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!

Pretensioning Low-Strength Anchor Rods

We have a project that is all braced frame construction. The anchor rods specified were ASTM A36 from 1¾ to 2½ diameters. The design specifications are requiring us to pretensioned A36 anchor rods to the minimum yield point specified for A36 steel. They request that this be done by the turn of nut method as indicated by the RCSC *Specification for Structural Joints using ASTM A325 or A490 Bolts*. This does not appear correct to our people. We see no correlation between RCSC and A36 anchor rods. Please give us your thoughts.

Question sent to AISC's Steel Solutions Center

The RCSC specification applies to fasteners in steel-to-steel connections; not anchor rods. ASTM A36 material is a mild carbon steel not suitable for use as a pre-tensioned fastener. The RCSC specification only deals with high strength, ASTM A325 or A490 bolts which in some connections are required to be pretensioned. When pretensioning is required, a minimum bolt force equal to 70% of the specified minimum tensile strength of the bolt is to be achieved. This does not apply to bolts of other materials. AISC specifications do not address pre-tensioning of A36 rods or bolts. Similarly, the RCSC specification "Turn of Nut" tightening method is not applicable to A36 bolts.

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Beam Stability under Reverse Loading

A composite steel beam (W12×19) with concrete on metal deck diaphragm 4″ total thickness is loaded with uplift. Can we assume that although the slab is at the tension flange it still restrains the beam against lateral-torsional buckling, so that only local buckling of flange/web has to be checked?

Question sent to AISC's Steel Solutions Center

You would need to check the web distortional stiffness before you make the assumption that bracing the tension flange is sufficient to prevent buckling. Refer to Section C3.4b of the 1999 *LRFD Specification* (www.aisc.org/lrfdspec) for torsional bracing. Your case is continuous torsional bracing. Be sure to read the commentary section to C3 to get a good understanding of what you are checking.

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Knee Braces on Crane Support Columns

It is my understanding that the practice of placing knee braces from crane support columns to the underside of crane support rails is not acceptable (in the past this was done to reduce the unsupported length of the crane rail). Just what is the reason for elimination of the knee braces?

Question sent to AISC's Steel Solutions Center

Knee braces were often used in the past as you have stated. The reason that many engineers have stopped using these is that problems developed due to the knee brace acting as a support to pick up the vertical wheel load each time the crane passed. Conditions of a gradient stress range often resulted in fatigue problems of the girder. If knee braces are used, the effects of the brace support should be analyzed in relation to the geometry and resulting stresses in the girder. Typically, these should only be considered in very lightly loaded and low fatigue situations.

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Fillet Weld Design Strength Increase

I have a project for which I am using flange-welded moment connections. The fabricator has asked us to look into the possibility of replacing the full-penetration weld from the plates to the column flanges with two-sided fillet welds, sized to achieve the same capacity as the full-penetration welds. However, my supervisor believes there is some issue with fillet welds in direct tension, and that he is not comfortable with this connection as proposed by the fabricator. Can you refer me to any publication that shows the fillet welds are not the preferred way of making the connection?

Question sent to AISC's Steel Solutions Center

AISC specifications and the AWS D1.1 welding code both allow fillet welds to be loaded transversely to their longitudinal axes. It is not tension on the fillet welds, rather shear on the effective area of the welds produced by the beam flange force. In fact, research has been conducted and AWS D1.1 provides a 1.5× increase in weld design strength for this case (i.e. transversely loaded fillet welds now have a 150% increase in design strength compared to welds loaded parallel to their longitudinal axes). This increase was also adopted by AISC and is found in Appendix J of the 1999 *LRFD Specification* (www.aisc.org/lrfdspec). Please note that the 1989 *ASD Specification* does not contain this relatively new provision.

I believe your supervisor has a concern with ductility of the weld. Transversely loaded fillet welds are indeed less ductile than parallel loaded fillet welds. However, they can be used and the specification and code allows them. The use of fillet welds in the beam to column flange connections makes the design more economical. It is important to note that fillet welds can be used instead of CJP groove welds for wind and low-seismic applications (R = 3 or less). For high-seismic applications ($R > 3$; the AISC *Seismic Provisions*), you must use welded joint details that are similar to those used in the tested assembly upon which the moment connection is based. Most tested assemblies have used CJP groove welds in such welded joints.

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Prequalified Shop Welding

I have a number of "crippled" beams where the engineer of record calls for the web and flanges to be complete penetration welded at the miter. Since the beam size is such that the flange and web are less than 5/16" thick, I would like to detail the piece with out prepping the flanges or web and use the designation B-L1b-GF. Is it reasonable to believe that most shops and the field can use this welding process?

Question sent to AISC's Steel Solutions Center

The B-L1b-GF prequalified welded joint can be done by either of the two processes listed, GMAW or FCAW. The availability of any of the welding processes in the shop or in the field is dependent on the size of the project and the equipment that the particular fabricator or contractor may have. It is reasonable to believe that most structural steel fabricating plants likely can produce welds by one of these processes. Also, where there is a significant amount of welding in the field you are likely to find one of these processes available. It is on the projects with only a small amount of field welding that you may find that the erector is not equipped to provide this type of weld. In any case it is suggested that you question the availability as early as possible to avoid surprises.

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Reduction in Stud Shear Connection Design Strength

Section I3.5b in the 1999 *LRFD Specification* limits N_r to the "number of stud connectors in one rib at a beam intersection, not to exceed three in computations, although more than three studs may be installed." My question is does the above definition mean that more than three studs can be installed and counted on for strength but that only in equation I3-1 need the value for the parameter N_r be limited to a maximum value of three? Or does it mean that never more than three studs per rib at a beam intersection can be counted for any part of composite design?

Question sent to AISC's Steel Solutions Center

You can use more than three shear connectors in the beam intersection, although numerically, N_r should not exceed a value of three in the shear connector strength reduction expression. The three shear connector limitation applies only to the N_r expression calculation for strength reduction per shear connector. You may have more than three shear connectors in a rib, but each stud is limited to the reduced capacity. There is still a geometric limitation based on the intersecting area of rib and beam flange width with respect to spacing requirements as defined in Section I5.6.

An *Engineering Journal* (**www.aisc.org/ej**) paper entitled "Composite Floor System for Sears Tower" by Iyengar and Zils (3rd Quarter, 1973) discusses the results of composite slab testing for deck ribs oriented perpendicular to the steel beam. They found that increasing the number of shear connectors in the beam intersection created overlapping stress cones, thereby reducing the ultimate strength of each individual stud. However, the ultimate strength of the beam intersection increases with additional shear connectors, albeit, by a smaller amount as additional shear connectors are added.

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HSS Connection Economy

Which is more economical to specify—shear tabs or through-plates when connecting a wide-flange beam to a HSS column?

Question sent to AISC's Steel Solutions Center

Conventional shear tabs will almost surely be less expensive than through-plates. The main issue is the cost associated in making through-plate connections compared to using single-plate shear connections on both sides of a tubular column. The best way to confirm this is to ask your fabricator for a comparison quote. In general, through-plate connections are not the preferred shear connections for tubular columns because of fabrication cost. Note that through plates may become necessary if the HSS wall is slender or if the punching shear limit state requirements are not met.

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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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