

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.

Bolt Spacing for Prying Action

All of the equations for prying action include the symbol p (the spacing between bolt rows). However, if we have only one bolt row in a connection, there is no spacing between rows. If $p = 0$, it seems to invalidate equations related to calculation of prying action. Are there alternative equations when $p = 0$?

Think of p as the tributary width per pair of bolts, which is the definition given on Page 9-11 of the 13th Edition AISC *Steel Construction Manual*. If you have only one pair of bolts (or one bolt in the case of a single angle as shown in Figure 9-4b), the tee or angle length along its axis can be used. Note that the definition of p says to preferably not exceed the gage, though, so if the tee or angle is long, g should be used as the upper limit on p .

Brad Davis, Ph.D., S.E.

Shear Center

How do I calculate the shear center of a rolled shape?

The shear center of an open cross section is the point where the resultant moment of external forces and internal stresses in the plane of the section must be zero. For most common rolled shapes, the location of the shear center is illustrated in AISC Design Guide No. 9 *Torsional Analysis of Structural Steel Members* (a free download for AISC members at www.aisc.org/epubs). See Figure 2.1. For symmetric shapes, the shear center is on the line of symmetry. For doubly symmetric shapes, such as W-shapes, the shear center is at the centroid. For channels, see Figure 3.1 and the associated equations for e_0 . For angles and tees, the shear center is at the intersection of the midlines of the elements of the cross-section.

If you are looking for the general location of the shear center with respect to the geometric axes for other shapes, I suggest looking at the information in Design Guide No. 9 or a structural steel design textbook.

Kurt Gustafson, S.E., P.E.

Section Modulus Relative to Angle Leg Toe in Compression

How does one calculate S_c for Section F.3 in AISC 360-05? What does "elastic section modulus to the toe in compression relative to the axis of bending" mean? What if the entire angle leg is in compression? When one has an equal-leg angle with no lateral-torsional restraint bent about a geometric axis, $S_c = 0.8S_x$, what if the angle does not have equal legs?

S_x refers to the section modulus of the entire shape taken about the axis of bending, with respect to the extreme fiber that is in compression. For example, the value of S_x published for angles in Table 1-7 of the Manual represents the smaller S_x for the shape. If the leg tip closest to the x-axis is in compression, the value of S_{xc} would be larger.

If the entire leg of the angle is in compression, the toe of that leg is also in compression. That would be the leg in compression due to flexure about the geometric axis of an equal leg angle. If the angle has unequal legs, the 0.8 simplification method does not apply; the provisions for principal-axis bending would apply instead.

Kurt Gustafson, S.E., P.E.

Edge Distances for Single-Plate Shear Connections

In the October 2006 issue of *Modern Steel Construction*, a range for the support line to bolt line (a -distance) is given as 2½ in. to 3½ in. for single-plate shear connections. Does this necessarily mean $a = 2$ would not work properly? If the connection were a through plate passing through a 6-in. column could $a = 2$ in. be used?

It is not prohibited to use $a = 2$ in. with the procedure in the 13th Edition AISC *Steel Construction Manual*. However, 2½ in. to 3½ in. is the typical range. There are a few dimensional limits to consider, though, depending on whether you're using the "conventional configuration" or the "extended configuration" (which covers anything that's not a conventional configuration).

Conventional Single-Plate Shear Connections:

Dimensional limit #2 requires that a must not exceed 3½ in., so $a = 2$ in. satisfies this.

Dimensional limit #4 might cause problems with $a = 2$ in. though. The horizontal edge distance from the center of bolt hole to end of steel beam must be equal to or greater than twice the bolt diameter. If your beam has a ½-in. setback from the face of support, the horizontal edge distance is only 1½ in. Granted, if you're using typical ¾-in. bolts, $2(¾ \text{ in.}) = 1½ \text{ in.}$, but don't forget about the beam length tolerance here. If the beam is cut just a little short, then the edge distance will be less than 1½ in. This matters because this connection gains its rotational ductility from the bolt causing bearing deformation at the side of the hole. If it's too close to the end, it might tear-out the end of the beam web. Note that the developers of these provisions felt the necessity to impose a requirement for $2d_b$ edge distance, which exceeds the requirements in AISC Specification Table J3.4. If your connection satisfies this limit somehow, it can be designed using the conventional configuration procedure. If not, then perhaps it can be treated using the extended configuration procedure.

Extended Single-Plate Shear Connections:

Dimensional limit #4 is easier for extended shear tabs. For ¾-in. bolts in standard holes, the minimum edge distance is 1¼ in. at a sheared edge. Assuming a ¼-in. beam length tolerance, this limit is still satisfied.

Design check #2 might cause problems, though. This is the method for ensuring rotational ductility for in the extended configuration; the plate must be able to yield in flexure at a lower moment than will cause bolt shear failure in pure moment. The exceptions below this design check are probably not useful in this case because they are based on bearing deformation as in the conventional procedure.

Brad Davis, Ph.D., S.E.

Use of the Overstrength Factor Ω_0

How do I use the system overstrength factor Ω_0 . Is it used as a multiplier on the load side or the strength side of the equation?

The system overstrength factor Ω_0 is used as a multiplier on the load side of the equation. For example, in a braced-frame system, it is applied as a multiplier on the axial load in the brace. The resulting amplified force is used to compare to each applicable limit state.

Kurt Gustafson, S.E., P.E.

steel interchange

Evaluating Existing Bracing Connections

We are modifying an existing structure that has wide-flange vertical bracing members that were designed using the 7th edition AISC Manual. The connections used friction-type bolted joints with oversize holes in web plate connections to the gusset on both edges. The AISC Specification requirements applicable to the 7th Edition did not require friction bolts to be checked for bearing, as is required today.

In general, the connections work for the slip-critical portion of the calculation, but many fail the bearing strength check. In order to prevent the connections from slipping into bearing, I've considered adding welds to the joint. Do you have any suggestions as to how to evaluate these connections?

You are correct that these connections should be checked for bearing as we do today. Where the bearing limit state will not satisfy the design loads, something should be done to ensure that slip into bearing does not take place. The welding approach you have suggested seems like a good option, and I would design the welds to resist the full design load, neglecting the bolts entirely. This eliminates the difficulty of addressing the differing stiffness and ductility of the welds and bolts were they to be used to share the load.

Larry S. Muir, P.E.

Beam Bracing

What constitutes a lateral brace for a beam? Does properly attached roof deck act as a continuous brace for the compression flange?

Beam bracing can be lateral, torsional, or a combination of these. Accordingly, a brace for a beam must restrain lateral movement of the compression flange (lateral brace), twist of the entire section (torsional brace), or a combination of these. Lateral braces are covered in Appendix 6.3.1 of the 2005 AISC Specification, and torsional braces are covered in Appendix 6.3.2. These sections address both the strength and stiffness requirements that must be met to consider a point braced.

This information also can be used to determine the answer to your second question. For the typical case, it is easy to see that roof deck spanning perpendicular to the beam and attached to the compression flange with typical deck welds is strong and stiff enough to be considered a lateral brace. If you have an atypical case, such as a heavy beam with large loads combined with a small deck, or long-span deck, you should evaluate it using the strength and stiffness equations in Appendix 6.

Note that if you have an uplift case, the deck won't count as a lateral brace because it is attached to the tension flange. It may provide enough restraint to be considered a torsional brace, however. Another case to evaluate further is that of deck spanning parallel to the beam.

Brad Davis, Ph.D., S.E.

Conventional Configuration Single-Plate Shear Connections

Design limitations for conventional configuration single-plate shear connections imply that long-slotted holes are not permitted. Why are these not permitted? Also, for the extended con-

figuration, are long-slotted holes permitted? The limitations for the extended configuration refer to AISC Specification Section J3.2 requirements, which imply that long-slotted holes may be used. If these are permitted, would they need to be slip critical?

Yes, the procedure presented in the Manual for the conventional configuration of single-plate shear connections is specific to the use of standard or short-slotted holes. This procedure is based upon testing to define a simplified approach that can be used for the majority of cases. No tests were run on connections with long slots, so I would not apply the procedure to connections with long slots.

Long-slotted holes can be used with the extended configuration procedure, however. When using long slots, I would design the connection as slip-critical. I do not think the eccentrically loaded bolt group *C*-values in the Manual can be obtained in bearing-type connections using long slots.

Larry S. Muir, P.E.

Delayed Steel Erection

A delivery on a job has been delayed by eight months. Because the steel is to be fireproofed, it is not painted and has rusted. The owner has expressed concerns about the steel and the effects of the rust. Would this affect the steel as far as strength?

If there is a loss of section caused by rusting, the strength of the member could be compromised. However, except for extremely corrosive environments, deterioration due to rusting is a very long-term process, and I would be surprised if this is a factor for an 8-month exposure.

The effect of the rust on the bonding of the fireproofing might be more of a consideration. FAQ 11.1.1 on the website at www.aisc.org/faq addresses the subject of surface preparation for application of fireproofing. If the rust bloom is tightly adherent, it may not affect the adhesion. I suggest that you should check with the fireproofing applicator to review the conditions for suitability of their product application.

Kurt Gustafson, S.E., P.E.

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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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