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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BOLT HOLE SIZES

Maximum hole sizes for bolts are specified in the 1999 *LRFD Specification* Table J3.3. What if an actual hole dimension is between two of the values?

Question sent to AISC's Steel Solutions Center

AISC LRFD Specification Table J3.3 is based upon Table 3.1 in the 2000 RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts* and contains the maximum dimensions of standard, oversized, short-slotted, and long-slotted holes. If an actual dimension exceeds the tabulated maximum by more than the 1/32-inch tolerance given in the RCSC *Specification*, it must be treated as the next larger hole size.

For example, a 13/16-in. by 1 1/4-in. slotted hole for a 3/4-in. diameter bolt must be treated as a long-slotted hole because it exceeds the maximum short-slotted hole size, which is 13/16-in. by 1 in.

Keith Mueller, Ph.D.
American Institute of Steel Construction
Chicago, IL

HSS AND STEEL PIPE SPECIFICATION

from October 2002

Can I take advantage of the 2000 *LRFD HSS Specification* for structural designs involving ASTM A53 Grade B steel pipe? If so, what are the differences between round HSS and steel pipe if both use the same design rules?

Question sent to AISC's Steel Solutions Center

According to Section 1.2 of the 2000 *LRFD HSS Specification* (available free-of-charge from www.aisc.org/freedownloads), various HSS materials and one steel pipe material are approved for use under the specification.

HSS are produced as ASTM A500, A501, A618 and A847 and include square, rectangular and round sections. ASTM A500 is the most commonly available and used, with Grades B and C having minimum specified yield strengths of 42 ksi and 46 ksi, respectively, for round HSS. Steel pipe should not be confused with round HSS, as pipe is produced to the requirements in ASTM A53 Grade B, which has a minimum specified yield strength of 35 ksi.

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

SHOP & ERECTION DRAWING STANDARDS

from October 2002

Does AISC or another organization publish specific standards or specifications for steel detailing and shop drawings? The drawings that I've seen coming from some of the new CAD software have not been consistent from one job to the next nor have they matched the clarity that good steel detailers produce by hand. NISD publishes guidelines for information to be shown by the design engineer but nothing on standards for what the detailers will provide. I'm looking for some good balanced standards to reference as minimum requirements for steel shop drawings that are submitted to us. Some detailers have advised me that the information is there in the software. So, how can I communicate my requirements up front, so that the advantages of electronic data transfer are realized and properly balanced with the need for clear record documents?

Richard A. Meloy, P.E.
Butler Heavy Structures
Kansas City, MO

You have hit upon a problem which is becoming pandemic in our industry today: users who think that the software they use *is* the industry standard. As a detailer, I look at software only for the drawings it produces and, if it does not come up to my standards, I was taught to reject it without consideration of any peripheral benefits. AISC has a book *Detailing for Steel Construction* that teaches the standards you seek. I have it on good authority that this publication will be updated and released shortly. That will prove to be the beginning of a long training process to shape the next generation of steel detailers. In the meantime, all you can do is insist on drawings produced to your standards and require that your detailers do so if they wish to continue working for you. Issue a sample set of drawings to bidders and tell them that these are it, no compromise. We produce drawings to the highest industry standards and not a single one is by hand. All come complete with all electronic data transfer you may need. There are many other detailers out there who can say the same.

Ronald Yeager
Steel-Art, Inc.
Galeton, PA

Editor's Note: The updated *Detailing for Steel Construction* should be available by the end of this month.

STEEL INTERCHANGE

TEES UNDER FLEXURE (STEM IN COMPRESSION)

from October 2002

How does one design a structural WT member under flexure when the stem is in compression? Chapter F of the 1989 ASD *Specification* does not appear to address this particular case.

When using LRFD, the current manual is straight forward for the design of tees. Both the equation for yielding strength and the equation for critical buckling strength of tees is shown in Chapter F of the AISC *Specification*.

An article written by William A. Milek in the 3rd quarter 1965 AISC Engineering Journal, under the title "One Engineer's Opinion," addresses this question. In this article, Milek uses an approximation of the results obtained by an exact solution for lateral buckling critical stress for members symmetrical about the y-y axis but unsymmetrical about the x-x axis to determine the allowable bending stress for tee sections, σ_{cr}. Of course, the allowable stress is limited by $0.6F_y$.

$$\sigma_{cr} = \frac{143 I_y d}{S_c L^2}\left[\sqrt{1+\frac{JL^2}{5.95 I_y d}} - 1\right] \times 10^6$$

It is also acceptable to use the equations of Section F1.3, excluding the equation (F1-8), to get a poor approximation of the allowable stress. Equation (F1-8) should not be used because it can be unconservative since there isn't a compression flange.

Greg Gertsen, P.E.
Albert Kahn Associates, Inc.
Detroit, MI

Section 9.12 of Salmon and Johnson's *Steel Structures Design and Behavior*, 4th edition, states that a tee whose stem is on the compression side of the neutral axis is similar to an I-shaped section bent in its weak direction. If the stem of the tee satisfies λ_p for an unstiffened flange $[0.38(E/F_y)^{0.5}]$ then it is acceptable to use the maximum moment strength M_n as high as M_p as long as the extreme fiber in tension does not exceed F_y. Note that $F_y Z < 1.5 M_y$. From a practical point of view, rolled structural tee webs will never satisfy this limit and web local buckling will control. For inelastic buckling, Q_s should be calculated per Appendix B of the LRFD Specification and M_r found as $Q_s F_y S_{xc}$. L_r can then be found and the problem solved as a normal beam problem. For elastic buckling, LRFD Equation F1-15 is still valid for stems in compression as long as B is taken as negative.

Will Jacobs, E.I.T.
Virginia Polytechnic Institute
Blacksburg, VA

NEW QUESTIONS

BOLTED HANGER-TYPE CONNECTIONS

The AISC 9th Edition (ASD) illustrates procedures for bolted hanger-type connections with a single line of resistance to prying action on each side of the hanging member. If each line of resistance consists of a bolt group, what design and analysis methods should be used?

Jay Shniderman, P.E.
Van Nuys, CA

PREVIOUS QUESTIONS

SQUARE TUBULAR SECTION ARCHES

(June 2002)

What reference material is available for the design of square tubular section arches with respect to in-plane and out-of-plane buckling? Loading may be full uniform, partial uniform, or concentrated load at quarter or half point. Arches are braced laterally at third points for typical spans. Most technical papers that I have reviewed are concerned with I shape sections although some of the analysis is transferable.

Susan Guravich, P. Eng.
Skarborn Engineering Ltd.

WEB PANEL-ZONE SHEAR

(March 2002)

In the 1992 and 1997 Seismic Provisions, for SMF, the resistance factor for panel-zone web shear is 0.75. The Seismic Provisions are somewhat silent for panel-zone web shear in OMF. LRFD Specification Section K1.7 using a resistance factor for panel-zone web shear of 0.90. For OMF, do we default to Section K1.7 and use a resistance factor for panel-zone web shear of 0.90? Or is the resistance factor always 0.75 in OMF and SMF if the loading is non-static?

Stephen Crockett
D. M. Berg Consultants, P.C.

CURVED STRUCTURAL MEMBERS

(September 2001)

Due to architectural characteristics, I am in the process of plan-checking a few moment frames using curved members (curved beams to column). I have questions regarding curved moment frames. Are they allowed in current codes? Has there been testing done on curved moment frames? In calculations, how do you design the beam-column connection? Will torsion be introduced in this connection and the frame members? If HSS beams and columns are used, please suggest a beam to column connection that will be acceptable.

David Chung, P.E.