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!

Thermal Cutting

I received an RFI requesting a modification that includes thermal cutting (torching) of the deck closure angles (diaphragm chords). The special inspections agency is requesting that we verify this modification. I am trying to research the effects of thermal cutting. Do you have any recommendations on where to look for information as far as codes or technical references?

Question sent to AISC's Steel Solutions Center

Thermal cutting is a normal operation in the fabrication of structural steel. You may want to look at Section 2.2 of the Frequently Asked Questions section of the AISC website at **www.aisc.org/faq**. You can also find requirements for thermally cut edges in Section M2 of the AISC specification (a free download at **www.aisc.org/2005spec**). AWS D1.1 also provides requirements and discussion of thermal cutting practices.

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

Weld All Around?

Current details show an all-around fillet weld symbol at wide-flange column to base plate connections. I thought that this wasn't a good practice. Could you please comment?

Question sent to AISC's Steel Solutions Center

It isn't good practice. We recommend welding on the flat surfaces only. The small amount of weld across the toes of the flanges and in the areas of the fillet radii add very little strength and are very costly. Yet welding around the corners and across the toes of the flanges is difficult and may result in rapid melting at the corners and a resulting gouge during welding. The repair implications outweigh any benefits of welding all around.

Further discussion of the subject can be found on page 4 of AISC *Design Guide 1: Base Plate and Anchor Rod Design (Second Edition)*, available from **www.aisc.org/epubs**.

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

Design Stress for Compression Members

In the third edition of the LRFD *Manual of Steel Construction*, on page 16.1-143, there is a table for design stress for compression members in relation to *Kl/r*. Where is this table in the *Steel Construction Manual*, 13th edition?

Question sent to AISC's Steel Solutions Center

This information is now found in Table 4-22, which is in Part 4 of the 13th edition *Manual*. Since these specific tables are derived values based on specification equations, it was decided to remove the tables from the specification, and include them in the column section of the *Manual*.

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

Section Properties of Back-to-Back Channels

Does AISC provide a spreadsheet that calculates the section modulus of two channels welded back to back?

Question sent to AISC's Steel Solutions Center

No, but it is a simple calculation. I_x is twice that of a single channel, and I_y can be determined by application of the parallel axis theorem using the area of each channel multiplied by the distance squared from the y-axis to the center of gravity of each channel. The section modulus can then be derived based on the moment of inertia value.

If you create a spreadsheet to do this, consider posting it for others to use on a web site like **www.steelutilitiesonline.org**.

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

Minimum Connections

Section J1.7 in the AISC specification covers "minimum strength of connections" and requires a factored load not less than 10 kips for connection design. Where does the "10 kips" come from? Is that empirical? Can I use this load for the connections of precast concrete structures?

Question sent to AISC's Steel Solutions Center

Your reference to Section J1.7 must be in relation to an older LRFD specification where the 10 kip minimum strength of connection was included; this was an arbitrary requirement based upon the older and similarly arbitrary 6 kip minimum strength in ASD. Such a minimum requirement is no longer included in the current 2005 specification, as it was deemed a rather meaningless requirement; most standard connections provide far in excess of that minimum capacity. In addition, OSHA now requires that beams have at least two bolts installed prior to release from the load line during erection.

The AISC specification does not cover design parameters required for precast concrete structures. Perhaps an organization that addresses precast concrete structures could assist you with that part of your question.

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

Workable Gage

I noticed that the workable gage for W8×24 and W8×28 in the 13th edition *Manual* changed from previous editions of the steel manual. The 13th edition shows the workable gage as 4 in. for those members; previous editions list them as 3½". Is this intended or was it a typo? Also, I was wondering if you can explain the difference between workable gage and useable gage as was listed in older U.S. Steel or ASD manuals.

Question sent to AISC's Steel Solutions Center

The historical flange gages for these W8s and other W-shapes, with flanges ranging from 6 to 8 in. wide, is 3½". A detailer changes this gage to 4" when a designer uses these sections as columns. Many fabricators just use 4" as a standard gage for these sections. This is one of those areas where "workable gages" is a flexible term. There isn't enough clearance to tighten bolts if one uses 3½" centers on framing angles.

The usable or workable gage is a dimension that allows sufficient clearance from the k_1 dimension (fillet) to the washer or bolt head to properly install the bolt. Fabricators generally group them so that only a few gages will work for all beams made. The usual range is 2¼", 2¾", 3½" (or 4") and 5½" (or 6"). The custom spacings come into play for extremely wide flanges and heavy framed connections. For framing into column flanges, 4" and 5½" gages were used traditionally, but 6" is replacing 5½" these days.

The use of 3½"gages comes from the need to fit seats into the webs of 8" columns. If you use 4" on the beam flange you couldn't fit the seat into the *T* dimension: 3½" + [(1¼" edge distance)(2)] = 6" seat width. Thus the workable gage varied depending on what you were trying to fit. If the section is used as a beam we use 3½"; if it is used as a column we use 4."

Ron Yeager
Steel-Art, Inc., Galeton, Pa.

Welding to 65 ksi Steel

Full penetration welded column splices of columns with 65 ksi yield strength were completed using 70 ksi weld metal, when 80 ksi weld metal was required. We realize something needs to be done as the weld no longer develops the strength of the base material. Can the columns simply be reinforced at the connection, i.e. reinforcing flange and web plates to resist for the excess not developed by the weld; or does the 70 ksi weld no longer have any analytical capacity because it used with 65 ksi material?

Question sent to AISC's Steel Solutions Center

My first thought is that it is not necessarily true that you need to do something to reinforce this splice. While you have not achieved the requirements to produce a splice that is "full strength," it is possible you have achieved the required strength. For example, if this is a column in a special moment frame, the required strength according to *Seismic Provisions* Section 9.9 is based upon a moment at the splice equal to R_yF_yZ for the column. With R_y for A913 Gr. 65 equal to 1.1, R_yF_y is 71.5 ksi, and your 70 ksi full thickness weld narrowly misses the required strength. The engineer could review the specific test report for the column in question and the test results on the certificates of conformance of the filler metal classification tests and find the strength of the weld sufficient to develop the strength of the column. Doing so is not strictly in accordance with the *Seismic Provisions*, but may be acceptable to the authority having jurisdiction. If this is not a special moment frame, a lower required strength may be applicable and a comparison like that above may be favorable.

If the analysis above is not successful, reinforcement is necessary. Fillet-welded splices with appropriate filler plates to make up the difference in column strength can be used to supplement the strength of the existing groove weld.

Tom Schlafly
American Institute of Steel Construction

Deep Column Section for SMF

I am designing a Special Moment Frame (SMF) in accordance with FEMA 350 and using a reduced beam section. In the past, and in the FEMA 350 document, the columns for SMF have been limited to W14s. However in a recent AISC seminar, the speaker indicated that it is acceptable to use deeper column sections (W24s, W27s, etc.) for SMF using the reduced beam flange connection (dog-bone). I could not find any documentation supporting this unless specific testing is required. Can you point me in the correct direction?

Question sent to AISC's Steel Solutions Center

The direction you are seeking is available in ANSI/AISC 358-05, *Prequalified Connections for Special and Intermediate Moment Frames for Seismic Applications*. This document is the successor to FEMA 350, and is available as a free download at **www.aisc.org/aisc358**. Section 5.3.1 of the document lists beam limitations where the beam depth is limited to W36 for rolled shapes. See the document for other prequalification variables.

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

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.

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

Steel SolutionsCenter
One East Wacker Dr., Suite 700
Chicago, IL 60601
tel: 866.ASK.AISC • fax: 312.670.9032
solutions@aisc.org