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!

Tightening Tension Control Bolts

On one of our projects, the EOR specified snug-tightened bolts for the majority of the connections. The erector chose to use twist-off bolts for the entire job. He snapped off all the splines without telling the EOR. As the inspector, should I approve them?

Question sent to AISC's Steel Solutions Center

There is no concern for the level of pretension that may be present in a snug-tightened joint. Said another way, snug-tight is a minimum condition with no maximum necessary. Accordingly, the RCSC *Specification* allows the use of tension control bolts in a snug-tight joint. To do so does not affect the structural integrity of the connection. It is just as easy for the erector to break the splines as to leave them in place, and it gives both the installer and inspector a way of tracking the completion.

Bill Liddy
American Institute of Steel Construction, Inc.

Expansion Joints Required for Temperature

I am designing a steel structure with 960′ x 450′ plan dimensions. Where can I find the design guideline for setting and designing expansion joints? Is it possible to design the structure without expansion joints if the architect does not want the joints?

Question sent to AISC's Steel Solutions Center

The subject of expansion joints for structures is briefly discussed in the AISC *LRFD Manual of Steel Construction*, 3rd Edition, starting on page two through 48. If you require additional subject information, the referenced basis of this discussion is a 1974 publication titled *Expansion Joints in Buildings*. This document was prepared by the Standing Committee on Structural Engineering of the Federal Construction Council, Building Research Advisory Board, Division of Engineering, National Research Council as *Technical Report No. 65*. This document is available for purchase from the following web link: **www.nap.edu/catalog/9801.html**.

Kurt Gustafson, S.E., P.E.
American Institute of Steel Construction, Inc.

Minimum Flange Thickness for Welding Studs

I am looking for information on what the minimum flange thickness requirements are for welding shear studs for composite action. Because I do not know what the actual weld size is, I cannot determine what the minimum flange thickness should be. We do specify 1/4″ minimum.

Question sent to AISC's Steel Solutions Center

The AISC *Specification* does not address shear stud weldment requirements. However, the required base metal thickness is specifically addressed in Section 7.2.7 of AWS D1.1:2004. "When welding directly to base metal, the base metal shall be no thinner than 1/3 the stud diameter." There are different requirements when welding through metal deck. Refer to AWS D1.1 or contact AWS technical support for further information pertaining to AWS code parameters (web site at **www.aws.org**). Additional information on stud welding is available on the Nelson web site at the following link: **www.nelsonstudwelding.com**.

Sergio Zoruba, Ph.D.
American Institute of Steel Construction, Inc.

Design of Cantilever Beams

The following is a brief paper presenting one engineer's opinion on design parameters for use with cantilever beams. The author has included a list of pertinent reference sources on the subject.

Cantilever Flexural Member Design
By Sam Eskildsen, P.E., Structural Design Group, Birmingham, AL

Introduction

The AISC 1999 *Load and Resistance Factor Design Specification for Steel Buildings*[1] has no specific flexural design requirements for cantilever beams beyond requiring $C_b = 1$ when the free end is unbraced. A review of the literature on cantilever analysis reveals this minimal requirement may not be enough to steer the engineer from creating cantilever designs that, while meeting the letter of the specification, at times may be unconservative.

Nethercot[2,3] has done extensive research on cantilever analysis and design. His relevant findings are summarized in the *Guide to Stability Design Criteria for Metal Structures.*[4] Nethercot's approach is to use an effective length factor, designated K_c, to account for a variety of restraint and loading arrangements. K_c values can range from less than one to greater than one.

Based on Nethercot's work, one can modify the 1999 *LRFD Specification* equations as follows for use with cantilevered wide-flange beams:

Equation F1-6: $$L_r = \frac{r_y X_1}{K_c F_L}\sqrt{1+\sqrt{1+X_2 F_L^2}}$$

Equation F1-13: $$M_{cr} = C_b \frac{\pi}{K_c L_b}\sqrt{E I_y G J + \left(\frac{\pi E}{K_c L_b}\right)^2 I_y C_w}$$

where K_c is given by Figure 1 (right, adapted from Nethercot); and $C_b = 1$ for any cantilever section, regardless of bracing conditions.

Figure 1
K_c Values for Cantilevers

Restraint Continuous		K_c	
Cantilever Support	Tip	Top Flange Loading	All Other Cases
		1.4	0.8
		1.4	0.7
		0.6	0.6
		2.5	1.0
		2.5	0.9
		1.5	0.8
		7.5	3.0
		7.5	2.7
		4.5	2.4

Bracing Type and Location

Traditionally, engineers expect that bracing should be of the compression flange. However, in the case of the cantilever, it is the tension flange that deforms most during bucking. Bracing a cantilever beam's compression flange alone has almost no impact on the beam's stability.

Kitipornchai[5] studied efficiency of bracing types and location along the span. He found that for discrete lateral bracing the most effective location is the top (tension) flange and as close to the cantilever tip as practical. He also found that for cantilevers fully restrained at the support, lateral restraints placed within the first 40% of the span are practically useless. Until more research is done on multiple or continuous restraints along the cantilever length, it seems prudent to consider only the restraint provided at the support and the tip.

K_c and C_b

Figure 1, a modified form of tables given by Nethercot, identifies K_c values that were derived assuming $C_b = 1$. Nethercot has published work in which $K_c = 1$ and the C_b factor is used to adjust for restraint and loading conditions at the cantilever, but using an effective length factor seems more intuitive and $C_b = 1$ should be used when using the K_c values from Figure 1.[2,3,6]

Conclusion

Let's consider one case where this approach is relevant. An under-slung crane beam is projecting out from the side of a building. Inside the building, the beam is over a mechanical space and is supported by roof beams. The tip of the crane beam extends outside the building such that deliveries can be hoisted up and brought into the building. The hoisting device has wheels that run on the bottom flange of the beam, so no stiffeners or bracing can be provided to the bottom flange of the cantilever beam without interfering with the operation of the crane device. Further, the architect is adamant that you are not to provide braces to the flanges of the cantilever beam tip at the exterior of the building. Not only does he feel this will not look good, but he's also driven around and seen this condition without them. You don't want him going to another engineer because he might not come back.

In this case, you have loading that is not top flange loading, the tip is unbraced at the top and bottom, and the "root" of the cantilever beam is only braced at the top flange. A K_c value of 3.0 is selected, and the beam is designed using the formulas above.

There are several other methods out there.[8,9] Some of them are more exact. However, in each case the methods are situation specific, are a bit too complex to be used in an office setting, or just do not cover enough cases. The nice thing about Nethercot's work is that in a straightforward way it covers most possible restraint conditions and loading conditions, and it is conservative. (Note that built into the K_c values are the effects of skip loading, uniform loading, point loading, varying ratios of back span length to cantilever length, varying support conditions for the "far" end of the supported portion of the beam, etc. He just used the worst case).

As a final comment, the proposed method lends itself to expansion to cover other problematic areas. Tables similar to Figure 1 could be developed to aid in the design of continuous beams and laterally unsupported beams with varying end restraints.

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One East Wacker Dr., Suite 3100
Chicago, IL 60601
tel: 866.ASK.AISC
fax: 312.670.9032
solutions@aisc.org