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!

Seal Welds

My question is relative to seal welding on a typical painted industrial project with the steel exposed to the environment. If there are no requirements for seal welding of all unwelded joints on a project in either the contract documents or the bid drawings, what should be the presumption made by the fabricator?

Question sent to AISC's Steel Solutions Center

Refer to AISC FAQ 8.7.3 at www.aisc.org/faq. Seal welds are rare in building construction and therefore should be called out in the contract documents when required. Simply adding them could create a different problem. According to AWS D1.1-02 Section 6.5.1 *"The inspector shall make certain... that no unspecified welds have been added without approval."* Alternatively, you may want to investigate the use of silicone caulk to seal exposed joints from the elements after the structural steel is painted.

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

HSS Seismic Braces

One of our observations is that the criteria $\phi F_u A_e \geq R_y F_y A_g$ (Design Strength ≥ Required Strength) often cannot be satisfied with an ASTM A500 HSS anywhere along the whole length of the brace, even where there are no holes or slots. It seems to me the intent is to meet the criteria at the connection (not to build-up the section along its entire length.) Are HSS suitable for use as braces in concentrically braced frames?

Question sent to AISC's Steel Solutions Center

This check is intended to be made at end connections, not along the member length for the gross section. Accordingly, the Commentary of Section 13.3b of the 2002 AISC *Seismic Provisions* suggests that local reinforcement would be needed for HSS and pipe, but it does not imply that the entire bracing member would need reinforcement. In addition, Commentary Section C6.2 states:

Specific provisions for some Seismic Load Resisting Systems stipulate that the Required Strength be determined by multiplying the Nominal Strength of a certain member or connecting element by the value of R_y *for the corresponding material grade. This overstrength is primarily of interest when evaluating the Design Strength of another connecting element or member. It is not of interest, however, when evaluating the Design Strength of the same member to which the value of* R_y *was applied in the determination of the Required Strength. Therefore, when both the Required Strength and Design Strength calculations are made for the same member or connecting element, it is also permitted to apply* R_y *in the determination of the Design Strength.*

Keith Mueller, Ph.D.
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Tension Rod Braces

I have a project that is designed with hanger braces, uplift braces and horizontal braces. The design has custom fabricated connections at the ends of the rods that resemble a clevis but...no turnbuckles, just right and left hand threads at the clevis. Is there an industry standard for setting tension in these rods, or is it generally accepted to install them more or less loose?

Question sent to www.steel-link.com

There is no standard. Generally, draw is used for light tension members like angles. The turnbuckles are usually the means to take slack out of rods. The idea is just to eliminate the sag, not to induce any specific level of pretension.

Without the turnbuckles, this will be a real trick. Perhaps someone else has an idea better than this one, but I'd put one diagonal in, pull up the other diagonal dimension with a come-along to just beyond taut in the first, install the second diagonal, and release the come-along. The framing should go "boing" (technical term) into shape with both diagonals, then in similar pretension.

Charlie Carter, P.E., S.E.
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Sufficient Thread Engagement

The 2000 RCSC bolt specification in the 3rd Edition LRFD *Manual* defines sufficient thread engagement as having the end of the bolt extending beyond or at least flush with the outer face of the nut. That is easy to determine for ASTM A325 and A490 bolts. However, how would this apply to ASTM F1852 twist-off-type tension-control bolts?

Question sent to AISC's Steel Solutions Center

The definition for sufficient thread engagement in the 2000 RCSC bolt specification contains a requirement that the strength of the bolt must be developed. The exact definition is:

Sufficient Thread Engagement—having the end of the bolt extending beyond or at least flush with the outer face of the nut; a condition that develops the strength of the bolt.

For ASTM F1852 TC bolts, the same definition applies. You should find that when the spline breaks during pretensioning, the end of the bolt (remaining face at the break neck) is at least flush with the outer face of the nut. That is, if the TC bolt does not experience a thread stripping failure during pre-installation verification, it will not experience it during pretensioning (or during service.) We have seen cases where the remaining face at the break neck has a small dimension unthreaded. However, this condition still satisfies RCSC intent if the TC bolts meet the pre-installation verification requirement.

Sergio Zoruba, Ph.D.
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ASTM A325/A490 Bolts

What constitutes "reuse" of ASTM A325 and A490 high-strength bolts? It has been my understanding that "reuse" would be a bolt that has been brought to specified pretension a second time for size and grade.

Question sent to AISC's Steel Solutions Center

Reuse is when a previously pretensioned bolt is removed and reinstalled in a pretensioned joint. Refer to the Commentary in Section 2.3.3. of the 2000 RCSC bolt specification (a free download from www.boltcouncil.org). It states the following:

Pretensioned installation involves the inelastic elongation of the portion of the threaded length between the nut and the thread run-out. ASTM A490 bolts and galvanized ASTM A325 bolts possess sufficient ductility to undergo one pretensioned installation, but are not consistently ductile enough to undergo a second pretensioned installation. Black ASTM A325 bolts, however, possess sufficient ductility to undergo more than one pretensioned installation as suggested in the Guide [Guide to Design Criteria for Bolted and Rivet Joints, *a free download from www.boltcouncil.org*] *(Kulak et al., 1987). As a simple rule of thumb, a black ASTM A325 bolt is suitable for reuse if the nut can be run up the threads by hand.*

Bill Liddy
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Bearing Strength

In Section J3.10 of the 1999 LRFD *Specification*, how does one determine if deformation is a design consideration? Does this "deformation" refer to slip, i.e., does equation J3-2a apply only to slip critical connections? We are evaluating an existing building and there are no notes or details to indicate that we are dealing with slip-critical connections. Can I assume the use of equation J3-2b (which gives a higher value)?

Question sent to AISC's Steel Solutions Center

The reasons for designing for deformation being or not being a design consideration are explained in AISC FAQ 5.1.2 (www.aisc.org/faq).

Typically, designers will use the bearing-strength expression with deformation considered, which restricts bearing deformations to less than ¼". In cases when the designer does not consider large deformations in bearing to be detrimental to performance, the higher strength equation can be used.

In either case, this deformation has nothing to do with slip. Once slip occurs, the bolt would be bearing against the edge of the hole. Then it becomes a matter of selecting if you want to design for deformation being or not being a design consideration, which implies up to ¼" or more than ¼" deformation beyond the original hole edge, respectively.

Please note that a slip-critical connection is designed not to slip in the service load range. However, a slip-critical connection must be designed for bearing because slip can occur before the ultimate load is reached.

Sergio Zoruba, Ph.D.
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Welding Anchor Rods

How can rod-type concrete anchors be welded to embedded plates?

Question sent to AISC's Steel Solutions Center

The answer to your question is found as AISC FAQ 7.3.1 at www.aisc.org/faq:

If a common shear-stud connector size is suitable, the stud-welding provisions of AWS D1.1:2002 Section 7 can be used. Other rod-type anchors can be square-cut and fillet-welded if strength is adequate. Because the weld length is π times the rod diameter, such welding provides limited strength. When a greater welded strength is required, the rod can be beveled on two sides to a chisel point, which allows for easier deposition of weld metal than beveling to a pencil point.

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Steel Interchange is an open 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.

The opinions expressed in *Steel Interchange* do not necessarily represent an official position of the American Institute of Steel Construction, Inc. and have not been reviewed. It is recognized that the design of structures is within the scope and expertise of a competent licensed structural engineer, architect or other licensed professional for the application of principles to a particular structure.

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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