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steel interchange

Prying Action – A Wider View

Are the prying action checks in Part 9 of the AISC *Steel Construction Manual* only applicable to double-angle and tee connections? Can they be applied to other conditions like angles attached only to the outside wall of an HSS or a perforated, end-plate splice used to join segments of large diameter HSS?

It all depends on the details. Thornton (1992 and 1996) provides background on the prying action equations presented in the *Manual*. It is important to understand the assumptions made when employing any engineering check, and especially this one. And while these papers and Swanson (2002), which is also referenced in the *Manual*, assume that the load is delivered by a tee, the checks can be applied to similar situations.

In Figure 9-4 of the *Manual*, the dimensions *b* and *b*' are measured from the face of the tee stem or the center of the angle leg. These values are valid for tees and for angles if the load, 2*T*, is delivered symmetrically and the angle shown represents one of a pair of back-to-back angles. It is reasonable to assume that the increase in b and b' for the double-angle connection is due to the reduced stiffness of the angles as opposed to the tee. When the angles are not back-to-back and connected to a relatively flexible support, the effective eccentricity may be increased and a distance measured to the heel of the angle, or possibly somewhat greater, might be warranted.

When the load is delivered asymmetrically, an even greater moment might result. For instance, if the angle were attached to only one flange of a wide-flange member used as a hanger and the hanger were not restrained from rotating about the bolt line, then eccentricity would have to be measured from the centerline of the hanger or to the point of application of the load. The prying action discussion in Part 9 of the *Manual* is not intended to be applied to asymmetrical conditions.

There are other references to which you can turn for special cases. One-sided flanges are commonly used for connections in steel stacks, wind turbines, bins, hoppers, transmission poles and other plate and shell structures. The simplest approach to the flange design is in the CICIND (2005) chimney book. Using the terminology in Figure 9-4b on Page 9-11 of the 14th Edition AISC *Steel Construction Manual*, the total force on the bolt, including the prying force, is calculated based on equilibrium of the flange:

$$T + q = \frac{T(b + a)}{a}$$

Theoretically, the maximum moment in the fitting is at the center of the bolt. The CICIND method neglects the reduction in bending strength due to the presence of the bolt hole, which gives the following equation for the nominal strength (that is, with no ASD safety factor or LRFD phi factor applied):

$$T = \frac{F_y t^2 p}{4b}$$

Adding these and also reducing the bending strength for the presence of the bolt hole, the available strength of the fitting is:

LRFD	ASD
$T = \dfrac{\phi F_y t^2 (p - d')}{4b}$	$T = \dfrac{F_y t^2 (p - d')}{4b\Omega}$

All of these equations assume there is no moment transfer between the flange and the structure to which it is attached. That is, they assume all of the moment required for equilibrium of the flange is taken at the bolt line.

The engineer must also choose an effective length over which to assume the bending occurs. The discussion in the AISC *Manual* recommends a 45° spread and the resulting tributary length of 2b, but not exceeding the spacing between the bolts. This is a conservative simplification, and Dowswell (2011) and Wheeler et al (1998) provide alternative approaches that are less conservative.

Another choice must also be made between the use of the yield stress or the tensile stress of the flange. The use of the tensile stress in the *Manual* is based on empirical results and produces a prying action calculation that better matches the results of tested connections. It also likely reflects contributions from strain-hardening that occurs in the flange as it is bent about its weak axis.

There are other special cases addressed in the literature as well. For example, prying action for two-way bending is treated for stiffened end-plates in AISC Design Guides 4 and 16 (Murray 2002 and 2003).

References and Resources

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Question and answer compiled from questions addressed by Larry S. Muir, P.E., and Bo Dowswell, P.E., Ph.D.

Design of Continuous Gusset

Figure 5-43 of the AISC *Seismic Design Manual* shows a connection with a continuous gusset plate. There are not calculations as to how the ¾ in. plates and welds were sized. Is there a design example available that provides guidance as to how to design this type of continuous gusset?

I am not aware of any published design examples for this case. It is common to use the uniform force method and model the connection as two separate gussets and as if a column web were present between them. However, any other suitable model that satisfies equilibrium should also be acceptable. If modeled as a single, continuous gusset, other additional free-body diagrams beyond that cut at the "column"-to-gusset interface may be required to establish the required gusset thickness. For example, cuts near the elevations of the beam flanges would be a logical location.

Larry S. Muir, P.E.

Deflection of Crane Supports

Several crane systems are suspended from roof trusses with a 200-ft span. The truss deflections under dead load are causing issues with the crane. Are AISC deflection requirements sufficient or should some other standard govern?

The AISC requirements are sufficient and must be met. However, though the AISC *Specification for Structural Steel Buildings* (a free download at **www.aisc.org/2010spec**) requires that deflections must be considered, explicit deflection criteria are not provided because the appropriate criteria will vary by system and application. Deflection requirements are addressed in Chapter L of the AISC *Specification*, which states: "Deflections in structural members and structural systems under appropriate service load combinations shall not impair the serviceability of the structure." The Commentary states: "Deflection limits depend very much on the function of the structure and the nature of the supported construction."

The fact that the truss deflections are causing problems with the crane is an indication that the intent of Chapter L has not been satisfied.

AISC Design Guide 7 (a free download for members at **www.aisc.org/dg**) states: "Crane runway fabrication and erection tolerances should be addressed in the project specifications because standard tolerances used in steel frameworks for buildings are not tight enough for buildings with cranes. Also, some of the required tolerances are not addressed in standard specification."

While not specifically addressing your particular situation, Commentary Section 7.13 in the AISC *Code of Standard Practice* states: "The effects of the deflection of transfer girders and trusses on the position of columns and hangers supported from them may be a consideration in design and construction. As in the case of differential column shortening, the deflection of these supporting members during and after construction will affect the position and alignment of the framing tributary to these transfer members."

Carlo Lini, P.E.

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Larry Muir is director of technical assistance and Carlo Lini is a staff engineer–technical assistance, both with AISC. Bo Dowswell is a consultant to AISC.

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:

1 E Wacker Dr., Ste. 700, Chicago, IL 60601
tel: 866.ASK.AISC • fax: 312.803.4709
solutions@aisc.org

Steel SolutionsCenter