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steel interchange

Slender Web Flexure/Shear Interaction

In editions of the AISC *Specification* prior to 2005, interaction between bending and shear was a required check. Why is this requirement not included in the 2005 and 2010 AISC *Specifications*?

This is discussed in the Commentary to Section G3.1 of the 2010 AISC *Specification* and Section 6.8 of the SSRC *Guide to Stability Design Criteria for Metal Structures*, 6th Ed. (Ziemian 2010), which offers the following explanation:

"Although expressions accounting for shear-moment interaction were considered in past design specifications, the expressions are not included in the 13th edition of the AISC *Specification* or the 3rd or 4th editions of *AASHTO LRFD Bridge Design Specification*. These provisions were removed based upon work by White (2008) that showed that the tension-field design expressions sufficiently capture the behavior with a reasonable amount of accuracy relative to experimental test results."

That is, the design approach was simplified in 2005. The complete reference for White is:

White, D.W. (2008). "Unified Flexural Resistance Equations for Stability Design of Steel I-Section Members—Overview." *Journal of Structural Engineering*, 134(9), 1405-1424.

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

Bolted Connection Ductility

The standard holes given in AISC *Specification* Table J3.3 are typically 1⁄16 in. greater in diameter than the bolt. If a beam with a simple shear connection is erected such that only one bolt in the bolt group is initially resisting the load and the others are not, will the connection be able to deform sufficiently such that all bolts end up sharing the load?

Yes. Tests on bolted joints show that some of the bolts will start out in bearing due to fabrication tolerances in any joint of appreciable size. This means that some other bolts will start out "floating" in the holes. It could take up to 1⁄16 in. of movement before all the bolts bear. Is this a problem? There has been no indication of this in the research that I have seen. Additionally, it is also possible to investigate this situation using rational analytical models.

First, consider the case of a two-bolt connection with very thick connection plates and a relatively small bolt size. The load deformation behavior for such a condition is illustrated in the AISC *Manual* Figure 7-3. Manual Equation 7-1 can be used to determine the proportion of the strength that is developed under a given deformation. Assuming a deformation of 0.0625 in., Equation 7-1 results in a load equal to 65.6% of the ultimate strength of the bolt. Although approximately two-thirds of the bolt strength is achieved before all the bolts have come into bearing, the load-deformation curve flattens out considerably as the ultimate strength is approached.

Equation 7-1 is calibrated so the ultimate strength is obtained at a deformation of 0.34 in. Note that this is nearly half the diameter of the tested bolt—a good bit of ductility. When the bolt that starts in bearing is at 0.34 in. of deformation, it will be predicted to reach 98.2% of its ultimate strength (not 100%, due to the curve fitting involved). The second bolt has a deformation of 0.34 in. – 0.0625 in. = 0.278 in. Equation 7-1 predicts the second bolt will reach 96.5% of its ultimate strength. The predicted percentage of full strength that can be developed by the two-bolt connection is (0.982 + 0.965) / 2 = 97.3%. This is very good, considering some of the reduction is due to the curve fitting and the predicted reduction in strength in the first bolt. Let's also consider the other extreme, a 12-bolt connection with one bolt in bearing and all other bolts at the opposite end of their holes. The predicted strength would be (0.982 + 11(0.965)) / 12 = 96.6%—also very good.

The absolute lower bound is 96.5%, which I would still consider an acceptable result considering that the values in Table J3.2 are reduced by about 10% to account for uneven stress distributions in long end-loaded joints. This reduction in most common joints will be greater than the slight loss of strength that accompanies bringing the bolts into bearing.

Just to consider everything, we should also think about a joint with very stiff and strong bolts connecting relatively thin plates. The bearing checks that are made on joints typically will not predict fracturing of the joint. Instead, they are intended to limit deformations to around ¼ in. when deformation is a consideration. Obviously, this ¼ in. of deformation will go a long way toward evening out the forces in the individual bolts.

Most joints will accommodate the need deformation through some combination of bolt deformation and plate deformation, thereby mitigating the effects of both.

Larry S. Muir, P.E

Bent Plate

The contract documents for a project call out V-shaped stiffener plates that are made by CJP groove-welding two plates at a 45° angle. Is bending a single plate to form these stiffeners a viable alternative to the welded detail?

In the general sense, yes. Plate can be bent to form L-shapes. AISC *Manual* Table 10-13 contains the minimum recommended inside radius for cold bending plate. The minimum radius is dependent on the plate material and orientation of final rolling direction to the bend line. The information in this table is based on the research described in "Brockenbrough, R.L. (1998), *Fabrication Guidelines for Cold*

Bending, R.L. Brockenbrough and Associates, Pittsburgh, PA." This report is available at **www.steel.org**.

In the specific case of your contract, the answer is: yes, with approval—i.e., since you are proposing a change to what is shown in the contract documents, you will need the EOR's approval to substitute bent plates for the built-up shape.

Heath Mitchell, S.E., P.E.

Eccentrically Loaded Single Angles

I am designing a single-angle compression member. The end connections consist of one leg welded to a plate. I would like to use AISC 360-10 Section E5 so that I can neglect the eccentricity of the end connection in the design of the angle. Is it possible to use the modified *KL/r* calculated in this section with AISC *Manual* Table 4-11 for concentrically loaded single-angles?

Yes, AISC *Manual* Table 4-11 can be used for the design of eccentrically loaded single angles by using an effective $(KL)_z$. The single angles must meet the requirements of AISC *Specification* Section E5, including a $b/t \leq 20$. Section E5 allows eccentricity to be neglected if a modified slenderness ratio is used in determining the compressive strength of the angle.

AISC *Manual* Table 4-11 can be used because single angles with $b/t \leq 20$ are designed using AISC *Specification* Sections E3 and E7. Section E4 is not required to be checked for angles with element width-to-thickness ratios in this range. Table 4-11 is to be entered using the effective slenderness ratio calculated in accordance with Section E5(a) or E5(b). You will need to multiply the effective *KL/r* from E5 by r_z to determine the effective $(KL)_z$ to use in Table 4-11.

As another option when using AISC *Specification* Section E5, the effective *KL/r* ratio calculated using this section can be directly used to determine ϕF_{cr} using AISC Manual Table 4-22. Then, ϕP_n can be determined as $\phi F_{cr} A$ per Section E3.

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

Reusing Rivet Holes for Bolts

I am upgrading an existing structure that has riveted connections. Some existing members will need to be replaced with new members. After the existing rivets are removed, is it possible to reuse the existing holes for new ASTM A325 bolts?

It is permitted to replace the existing rivets with ASTM A325 (or A490) bolts. The appropriate bolt diameter should be installed for the hole size per AISC *Specification* Table J3.3. AISC *Design Guide 15* states the following regarding this practice:

"In all types of riveted and bolted connections, old rivets or common (A307) bolts can be removed and replaced with A325 or A490 bolts. If necessary, the old holes can be reamed and larger diameter bolts inserted. It may not be necessary to remove all of the rivets. A325 and A490 bolts tightened to the requirements for slip-critical connections can be considered to share the load with the rivets. The strength of A307 bolts used in combination with rivets or high-strength bolts should be ignored."

Modern Steel Construction has information (in back issues at **www.modernsteel.com**) on rivet removal and installation of A325 bolts. Just type "rivet" in the search to see numerous articles and Steel Interchange questions on rivet removal.

Erin Criste

CJP Groove-Welded Flanges in S-Shapes

Does AISC have a standard for CJP groove-welded splices in flanges of S-shapes?

No. AISC does not have a standard detail.

If the splice is for strength and a CJP groove weld is needed, the concern is the taper of the flange. The answer is related to the joint used from AWS D1.1. As long as the joint has dimensions that don't have a minimum or maximum for the thickness of the parts joined there should not be a problem. As an example, look at Joint Designation B-U4a-GF; the T_1 dimension is unlimited and has no minimum or maximum, so the prep can be tapered (constant angle). To avoid V-shaped weld backing, you may want to choose a joint that uses a back gouge of the root instead of one that requires a backing bar.

If the splice is not for strength, such as for a monorail (a common use of an S-shape), and made to allow the wheels to run continuously on the bottom flange, then the splice need only be good enough to provide a smooth ride for the monorail's wheels. In this case, a PJP groove weld likely can be designed to work.

Erin Criste (with assistance from Mark V. Holland, P.E.)

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Heath Mitchell is director of technical assistance and Erin Criste is staff engineer, technical assistance at AISC. Brad Davis and Larry Muir are consultants to AISC.

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