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steel interchange

Backing for Seismic Moment Connections to the Weak Axis of Columns

This question is about designing a moment connection between a beam and a column web for an ordinary moment frame, where the beam flanges are welded to stiffeners that extend beyond the column flanges. If these welds are subjected to seismic demands and considered demand-critical, must the backing be removed? If the backing is left in place, must a fillet weld be applied between the backing and the flange or stiffener?

This topic is not directly addressed anywhere, so I will offer my opinion along with some rationale.

Fully restrained connections in an ordinary moment frame can be designed using option (a), (b) or (c) of Section E1.6b of the AISC *Seismic Provisions*. Option (c) is not applicable to weak-axis connections. Options (a) and (b) do not explicitly require that backing be removed or that an additional fillet be applied. There is likely to be little benefit from the removal (or further welding) of backing at a moment connection between a beam and a column web. For both of these reasons, the backing should be left in place.

The November 2008 SteelWise (available at **www.modernsteel.com**) stated: "When steel backing is used in tee joints, typical of beam-to-column connections in special moment frames (SMF), the lateral forces will cause bending moments, which impose tensile stresses on these connections, particularly on the bottom beam flange connection in this case. The notch-like condition created by backing left in place in tee joints can serve as a stress concentrator and crack initiator. To eliminate this condition, for the bottom beam flange-to-column flange connection the steel backing is removed and the root pass is gouged to sound weld metal…For top beam flange-to-column flange welds the backing can be left in place with the addition of a reinforcing fillet weld between the backing and column."

The key here is that the article refers to tee joints. A tee joint does not exist at the beam end of a moment connection to the weak axis of a column. The field weld occurs at a butt joint. The stress flow will be different and the backing does not represent as great a concern—and the backing can remain without the reinforcing fillet.

The more critical variable relative to a moment connection to the weak axis of a column is the length and contouring of the plate attached to the column web. Figure 12-14 in the AISC *Steel Construction Manual* provides details and their ductility ratios. Details A and A2 appear to provide a good bit of ductility and are the details I have seen used most commonly.

Larry S. Muir, P.E.

Tension in Bolted Connections with Multiple Lines of Bolts

Do you have any information for the design of unstiffened bolted end-plate hanger connections that have multiple bolt rows (e.g., an HSS welded to an end plate that has two rows of two bolts on each side of the HSS)? I would generally assume that the entire load, including prying, is taken by the first bolt rows and the outer bolts are ineffective. But if the plate is thick enough to eliminate prying at the first bolt row, is it reasonable to use a portion of the outer rows?

The *Guide to Design Criteria for Bolted and Riveted Joints* by Kulak, Fisher and Struik states that more than two gage lines of fasteners are not appreciably effective (page 283). However, it also references a paper by Munse, which indicates that the outer row is not entirely ineffective and that the effectiveness of the outer row of bolts is indeed dependent on the thickness of the joined parts. I am not, however, aware of any published guidance related to determining the participation of the outer bolts, so you would have to determine this based on your own judgment. It is common practice to simply neglect the outer bolts as suggested by Kulak, Fisher and Struik.

Reference:

- W.H. Munse, K.S. Peterson and E. Chesson, Jr., "Strength of Rivets and Bolts in Tension," *Journal of the Structural Division, ASCE*, Vol. 85, ST3, March 1959.

Carlo Lini, P.E.

Column Web Subjected to Out-of-Plane Loading

I am connecting a brace to the web of a wide-flange column using a gusset plate. There is no beam at this location. Section J10 in the AISC *Specification* only deals with member local checks for forces applied on flanges. What local checks apply for this condition?

The weak-axis flexural strength of the web can be determined using a yield line analysis. Punching shear is also a consideration. It can be calculated as the shear strength of the web assuming an effective length equal to the perimeter of the gusset plate at its connection to the web.

In addition to strength, deformation may also be a consideration. The arrangement you describe will possess little stiffness and therefore may allow significant deformation. If this is a concern, then you may want to consider some other configuration.

Carlo Lini, P.E.

Beam-Column Restrained at One Flange

If one flange of a column is restrained by a diaphragm, what are the unbraced length and effective length factors to be considered in the minor direction for the compression check? What is the unbraced length of the column for the bending check?

The diaphragm will provide some resistance to flexural buckling and lateral-torsional buckling. Let's start with flexural buckling under compression axial loading. If the diaphragm provides adequate shear strength and stiffness, it will force the member to buckle in a flexural-torsional mode about the centroid of the plate. This constrained-axis buckling is discussed on page 36 of AISC Design Guide 25 (a free download for members at **www.aisc.org**). The theory behind the equation is from the classic book by Timoshenko and Gere (1961). Design examples and tables were developed by Liu et al. (2013) to aid in the design of wide-flange compression members constrained about one flange.

For flexure, if the compression flange is attached to the diaphragm, the plate will act as a lateral brace, preventing lateral-torsional buckling. If the tension flange is attached to the plate, most design engineers neglect the bracing effect of the plate when calculating the lateral-torsional buckling strength of the beam. This is conservative, because the plate provides some lateral and torsional bracing to the beam. A more refined calculation is usually not justified for the small strength increase typically realized for these members. However, if you are interested in a rigorous solution, Trahair (1993) is a great resource.

References:

- Liu, D., Davis, B., Arber, L. and Sabelli, R. (2013), "Torsional and Constrained-Axis Flexural-Torsional Buckling Tables for Steel W-Shapes in Compression," *Engineering Journal*, AISC, Fourth Quarter.
- Timoshenko, S.P. and Gere, J.M. (1961), *Theory of Elastic Stability*, McGraw-Hill.
- Trahair, N.S. (1993), *Flexural-Torsional Buckling of Structures*, CRC Press.

Bo Dowswell, P.E., Ph.D.

Base Plate Washers

Do washer plates need to be field welded to the column base plate?

For a typical gravity load-only column, there is no need to weld anchor rod plate washers to the base plate. For a column that is also subject to lateral loads, it depends on what load path you are anticipating to get the lateral loads from the column to the foundation. If you are planning to transfer the column shear into the foundation through the anchor rods, then I recommend the washers for those columns be welded.

Section 3.5.3 of AISC Design Guide 1 has a fairly detailed discussion on the pros and cons of this approach. The authors of the design guide recommend using no more than two anchor rods to transfer shear unless you provide a means to ensure all anchor rods are loaded equally. This recommendation is based on the assumption that the washers are *not* welded, in which case the base plate would likely have to slip until it bears directly on a couple anchor rods. Since I am not generally a fan of letting my structural members slip and slide, my preference is to provide welds at washers any time I am designing the anchor rods to transfer the column shear. The Design Guide provides other approaches for transferring the shear that may eliminate the need to weld the washers.

Another thought: I typically require that the washers be welded when the holes have to be enlarged due to mislocated anchor rods and the distance between the edge of the base plate and the edge of the hole gets to be small.

Susan Burmeister, P.E.

Fillet Weld Limitations

In the design of welded connections in building structures, how do the limitations on the maximum and minimum sizes of fillet welds differ for two-sided fillet welds?

The minimum weld sizes provided in Table J2.4 of the AISC *Specification* (a free download at **www.aisc.org/2010spec**) are intended to ensure that there is enough heat input during welding to maintain the soundness of the weld. There is assumed to be no interaction in this regard between weld elements, so there is no change allowed in the minimum weld size for two-sided welds.

The maximum fillet weld size provided in Section J2.2b is not a general requirement, but instead applies to only a specific situation as illustrated in Commentary Figure C-J2.1. It is possible for this requirement to apply to only one of a pair of fillet welds.

Larry S. Muir, P.E.

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