibly many varieties of bolts. How does he know that the connection he is working with at the moment requires "X" bolts?

There is also a very small tolerance for installing a bolt with the threads excluded from the shear plane. For example; a bolt with a ½" shank (unthreaded portion under the bolt head) that is installed from the clip-angle side (5/16" thickness) to the flange of a W8x40 column flange has the threads excluded. This same bolt installed form the column flange side (9/16" thickness) does not exclude the threads.

Another aspect to consider is the ability of the structural steel inspector to verify that a bolted connection he observes after erection has in fact excluded the threads from the shear plane since the shank of the bolt is hidden from view.

David E. Ayers, P.E.
ESource Detailing
Richmond, VA

ASTM A325 and A490 high-strength bolts are purposefully proportioned and produced so that the threads can be excluded from the shear plane(s) when it is desired to do so. Such variables as the grip, number and placement of washers, bolt length, thickness of the ply closest to the nut, and dimensional tolerances all play a part in proper design to exclude the threads. For a complete discussion of this, see "Specifying Bolt Length for High-Strength Bolts" in the 2nd quarter 1996 issue of the AISC *Engineering Journal.*

Although it is often possible to exclude the threads in specific cases with lesser ply thicknesses (see the above referenced paper), the following general rules of thumb will always work. For 3/4-in. and 7/8-in. diameter high-strength bolts, a ply thickness of 3/8 in. adjacent to the nut will always exclude the threads. For 1-in. and 1 1/8-in. diameter high-strength bolts, a ply thickness of 1/2 in. adjacent to the nut will always exclude the threads. These thicknesses can be reduced to 1/4 in. and 3/8 in., respectively, if an ASTM F436 washer is used under the nut.

As far as the QA/QC issues you raised, you can ensure that threads will always be excluded from the shear plane without need for specific inspection quite simply by specifying all exterior plies in the appropriate joints to meet the minimum thicknesses listed above. If you're willing to spend a little more time looking at grips and placement and number of washers, you may be able to use lesser ply thicknesses and the information in the above referenced paper.

For routine joint designs, I'd use strength values assuming the threads will be included in the shear plane (N). With bolt bearing, block shear rupture and shear rupture as limit states in bolted joints, you often won't get to the bolt shear strength for threads excluded (X) before something else controls.

A final related, but separate item: ANSI/ASME B18.2.6 now contains all threading and dimensional standards relating to high-strength structural bolting products, including those for ASTM A325 and A490 bolts. The latest versions of ASTM A325 and A490, however, still reference the older standard ANSI/ASME B18.2.1, which covers many other applications than just high-strength bolts. With the variety of threading options covered in ANSI/ASME B18.2.1, there has been some confusion about what specific threading requirements must be met for ASTM A325 and A490 bolts. Fortunately, bolt manufacturers know what they are supposed to be doing and produce ASTM A325 and A490 high-strength bolts as they should—in accordance with ANSI/ASME B18.2.6. This inconsistency in referencing will surely be eliminated in the next available revisions of ASTM A325 and A490, as well as the AISC Specification and Manual.

Charles J. Carter, S.E., P.E.
American Institute of Steel Construction
Chicago, IL

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Question from February 2000:

Girts are typically designed to support the vertical tributary area weight of siding for each girt level as well as the horizontal (component and cladding) tributary area wind pressure for each girt member.

Considering that siding is necessarily erected from the base upward and that the diaphragm arching effect of the siding would certainly bridge between columns and load them directly, why does it make any sense to consider channel girts to eccentrically support siding weight on one flange? Suppose no sag rods are used?

James G. Brooks, P.E.
OnBoard Engineering
Newark, DE

The load components on girts in the vertical direction consist of the dead load of the girt and siding. The dead load component of the girt passes through its centroid. However, the siding weight component does not pass through the centroid of the girts since the siding is attached to the outside flange. As a result, the outside flanges of the girts are eccentrically loaded by the siding.

With this in mind, most of the channel girt's section offers little resistance to bending about its weak axis, therefore the moment of inertia of the outside flange alone (approximately one half total I_y) is used to compute stresses for weak axis bending. Since the sag rods only provide lateral support for the flange in the weak axis, they may be used to determine the unbraced length of the girts for weak-axis bending. In the absence of sag rods, the distance between columns becomes the girt's unbraced length for bending in the weak axis.

Sam Babatunde, P.E.
Engineering Dynamics & Associates
Edgewood, PA

Via email:

Table 9-3 and its accompanying paragraph (page 9-16) in the LRFD *Manual of Steel Construction*, 2^{nd} ed., Volume II, requires that for knife connections the leg of the clip angle welded to the supporting member be 4" (with reduction to 3 1/2" or 3" in some cases). If the connection is to be used on a column with a flange width of less than 8", can the clip be reduced to have a 2 1/2" leg welded to the supporting member without affecting its rated capacity in Table 9-3? The tables in the CISC *Handbook* are based on similar size angles.

Lorne Hyatt
Lambton Metal Works Ltd.
Ontario, Canada

The strength of the connection would not change with a reduced angle leg dimension, but the flexibility of the connection will change. The double angle connection is supposed to accommodate the end-rotation of the simple beam through flexing of the angle legs. The smaller you make the angle leg, the thinner you'll have to make the angles to ensure flexibility.

I suggest you confirm with the engineer that the combination of reduced angle leg dimension and thickness will be acceptable for this case. If you or they want to put some numbers on the acceptability of the combination, take a look at the calculations that are outlined for tee connections on page 9-170 of the AISC LRFD *Manual*, 2^{nd} ed., Volume II. Those same calculations can be applied to double angles to determine if the combination of thickness and angle leg dimension is acceptable.

Charles J. Carter, S.E., P.E.
American Institute of Steel Construction
Chicago, IL