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Stability of Beams During Erection

We are erecting a structural steel building with long, slender beams. The beams have significant camber. During erection the beams are acting more like open web joists than beams. As soon as a beam is released from the crane it bows out to the side, resulting in a need for temporary bracing to keep the beams "straight." The design engineer has confirmed that the beams are structurally adequate once the slab on metal deck is poured. Is there a way to anticipate such erection issues?

Page 37 of Design Guide 23 *Constructability of Structural Steel Buildings* (a free download for members from **www.aisc.org/dg**) provides guidance. It states: "Most girders, as designed, are stable only when their compression flange is laterally supported... As a rule of thumb, most girders with *l/b* less than 80 will be stable during erection; for values greater than 80, the erector should consider some form of temporary support during and/or after the lift. Note that this ratio is not a substitute for an engineering analysis." The presence of camber will also tend to make the beams less stable since it effectively raises the point of application of load.

Carlo Lini

Prequalified Moment Connections

I am designing a special moment frame (SMF). Do SMFs only allow the use of wide-flange columns oriented in the strong direction?

No, but there are no prequalified moment connections to the weak-axis of a column provided in AISC 358. However, AISC 358 does allow the use of several types of built-up sections. These include:

1) I-shaped welded columns that resemble standard rolled wide-flange shapes in cross section shape and profile.
2) Cruciform W-shape columns, fabricated by splitting a wide-flange section in half and welding the webs on either side of the web of an unsplit wide-flange section at its mid-depth to form a cruciform shape, each outstanding leg of which terminates in a rectangular flange.
3) Box columns, fabricated by welding four plates together to form a closed box-shaped cross section.
4) Boxed W-shape columns constructed by adding side plates to the sides of an I-shaped cross section.

Requirements for these built-up columns are given in Section 2.3.2b of the *Seismic Provisions*.

There is another option. Connections can be qualified through testing as described in AISC 341 Appendix S. Engineers usually use what already is prequalified in AISC 358 because qualifying tests can be expensive and time-consuming, especially considering that the configurations tested ultimately may not meet the requirements. Nonetheless, the option to test exists and can be used.

Carlo Lini

Fire Rating of Concrete-filled HSS Members

How is the fire resistance of a concrete-filled member determined? I wasn't able to find any technical information or listed UL ratings for this type of assembly.

The December 2002 *MSC* article "Structural Fire Protection" (available at **www.modernsteel.com**) states: "Concrete-filled HSS columns are another example of generic construction that is not listed in the UL directory. Article 5.2.3 of ASCE/SFPE 29-99 specifies how to determine the fire resistance of concrete-filled hollow steel columns. The relevant background information can be found in: V.K.R. Kodur, and D.H. MacKinnon, "Design of Concrete-Filled Hollow Structural Steel Columns for Fire Endurance," *Engineering Journal*, First Quarter, 2000, pp. 13-24."

AISC members can download *EJ* articles for free at **www.aisc.org/ej**.

Larry S. Muir, P.E.

Strength of Anchor Rods

Regarding the design of anchor rods (ASTM F1554), the tension and shear strengths provided in AISC *Specification* Section J3.6 is a function of F_u. However, ASTM F1554 lists a specific value of F_y and a range for F_u. Why is the strength based on the tensile strength and not the yield strength? And what F_u value should be used?

Threaded elements like threaded anchor rods will fail by rupture through the threads, unless they are upset to produce a larger diameter in the threaded region. Accordingly, the design strength is based on the tensile stress, not the yield stress.

F_u should be taken as the minimum specified value, just like when you check rupture limit states on ASTM A36 or A992 steel.

Larry S. Muir, P.E.

Weight of HSS Members

I understand that the design thickness of ASTM A500 HSS has been discounted by 7% to allow for manufacturing tolerances. Why weren't the weights reported in the AISC *Manual* decreased along with the strength-related properties?

The 7% reduction is taken to reflect the usual wall thickness condition based upon surveys of actual production of HSS. However, manufacturers are not required to produce HSS close to the lower tolerance allowed by ASTM A500. So in design, the conservative approach is to base the weight on the nominal thickness and the section properties on the reduced thickness.

Heath Mitchell

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Shear Lag

We have two pipes spliced end-to-end using knife plates in the shape of a cross (plates slotted into the pipe wall at each quadrant). This condition is not addressed in AISC *Specification* Table D3.1. The pipes are too large (outside diameter of 24 in.) to economically provide the length necessary to satisfy the l > 1.3D requirement in order to use U = 1.0. Relative to tensile rupture of the pipe sections, what is the appropriate shear lag factor, U?

You are correct: Table D3.1 does not address this condition, so you will have to rely on your own engineering judgment. Following is some guidance that may help as you do that.

Having two plates as opposed to just one (as shown in Case 5) should help reduce the effects of shear lag considerably. Using a length equal to 1.3D to get a U = 1.0 would be very conservative. Instead, you might consider an approach similar to the Whitmore check.

The circumference of a round HSS is πD. The Whitmore section for each of your two plates is $4L_w\tan(30°)$; that is, 2 times $2L_w\tan(30°)$ at each weld of the plate to the HSS. Setting the circumference equal to the Whitmore section width and solving for L_w, we get:

$$L_w = \frac{\pi D}{4n\tan(30°)}$$

where,

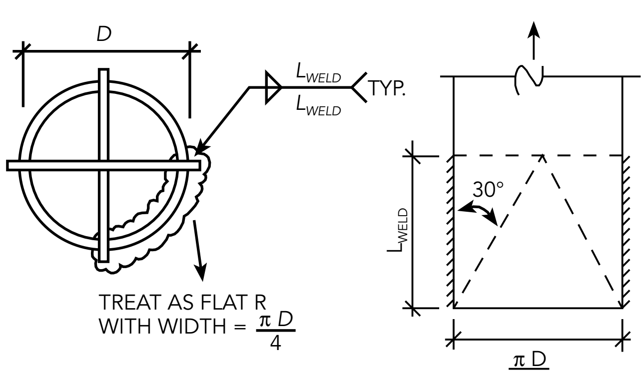
L_w = the length of the weld
D = the diameter of the round HSS or pipe
n = the number of splice plates

This approach compares favorably with Table D3.1 Case 5.

$$L_w = \frac{\pi D}{4(1)\tan(30°)} = 1.36D \text{ versus } 1.3D \text{ provided in Table D3.1.}$$

For the case with 2 plates we get:

$$L_w = \frac{\pi D}{4(2)\tan(30°)} = 0.68D\text{, or for the 24" pipe a weld about } 16\tfrac{3}{8}\text{" long.}$$



Carlo Lini

Reinforcing Fillet Over CJP Groove Weld

There are several prequalified CJP groove welds where the groove weld is reinforced with a fillet weld, shown in Table 8-2 of the AISC *Steel Construction Manual*. Does the reinforcing fillet weld add to the strength of the groove weld? What is the purpose of fillet welding over a groove weld?

Section 3.5 of AISC Design Guide 21, *Welded Connections—A Primer for Engineers* (a free download for members from **www.aisc.org/dg**) states: "Fillet welds can be applied to T- and lap joints and to the inside corner of corner joints. Fillet welds may be used to add strength to PJP groove welds and may be used to provide for a more gradual contour to CJP groove welds in T- and corner joints. When used in conjunction with CJP groove welds, the strength of the fillet weld is not added to that of the CJP groove weld."

The Commentary to Appendix 3 in the AISC *Specification* (a free download at **www.aisc.org/2010spec**) gives some insight into the reason a contouring fillet weld might be applied: "The addition of contoured fillet welds at transverse complete-joint-penetration groove welds in T- and corner joints and at reentrant corners reduces the stress concentration and improves fatigue resistance." The improvement applies to both low-stress/high-cycle behavior (fatigue) and high-stress/low-cycle (seismic) behavior.

Clause 2.16.3 of AWS D1.1 requires contouring fillet welds at corner and T-joints subject to tension, and therefore so does the AISC *Specification* as no exception to this clause is taken in AISC 360 Section J2.

Larry S. Muir, P.E.

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Larry Muir is director of technical assistance and Carlo Lini is staff engineer–technical assistance at AISC. Heath Mitchell is a consultant to AISC.

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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:

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

Steel SolutionsCenter