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.

Effective Length of Vertical Braces

We are having a discussion in our office about the design of a compression diagonal angle in an "X" braced frame. Some engineers believe the unbraced length for compression design is reduced based on the fact the other member in the brace is in tension and will prevent the compression angle from buckling. Is this a valid assumption? Is there any information that would support this design assumption?

It is possible that the tension brace can brace the compression brace at its midpoint. The answer depends on how much lateral stiffness is provided by the tension brace. Among other things this depends on which member is continuous and what assumptions are made about "catenary" stiffness contributions. Appendix 6 of the 2005 AISC *Specification* provides strength and stiffness requirements for column braces, which may be a starting point.

Energy methods are often used for evaluating such problems. An excellent AISC *Engineering Journal* paper by R. Shankar Nair can be found in the 4th Quarter 1997 journal at **www.aisc.org/epubs** and addresses this exact problem.

Amanuel Gebremeskel, P.E.

Specifying End Reactions

Non-composite beam end shear connections for wide-flange beams to columns are typically specified based on 50% of the total uniform load table capacity of the beam. These requirements are given to the steel detailer to develop/detail the connections. Is there a similar table for HSS beam end connection design; or a way to specify end connections for the steel detailer to detail, etc.? The AISC 13th edition *Steel Construction Manual* has Table 3-12 "available flexural strength, kip-ft" for rectangular HSS. Can end beam reactions be tabulated from this table?

I would not agree that specifying end connections at 50% of the total uniform load capacity of the beam is the "typical" method of relaying connection design parameters to the fabricator. It may be used in some cases where the EOR does not want to provide accurate reaction forces, but it is certainly not the preferred method to get the most economical bid. Providing the actual reactions is much more common, and appropriate, today.

HSS shapes are not commonly used as beams, and thus there are no uniform load tables for such in the AISC 13th edition *Steel Construction Manual*. The EOR can employ any method they wish to communicate design information, but it should be based on engineering logic and is most efficient if developed in a useful manner. One could develop a method of equating the available flexural strength back to a reaction value, but it would seem to be more straightforward to simply list the reactions.

Kurt Gustafson S.E., P.E

Beam/Column Connection with Axial Compression

I have a beam connected to a column flange using clip angles. The beam has quite a big compression load. Do I need to check column flange bending or anything else? The AISC *Manual* states that a flange local bending check is required for tensile forces only, but one of my coworkers told me I'd need to check column flange bending for the connection. Could you give me some advice?

There is no need to check flange bending for a compression load on a column flange. The compression load will not transfer through the bolts. Instead, it will transfer through the much stiffer load path provided by direct bearing between the angle legs connected to the supported beam and the column itself. Since it is likely that the column web (plus the k1 distance) is close to the same width as, if not wider than, the distance between the angles, no bending in the column flange will occur.

This said, I would be inclined to use an end plate rather than the double-angle connection for the condition you describe. The end plate will provide more bearing area. Additionally, if the beam end is cut and fitted so that it bears on the end plate, the welds between the end plate and the beam need not resist the compression load. It would likely be a more cost-effective connection.

Larry S. Muir, P.E.

Single Angle in Flexure

The 2005 AISC *Specification* has new single-angle bending equations. I have also looked at the design example provided by AISC on CD. When do you use geometric bending without lateral support as opposed to principal-axis bending? It seems to me that if there is no lateral support, you should use principal-axis bending, but that is not how AISC arranges the section. In the example, the single angle supporting a uniform load is only supported at the ends. This would allow the angle to deflect laterally and vertically, which would indicate bending about the principal axis. Should I use the geometric axis for design or should I use the principal axis for design?

The AISC *Specification* provides a simplified alternative where you can use modified geometric properties when analyzing an unrestrained equal-leg angle that is loaded in bending about the geometric axis. This is provided in section F10.2(i). In F10.2(ii), another simplified case is covered with restraint at the ends and the point of maximum moment. For all other cases the more general solution based upon principal-axis properties must be used. The general solution and simplified special cases all address the unsymmetrical behavior that single angles exhibit when subjected to bending.

Amanuel Gebremeskel, P.E.

steel interchange

Extended Single-Plate Shear Connections

In the procedure for the extended configuration of the single-plate shear connection, the bolts are designed for the eccentricity of the connection, and it says that the column does not need to be designed for a bending moment for the eccentricity. The first question I have is, according to these assumptions, the weld to the column will be only in shear. Why then is it taken as $\frac{5}{8} * t_p$, instead of designing it for the shear at the support? This produces some huge welds for the plates that are necessary for bending in some cases. The second question is, how is the rotation of a simple shear connection achieved when the eccentricity is in the bolts and the weld is only in shear? I am thinking of a 7- or 8-row connection where it is obvious that no rotation will be allowed at the weld.

Extended and conventional single-plate connections are designed in a significantly different way than other shear connections. In most other shear connections (single and double angles, end plates, seats) the inherent flexibility of the connecting elements (angles or plates) is relied upon to accommodate the simple beam end rotation. However, there are still limits related to bolt size, weld size, and plate or angle thickness to ensure ductility.

Obviously, the single-plate connections are much stiffer than the other connection types and require a different design philosophy. You state correctly that the bolts are designed for the full eccentricity between the bolts and the welds. You also state correctly that the welds are sized to be ⅝ of the plate thickness to develop the strength of the plate. Furthermore, the *Manual* states that the column need not be designed for an additional eccentricity. This seems to be a contradiction in that if the full eccentricity is present at the bolts, then none will exist at the weld and vice-versa.

The problem is that we do not know either the location or the magnitude of the moments within the connection. If the plate is welded to a very stiff column, then certainly the connection to the column will draw most of the moment to it. However, if we connect to a very flexible support—let's say a torsionally weak girder—then most of the moment will be drawn towards the much stiffer bolted connection. There is an infinite variation between these extremes. What we have tried to do with the extended tab procedure is accommodate all of the variations with a universally applicable model. The plate between the weld and the bolts is used as a fuse. Under extreme loads, which could develop with a very stiff support (approaching the fixed-end beam condition), the plate is allowed to yield and shed load. In order to accomplish this, the bolts and welds cannot be allowed to fracture prior to the plate yielding. This is the reason for the $\frac{5}{8} t_p$ requirement for the welds and the maximum plate thickness check related to the bolts.

This mechanism is also how the simple beam end rotation is accommodated. The plate can yield prior to rupture of either the bolts or the welds. There is an additional provision that allows the rotation and ductility to be met by plowing of the bolts through the material.

The column is not required to be designed for an additional moment for two reasons. The first is strictly based on precedence. Typically, in other shear connections the eccentricity from the face of the support to the center of the support is neglected. This has never been a problem to my knowledge, and it is allowed for single-plate connections as well.

The second reason is that though the single plate connection may add additional moment to the column due to its rotational stiffness, this same stiffness also adds restraint, which is usually not accounted for in the analysis.

Larry S. Muir, P.E.

Bracing for Cantilever Beams

Must braces be added to the bottom flange (compression) of cantilevered steel beams? After reading the 1999 *LRFD Specification*, Section C4a, it seems like I should brace the tension flange of the cantilevered portion instead. Is this correct?

It is true that adding tension (top) flange bracing on a cantilever is more effective than bracing the bottom flange. This is because it is the top flange that deforms more in the deflected shape. There is a good discussion on this subject in the *Guide to Stability Design Criteria for Metal Structures* by Ted Galambos. You will also find a table therein that defines effective-length factors for cantilevers based on the restraint provided at the base and tip of the cantilever.

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

The complete collection of Steel Interchange questions and answers is available online. Find questions and answers related to just about any topic by using our full-text search capability. Visit Steel Interchange online at **www.modernsteel.com**.

Kurt Gustafson is the director of technical assistance and Amanuel Gebremeskel is a senior engineer in AISC's Steel Solutions Center. Charlie Carter is an AISC vice president and the chief structural engineer.

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:

Steel Solutions Center
One East Wacker Dr., Suite 700
Chicago, IL 60601
tel: 866.ASK.AISC • fax: 312.803.4709
solutions@aisc.org