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!

Base Plate Edge Distances

Anchor traditionally have holes in base plates and in the anchor chairs/seats larger than is customarily provided for steel-to-steel connection bolts. What are the guidelines for edge distances for these holes? I have some field-enlarged holes for 1¾″ dia. bolts in an anchor seat/chair that are 3½″ in dia. What edge distance criteria should I use? We plan to use 1″ × 5″ × 5″ plate washers under the nuts for these bolts.

Question sent to AISC's Steel Solutions Center

A first suggestion is to start referring to anchorage to concrete devices as anchor rods, rather than anchor bolts. This will help to remember that the requirements for these rods are much different, both in terms of loading and in terms of detailing, than those for structural bolts used in steel-to-steel connections.

AISC requirements for structural bolts, such as hole sizes, edge distances, spacing, etc. are stated in Section J of either the current AISC LRFD or ASD specifications. Suggested hole sizes for anchor rods are listed in the AISC ASD *Manual of Steel Construction*, ninth ed., on pages 4-130 and 4-131. The LRFD *Manual of Steel Construction*, third ed., lists the hole size in Table 14-2 on page 14-27. You will note that the later (LRFD) manual has larger sizes than the much earlier (ASD) manual. These hole sizes have been increased from the earlier ASD editions to provide for better harmonization with actual performance of other trades in setting anchor rods.

The type of loading normally imposed on anchor rods are normally much different than those imposed on structural bolts in steel-to-steel connections. For further information concerning beam bearing plates, column base plates, and anchor rods, I suggest a review of Part 14 of the AISC LRFD *Manual of Steel Construction*, third ed. Edge distance criteria are dependent on what type of force the anchor rods are designed to transfer.

In many cases after erection, the anchor rods are not relied on to transmit any force. Should the rods be required to resist tensile or shear forces in the design, the EOR must evaluate the resulting forces and stresses in both the steel and concrete elements of the foundation. Significant guidance can be found in AISC *Design Guide 10 – Erection Bracing of Low-Rise Structural Steel Frames*, which is a free download for AISC members at www.aisc.org/epubs.

Kurt Gustafson, S.E., P.E.
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Rotational Ductility of Shear Connections

I received a response from the AISC Steel Solution Center concerning the rotational ductility of an all-welded shear tab connection. The response stated that they did not recommend this detail due to its inability to provide adequate rotational ductility—i.e. the lack of bolts eliminates plowing action against the tab holes as a means for accommodating beam end rotation. Another question has arisen, where the fabricator has detailed double angle connections that are welded to the beam web and then bolted (or welded) to columns or girder beams. The ninth edition ASD *Manual*, Chapter 4, clearly denotes the use of these two types of connections. Could you please explain why these two types of connections seem to be permissible when it appears as though the same situation as the "all welded shear tab connection" applies? Do these configurations assume the bolts in these connections can elongate to allow free rotation?

Question sent to AISC's Steel Solutions Center

These connections are different. In double-angle connections, it is the double angles that deform (the outstanding legs flex) to provide the rotational capacity for the beam end rotation. This same behavior does not occur in an all-welded single-plate shear connection. Furthermore, such a connection may be impractical because to erect them, you have to have bolt holes for the erection bolts, and then you still have to weld the angles.

All welded double-angle shear connections must be designed to flex at the top of the angles (at the supporting member), thus providing for rotational ductility. To ensure that this flexibility is available, Section J2.2b(3) of the 1999 AISC LRFD *Specification* requires that the top weld be terminated short by a distance equivalent to at least two times, but not more than four times, the nominal size of the weld.

For double-angle outstanding legs that are bolted to the supporting member, the top of the angles will also flex to allow for the rotational ductility. This flexing is maximum at the top and reduced as one progresses toward the bottom.

The same type of behavior occurs with tee-type shear connections.

Sergio Zoruba, Ph.D.
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Field Cutting of Openings

What is the correct method for field cutting new openings in the web of a steel beam? The beam is already erected and the opening locations are acceptable.

Question sent to AISC's Steel Solutions Center

I don't know if the term "correct method" is really applicable. It depends on the type of penetration and the equipment and capabilities of the contractor. Small round holes may be cored, while larger or rectangular penetrations may be more practical to prepare by flame cutting. If you can use a template to guide the torch, so much the better.

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Wind Connections

I am reviewing a steel building constructed in 1950 (no existing drawings). I don't see any braced frames in the structure, but find that the beam-to-column detail is built as a standard double-angle three-bolt shear connection, also with a top and bottom angle with bolts to the column flange and the other leg welded to the beam flange. This appears to be a semi-rigid connection for a moment frame. Was this approach widely practiced back in the 1950s?

Question sent to AISC's Steel Solutions Center

Yes, it does sounds like a "wind connection" (i.e. a simplified historic approach PR connection). There were certain assumptions that were made in designing such a connection, as outlined in the AISC *Manual*.

Please note that we now call wind connections FMC (flexible moment connections). The beams and shear connections are selected as though the beams behave as simple-span members, the flange connections are sized for the lateral moments only, and the columns are sized for combined gravity loads and lateral moments. The third edition LRFD *Manual* addresses two FMC connections, the flange-angle and the flange-plated flexible moment connections. The design criterion for both is found in the third edition LRFD *Manual*. We do not have design criteria for other types of FMC connections.

Refer to the following *Engineering Journal* papers:

- "Directional Moment Connections - A Proposed Design Method for Unbraced Steel Frames", R.O. Disque, *Engineering Journal*, first quarter, 1975.
- "Wind Connections with Simple Framing", R.O. Disque, *Engineering Journal*, 1964.
- "Simplified Frame Design of Type PR Construction", Michael H. Ackroyd, *Engineering Journal*, fourth quarter, 1987.
- "Behavior and Design of Flexibly Connected Building Frames", K.H. Gerstle and M.H. Ackroyd, *Proceedings of the 1989 NASCC*, pp 1.1-1.28.

All of these papers can be download from **www.aisc.org/epubs** (free to AISC Members). Additionally, you can look forward to a paper in the second quarter 2005 AISC *Engineering Journal* by L.F. Geschwindner and R.O. Disque that brings the historic approach to flexible moment connections up to current standards.

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Older Steel Grades

For a building constructed in 1961, what would have most likely been the specified yield stress for the steel? Would steel meeting ASTM A7 have been the typical steel used? Would the yield for A7 have been 33,000 psi?

Question sent to AISC's Steel Solutions Center

ASTM A7 was the common steel upon which the 1961 printing of the fifth edition of *Manual of Steel Construction* was based. This manual included the 1949 AISC *Specification*. The minimum yield point for A7 steel was given as ½ of the tensile strength or not less than 33,000 psi. However, the 1961 printing of the *Manual* also included "Supplementary Provisions for the use of ASTM-A36 Steel," applicable to what was then a new material. The use of this A36 material was permitted under the *Specification* based on these supplementary provisions until a new AISC specification could be developed. It is probable that you would find the A7 steel in building construction during that 1961 time frame; however, the use of A36 would have been a possibility.

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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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