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!

Welding to Galvanized Steel

Where can I find reference material (with paragraph number) to prove to the contractor that the galvanization layer from galvanized steel members must be removed before welding these members together? In other words, the presence of zinc in the weld may weaken the weld capacity and be a source of health concerns for the qualified welders.

Question sent to AISC's Steel Solutions Center

Refer to Section 5.15 of the AWS D1.1-2004 *Structural Welding Code*, which requires that the base metal be free of foreign material that would prevent proper welding or produce objectionable fumes. A similar requirement is found in Section M3.5 of the 1999 AISC *LRFD Specification*. Note that it is possible to coat the top flange of beams with something that keeps the galvanizing from adhering.

I suggest that you visit the American Galvanizers Association (AGA) at www.galvanizeit.org. They have an excellent white paper entitled "Welding and Hot-Dip Galvanizing" that explains all of the design considerations involved. Alternatively, look in AWS D19.0, which covers welding to zinc coated steel.

Bill Liddy
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All-Welded Single-Plate Shear Connection

I believe there is an inherent difference between the "classic" shear tab with one end welded and one end bolted, and an "all-welded" single plate connection, correct? Does a bolted/welded shear tab allow rotation via bolt hole deformation and/or bolt oversize? I haven't found any information on single plate connections with both plate ends welded. My concern is that these all-welded connections allow end rotation of the supported beam. I have a construction project where the detailer specifies single plate, all-welded, simple shear connections. I spoke with him and he told me he details this type of connection all the time. He insists this isn't a problem because the beam web (in this case 0.20" thick), will experience a small amount of local yielding before the welds or shear tab yield (plate is $^{5}/_{8}$" thick), which will allow for a small end rotation and hence provide a "simple shear" connection. Is this argument valid? I'm concerned the web could rupture or yield excessively.

Question sent to AISC's Steel Solutions Center

The AISC procedures for single-plate shear connections only apply to the bolted/welded variety. The design criteria is quite explicit in that the plate thickness must not exceed ($d/2$ + $^{1}/_{16}$"), where d = nominal bolt diameter. This allows the bolts to plow against the holes, allowing for rotational ductility.

All shear connections must provide for the required level of rotational ductility. When using a detail other than those shown in the AISC *Manual*, the designer must ensure that an adequate amount of rotational ductility is provided. It is unclear how a welded/welded single-plate shear connection would provide this.

Sergio Zoruba, Ph.D.
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Anchor Rods Under Combined Tension & Shear

I am looking for some insight into the tension shear interaction for ASTM F1554–Grade 55 anchor rods. Using ASD, Table J3.2 indicates the allowable stress on fasteners and Table J3.3 indicates the allowable tension based on the shear, however these tables are specific to A307, A325, A490, and A449 bolts. Since the transition to the ASTM F1554 for anchor rods, is there an updated recommendation for this interaction? If there is not an updated recommendation, is the use of the interaction based on a corresponding material with a similar F_u, the most appropriate solution?

Question sent to AISC's Steel Solutions Center

There is presently no reference to ASTM F1554 anchor rods for combined shear and tension checks in the AISC *Specification*, though such fasteners could be treated similarly. The tables in Section J were primarily intended for structural fasteners as used in steel-to-steel connections. When checking anchor rods for tension or shear, Table J3.2 could be used for the steel part of the anchorage system only. However, anchor rods are usually not subjected to the same forces as structural fasteners and typically it is preferable to avoid using the anchor rods in shear, due to the difficulty of transmitting shear into them. Besides the requirements for the steel rods, the anchorage design is also very dependent on the concrete elements of the foundation system. I also suggest that you refer to ACI 318 Appendix D as they have an interaction equation for checking combined stresses on the controlling elements.

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

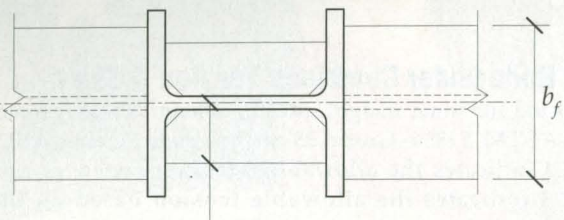
Narrow Columns with Continuity Plates

I am designing a moment connection between a W24×117 beam and a W18×46 column. The W18×46 column was sized to fit in a 6" wall. The beam is in a duct area, so width was not an issue. The column requires web stiffeners and *LRFD Specification* Section K1.9 (1) requires that the stiffener width be equal to one third of the beam flange width plus one half of the column web thickness. This equates to 12.8/3 + 0.36/2 = 4.1", which is problem due to the fact that my column width is only 6" and the stiffener would be past the column flange. Is it permissible to put in the required width of column web stiffeners, say 4¼" wide, and then notch the web stiffener the thickness of the column flange so that it meets back up with the abutting flange of the connecting beam? I don't know if it is allowed to have the web stiff-

ener stick out past the column flanges to meet the requirement of *LRFD Specification* Section K1.9 (1).

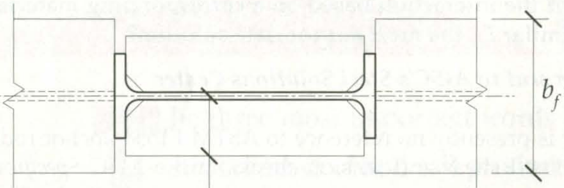
Question sent to AISC's Steel Solutions Center

The following sketches indicate the stiffener width details discussed:



$$\frac{b_f}{3} + \frac{t_w}{2} \text{ (min.)}$$

LRFD Specification Section K1.9 (1)



$$\frac{12.8}{3} + \frac{0.36}{2} = 4.1'' \text{ (min.)}$$

The *Specification* reference cited relates to the minimum stiffener width requirement, not the maximum. Transverse stiffeners are used to resist column flange bending and column web limit states such as buckling and yielding, rather than web panel-zone shear. You would have a situation where a beam with a wide flange would be attached to a column with a narrow flange. In such a case, a major question arises relative to the connection detail of the beam to the column flange and how the stiffener/continuity plate relates to this geometry. You may want to address this detail first and the minimum stiffener plate width may become a moot point.

One could notch the stiffeners as you said, but the stiffeners would likely need to be wide enough to abut against the remaining beam flange. Having only 4¹/₄″ of stiffener width would be insufficient to accomplish this task. Keep in mind that Section K1-9(1) is a minimum width criterion in order to make the stiffener effective in stiffening the column. Having the stiffeners running to the toes of the beam flange could effectively address the stiffening of the column, but you still need to address the transfer of the beam force into the column and plate. Watch out for beam width/column width differences like these.

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

Slender WT Stem in Flexural Compression

I am trying to design a WT with its stem in flexural compression per the 1999 *LRFD Specification*. The member has a slender stem and I have calculated Q_s in accordance with Appendix B5.3a. I have checked the flexural limit states of yielding and lateral-torsional buckling strength per Section F1.2c. Does the reduction factor Q_s apply to the lateral-torsional buckling strength and the yield strength? Or does it apply only to the yield strength?

Question sent to AISC's Steel Solutions Center

The flexural design is based on the maximum stem compressive stress not exceeding F_yQ_s per Appendix B of the 1999 *LRFD Specification*. This implies that local buckling governs the design rather than yielding or lateral-torsional buckling. Please note that you cannot use the Chapter F expressions for your case, as they only apply to compact and non-compact members (as mentioned in the first paragraph in Chapter F.) Your WT stem is a slender unstiffened element.

Sergio Zoruba, Ph.D.
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Editor's Note:

Bill Liddy of the AISC Steel Solutions Center is a periodic contributor to* Steel Interchange*. We would like to congratulate Bill on his 50-year anniversary with the steel construction industry. Keep up the good work, Bill!

Steel Interchange is a forum for *Modern Steel Construction* readers to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Opinions and suggestions are welcome on any subject covered in this magazine.

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If you have a question or problem that your fellow readers might help you to 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:

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