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 Sheared Edges

Is there data providing support to the theory that welding to a sheared edge of a connection angle is acceptable? Angle thickness will vary from 5/16" to 3/4".

Question sent to AISC's Steel Solutions Center

AWS D1.1 Structural Welding Code–Steel places requirements on welding to sheared edges if the joint is part of a cyclically loaded structure. Section 5.15.3 of AWS D1.1:2004 requires that in cyclically loaded structures, sheared material thicker than ½" shall be trimmed as required to produce a satisfactory welding edge. This sheared material stipulation is one of five conditions that are listed as requiring special attention. This requirement is based on experience that cracks may be present in such listed edge conditions, and that these may have a tendency to propagate under cyclically loaded conditions.

Lack of requirements pertaining to edge conditions that are not listed is an indication that the AWS committee did not feel there is concern regarding welding to those edges.

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

Backing Bar Removal

When using a field bolted/field welded moment connection, is it AISC standard practice to leave the backing bar in place after the weld is complete?

Question sent to AISC's Steel Solutions Center

The answer is sometimes yes, sometimes no, depending on the project conditions. The subject of backing bar removal is discussed in the Frequently Asked Questions section of the AISC web site (**www.aisc.org/faq**). FAQ 8.2.5 discusses when backing bars and run-off tabs should be removed after welding. (Note: The references cited in the FAQ section are presently being updated to current document standards.)

To produce sound welds on many welded joint geometries, run-off tabs projecting from the finished member may be required to permit starting and stopping welds beyond the edge of the member; thus AWS D1.1:2002 Sections 5.10 and 5.31 should be followed. Additionally, the 1999 LRFD specification (**www.aisc.org/lrfdspec**) Section J1.5 addresses requirements for the removal of backing bars and weld tabs at complete-joint-penetration groove welded splices in ASTM A6 Groups 4 and 5 rolled shapes and plates exceeding 2" thickness subject to primary tensile stresses. When such welding aids are required to be removed, the surface should be finished, as indicated in 2.2.6 and 2.2.7.

Damage to welded beam-to-column-flange moment connections in the 1994 Northridge earthquake has raised several welding and seismic detailing issues, and new criteria have been established. Explicit requirements for the removal of back-up bars and run-off tabs in seismic projects have been included in the 1997 AISC *Seismic Provisions for Structural Steel Buildings* and the corresponding Supplement No. 2. An exception is included for tested assemblies that can be demonstrated to have acceptable performance with alternative treatments.

The reference above to the 1997 AISC seismic provisions with supplement is equally applicable to the forthcoming 2005 AISC seismic provisions.

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

Fatigue Based on Stress Calculation

I have a column with a hot-rolled crane bracket fillet welded to the inside column flange. Table A-K4.2, Case 21, in the 1989 ASD specification seems to apply. I used the elastic (vector) approach to design the web-to-column flange weld. Should I just take the applied kips/in. that I used for the welds and divide that by the web thickness to get a stress in ksi? That seems like a slightly simplified and conservative method.

Question sent to AISC's Steel Solutions Center

We suggest using the latest 2005 AISC specification (**www.aisc.org/2005spec**) for both ASD and LRFD. This document represents the latest in terms of testing and information for structural steel design. Section 3.2 of the 2005 specification states, "Calculated stresses shall be based upon elastic analysis. Stresses shall not be amplified by stress concentration factors for geometric discontinuities." There are more sophisticated methods using finite elements analysis and hot-spot stress curves, but the SN curves in the AISC specification do not apply to those analysis methods. If you are interested in those methods, the International Institute of Welding (IIW) has design methods.

Case #21 of the 1989 ASD specification contains shear and bending moment force components. Hence you would need to determine the maximum stress where the weld and plate meet (at the top or bottom of the ends of the plate, as those points are the locations of the maximum flexural stresses). That would be the combination of V/A and M/S using the applied loads, the total load for the maximum stress, and the live (cyclic) loads for the stress range. Then compare that stress range to the allowable fatigue stress range outlined in the specification for the applicable stress category constant of both the base and weld metal components.

In the current 2005 specification, there are three fatigue stress categories to check. The attached plate would be checked against category C and equation A-3-5 as in case 5.6. If there is transverse stress in the supporting flange, which would not be expected, Case 5.7 would be relevant. The fillet weld throat needs to be sized using Category F from Case 8.2.

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

Weld Access Hole Requirements

I understand the detailing and fabrication requirements for weld access holes in moment connected beams have changed in the last few years. Could you give me information on what the new requirements are?

Question sent to AISC's Steel Solutions Center

Information on "Beam Copes and Weld Access Holes" is covered in Section J1.6 of the 2005 *Specification for Structural Steel Buildings*. In contrast to previous specifications pertaining to this section, the 2005 specification specifically defines the height of the access hole judged sufficient to permit welding and inspection requirements. Copies of the 2005 specification and commentary are available to download free from **www.aisc.org/2005spec**. For high-seismic prequalified moment connection weld access hole requirements, refer to the FEMA 350 and 353 documents. Contact the FEMA distribution facility at 800.480.2520 to order free copies.

AISC will soon issue a Connection Prequalification Review Panel (CPRP)-approved document that will replace the FEMA documents for prequalified connections.

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

Seismic Provisions and Collector Design

The definition of the seismic load resisting system (SLRS) in the 2002 seismic provisions glossary includes collectors. Does it follow that Section 7.2 (seismic bolting requirements) applies to collector design? If yes, then how can one design an axial bolted connection for a ductile limit state?

Question sent to AISC's Steel Solutions Center

Yes, the seismic bolting requirements found in the 2002 AISC *Seismic Provisions for Structural Steel Buildings* (**www.aisc.org/seismic**) also apply to collectors, struts, and diaphragms. James Malley, chairman of the AISC Seismic Task Committee, suggested that axially loaded collector bolted connections would need to be designed for a ductile limit state (i.e. yielding before fracture), similar to a brace connection.

This requirement may necessitate some reinforcement at the bolted end connection because the expected yield strength R_yF_y must be developed. Note that the 2005 AISC seismic provisions will, in addition to the R_y values, also introduce R_t values to account for the actual material tensile overstrength. R_t is useful in this case because the fracture check is made on the same material that is intended to yield before fracture.

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

Mill and Fabrication Tolerances

I am being asked to help resolve a dispute between a steel erector and another subcontractor who is installing handrail to the side of a vertical C channel. I believe that the top of this channel got pulled in during erection so the web is no longer exactly vertical. I can't find anything specifically addressing this case in the *Code of Standard Practice for Steel Buildings and Bridges* (COSP), but I am probably just missing something.

Question sent to AISC's Steel Solutions Center

There is no specific tolerance in the COSP that addresses the disputed installation condition.

Erection tolerances covered in the COSP are basic in defining the positioning and location of a member with relation to work points, and mill tolerances define permissible cambers and sweeps of the rolled members along the span length. Section 7.13 of the COSP defines the location of working points for horizontal members as the centerline of the top flange or top surface at each end, and the working line is a straight line between those two points. The work points define the general position elevation and alignment for the ends of the member. Tolerances for the rolled shape are given in ASTM A6, including permitted variations in cross section, camber, and sweep, which defines permissible variations along the length of the member relative to the work line. However, there is no specific tolerance for twist of a cross section along the length of the span.

This points to the importance of realizing that elements provided by different subcontractors, whether they be steel and steel, steel and concrete, or other subcontractor combinations, require coordination not just for schedule, but also for tolerances.

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

Base Plate Minimum Edge Distance

Are there any standards for the minimum edge distance required for column base plate holes?

Question sent to AISC's Steel Solutions Center

Anchor rod layouts should provide ample clearance distance for the washer from the column shaft and its weld, as well as a reasonable edge distance. When the hole edge is not subject to a lateral force, even an edge distance that provides a clear dimension as small as ½" of material from the edge of the hole to the edge of the plate will normally suffice, though field issues with anchor rod placement may necessitate a larger dimension to allow some slotting of the base plate holes. When the hole's edge is subject to a lateral force, the edge distance provided must be large enough for the necessary force transfer.

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