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steel interchange

Use of the Inflection Point as a Brace Point

I was told that the use of an inflection point to brace the compression flange of a beam is no longer allowed. Is this true?

Yes. The 1999 AISC LRFD *Specification* was the first edition to introduce strength and stiffness provisions, in Section C3, for beam and column braces. The language prohibiting the use of inflection points as brace points for beams is in Section C3.4 in that *Specification*. These requirements were moved to Appendix 6 in the 2005 and 2010 AISC *Specification*. The language on inflection points can be found in Appendix 6 Section 6.3 (page 16.1-193 of the 13th Edition *Manual*, and similarly on page 16.1-229 in the 14th Edition *Manual*.)

While we are aware that some engineers assumed unbraced lengths based on inflection point locations, this practice was never specifically permitted by the AISC *Specification*. For example, note that the 1989 *Specification* states in Chapter F, "braced laterally… at intervals not exceeding…" and "the laterally unsupported length of the compression flange does not exceed…" An inflection point does not qualify as either. Similar language can be found as well in older versions of the AISC *Specification*.

The proper way to account for an inflection point (moment gradient) is through use of C_b. The 2010 AISC *Specification* provides alternative equations for the calculation of C_b in the Commentary to Chapter F to assist in this exact case. One is applicable to beams bent in reverse curvature. There are also citations to references that contain C_b equations for other loading and support conditions not covered in the Commentary to Chapter F. The 2010 AISC *Specification* is available as a free download at **www.aisc.org/2010spec**.

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

Weld Access Hole Height

In ANSI/AISC 360-05 Section J1.6, the height of a weld access hole is required to be at least 1 in. (25 mm). The commentary to this section states, "The height of the weld access hole must provide sufficient clearance for ease of welding and inspection and must be large enough to allow the welder to deposit sound weld metal through and beyond the web." It seems one could satisfy this intent using an access hole with a height less than 1 in. Is there any provision that would allow use of a lesser access hole height?

Yes, based on revisions made in the 2010 edition of ANSI/AISC 360. Section J1.6 currently requires the weld access hole height to be a minimum of 1.0 times the thickness of the web, or ¾ in. A dimension less than ¾ in. was considered to be too small for the reasons listed in the J1.6 commentary.

Keith Landwehr

Beveled Gusset Corners

What are the benefits of "snipping" or bevel cutting the edge of a vertical bracing gusset where it lies under the bracing member?

Historically, not snipping the corner would save one step during fabrication and therefore the additional cut was often avoided. However, the snipping would often still be done if the sharp corner would be exposed once the brace was in place. This was done for safety, to avoid injuries that might occur if someone walked into the sharp corner.

As more fabricators use automated burning tables, avoiding the extra cut is less important, and a more important consideration may be that the gussets might be more tightly nested with the corner removed thereby allowing more gussets to be cut from a single plate with less waste.

There are no strength considerations that would be used to determine whether the corner should be snipped or not and the AISC *Specification* contains no requirements related to this topic, so it is really a matter of preference and judgment.

Larry S. Muir, P.E.

Moment Connection to HSS Column

I am working on the design of a moment connection between a wide-flange beam and an HSS column. The beam flange is wider than the HSS column it connects to. According to ANSI/AISC 360-10 Section K1.3b, B_p/B must be less than or equal to 1. Do we need to taper the flanges of the beam to be the same width of the column at the joint or can we keep the normal flange width with no taper and use $B_p/B = 1.0$?

Assuming you do not have concerns about fatigue, there is no need to taper the flange. The flange width should be assumed equal to the width of the HSS for calculation purposes. In ANSI/AISC 360-10 Chapter K, beta then will be equal to 1.0.

Chapter 4 in AISC *Steel Design Guide No. 24, Hollow Structural Sections*, provides guidance related to these connections. Example 4.3 addresses the directly welded connection and treats the flange as a transverse plate. However, this example is configured such that the beam flange is narrower than the HSS width.

For this type of connection with a beam flange width greater than or equal to the HSS column width, the applicable checks will be Equations K1-7, K1-9 and K1-10 or K1-11. Equations K1-9, K1-10 and K1-11 are similar to the local web yielding and crippling checks for wide-flange beams in Section J10. Equation K1-7 incorporates an effective width concept. If a CJP groove weld between the flange and the HSS wall is not used, this effective width concept should also be incorporated into the design of the weld, as shown in Equation K4-4.

Fatigue applications may require tapering.

Larry S. Muir, P.E.

Fillet Weld Design

Per ANSI/AISC 360 Table J2.5, the base metal shear strength for a fillet weld is governed by Section J4. Is the gross area of the base metal subject to shear based on the thickness of the base metal at the weld or the size of the weld leg? In other words, does the failure occur through the thickness of the base metal or at the fusion zone between the fillet weld leg and the base metal?

The strength of a fillet weld will be governed by the strength of the weld itself on its effective throat. The base metal also is checked, not on the fusion area, but rather through the thickness of the connected part. This is reflected in the design examples that accompany the AISC *Steel Construction Manual* and is also illustrated in ANSI/AISC 360 Figure C-J2.10.

This is based on historic and recent testing that shows that the fracture occurs through either the weld on the effective throat or through the thickness of the base metal, not through the fusion zone. Most recently, in a Closure to a discussion on the paper, "Design and Behavior of Multi-Orientation Fillet Weld Connections," the authors stated, "…a check of the weld throat and of the shear plane in the connecting material is sufficient; the fusion zone does not need to be checked." The discussion is in the 1st Quarter 2011 *Engineering Journal*, available online at **www.aisc.org/ej**.

Larry S. Muir, P.E.

CJP Groove Weld for Round HSS

I need to make a CJP butt-welded splice in a round HSS. I had planned on using a smaller pipe as a backing bar but cannot find any guidance concerning an allowable gap for the backing bar. How much gap is allowed between base metal and the backing bar for a CJP butt weld?

Paraphrasing AWS D1.1 clause 5.22.1.1 titled "Faying Surface," the separation between surfaces, including butt joints, shall not exceed 1⁄16 in. In your case, this would require that you use a "backing pipe" with an outside diameter that is 1⁄8-in. or less smaller than the inside diameter of the pipe butt joint being welded. If you can't find standard sizes that satisfy this fit-up, you may need to have your backing machined to fit.

Keith Landwehr

Built-Up Shape Tolerances

Does AISC specify fabrication tolerances for welded built-up I shapes?

AISC specifications and codes do not contain fabrication tolerances for welded built-up I-shaped members. However, there are such tolerances in AWS D1.1. Clause 5.23 contains tolerances for girder depth, web flatness, flange tilt, and camber, among others.

Keith Landwehr

SCBF Brace Slenderness

I noticed that AISC 341-10 Section F2.5b specifies that the slenderness of diagonal braces in SCBF must be less than or equal to 200. Section 13.2a of the 2005 edition has an upper limit of $4\sqrt{E/F_y}$, with an exception that allowed an upper limit of 200 if additional criteria are met. Can you provide an explanation for this change?

The exception in AISC 341-05 Section 13.2a permitted the slenderness ratio to reach 200 if the available strength of the column was at least equal to the maximum load transferred to the column considering R_y times the nominal strength of the braces connecting to the column. The analysis requirements in Section F2.3 of AISC 341-10 now require that the column design consider this load case. Because the new provisions are requiring this load case be used in the design of the columns, allowing the slenderness ratio up to 200 is appropriate.

The upper bound limit of 200 is based on research as discussed in the Commentary to Section F2.5b, which states:

"Research has shown that frames with slender braces designed for compression strength behave well due to the overstrength inherent in their tension capacity. Tremblay (2000), Tang and Goel (1989) and Goel and Lee (1992) have found that the post-buckling cyclic fracture life of bracing members generally increases with an increase in slenderness ratio. An upper limit is provided to preclude dynamic effects associated with extremely slender braces."

Erin Criste, LEED GA

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