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:

Steel SolutionsCenter
One East Wacker Dr., Suite 3100
Chicago, IL 60601
tel: 866.ASK.AISC
fax: 312.670.9032
solutions@aisc.org

SHEAR STUD REDUCTION FACTOR

The 1999 *LRFD Specification*, Section I3.5b states, *"Where there is only a single stud placed in a rib oriented perpendicular to the steel beam, the reduction factor of equation I3-1 shall not exceed 0.75."* Does this requirement apply to ASD design as well?

Question sent to AISC's Steel Solutions Center

Yes, this requirement applies to ASD. See *Supplement No. 1 to the ASD Specification*, a free download from www.aisc.org/freedownloads.

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

WEAK-AXIS BENDING OF CHANNELS

I am trying to clarify the appropriate specification section for the design of channels. Does 1989 AISC *ASD Specification* Section F2 apply to weak-axis bending of steel channels?

Question sent to AISC's Steel Solutions Center

Section F2 of the 1999 *ASD Specification* addresses weak-axis bending of I-Shaped members, solid bars and rectangular plates. However, it does not address channels.

The *LRFD Specification* has provisions for weak-axis calculations, provisions that are not found in the *ASD Specification*. For bending about the weak axis, the lateral-torsional limit state is not applicable. With thick, stocky flanges and webs thick enough, flange- and web-local buckling are not likely to control, in which case the channel can be designed for flexural yielding. In LRFD, $0.9F_yZ$ is appropriate.

If desired, the LRFD provisions can be converted by comparison to an equivalent ASD format. In ASD, this corresponds to $0.66F_yS$. If flange- or web-local buckling does control, the equations in Appendices B and F could apply.

Charlie Carter, P.E., S.E.
American Institute of Steel Construction
Chicago

SECOND-ORDER EFFECTS

When we performed the analysis of a frame, we found that our computed stresses were considerably higher than those computed by the design engineer. Upon further investigation, we found that these higher moments were due to second-order effects. The design engineer claims that he is not required to perform a second-order analysis under the *ASD Specification*. Is this correct?

Question sent to AISC's Steel Solutions Center

The 1989 AISC *ASD Specification* requires consideration of second-order effects, as it has since the introduction of the 1961 version of that specification. The *ASD Specification* does not require that a second-order analysis be performed but instead uses a simplified amplification of the first-order analysis to accomplish that goal. This can be found in Equation H1-1 of the 1989 AISC *ASD Specification* where axial and flexural stresses are combined. The computed first-order bending stress is amplified by a factor to account for the second-order effects.

Louis F. Geschwindner, Ph.D., P.E.
American Institute of Steel Construction
Chicago

FILLET WELD STRENGTH

Are fillet welds stronger when loaded transversely than when loaded longitudinally?

Refer to FAQ 8.3.1 on the AISC website at www.aisc.org/faq, as summarized below:

Yes. This long-known variation in strength as a function of load angle is now formally recognized in the 1999 AISC *LRFD Specification*, Appendix J2.4. The maximum strength increase permitted therein is 50 percent, which occurs for a load perpendicular to the fillet weld. When the load angle is intermediate between longitudinal and transverse, the strength increase will vary between 0 and 50 percent, respectively.

Bill Liddy
American Institute of Steel Construction
Chicago

HOT-DIPPED GALVANIZED BOLTS

Are there any special requirements if we decide to use hot-dipped galvanized ASTM A325 Type 1 (medium carbon) bolts?

Question sent to AISC's Steel Solutions Center

STEEL INTERCHANGE

Galvanized ASTM A325 high-strength bolts must be considered as a manufactured fastener assembly. Refer to AISC FAQ 6.2.3 on the AISC website at www.aisc.org/faq.

Four principal factors must be considered for a hot-dipped galvanized bolt and nut assembly:

(1) The effect of the hot-dip galvanizing process on the mechanical properties of high-strength steels;

(2) The effect of over-tapping for hot-dip galvanized coatings on the nut stripping strength;

(3) The effect of galvanizing and lubrication on the torque required for pretensioning; and,

(4) Shipping requirements.

Refer to the *Commentary* found in Section 2.3.3 of the 2000 RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts* (available as a free download from www.boltcouncil.org) for an expanded discussion of these requirements. Please note that the application of zinc to ASTM A490 bolts by metallizing or mechanical coating is not permitted because the effect of mechanical galvanizing on embrittlement and delayed cracking of ASTM A490 bolts has not been fully investigated to date.

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

UNPAINTED STEEL

What surface preparation should be specified for steel that is to remain unpainted?

Question sent to AISC's Steel Solutions Center

Refer to FAQ 10.3.1 on the AISC website at www.aisc.org/faq, as summarized below:

Steel that is to remain unpainted need only be cleaned of heavy deposits of oil and grease by appropriate means after fabrication. If other considerations dictate more stringent cleaning requirements, an SSPC-SP2 or other appropriate grade of cleaning should be specified in the contract.

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

UNAUTHORIZED SHOP SPLICES

Can fabricators shop splice scrap shapes to make them longer? Are these splices regarded as connections to be approved by the EOR on the shop drawings?

Question sent to AISC's Steel Solutions Center

Any such splice requires the approval of the Engineer of Record.

The 2000 AISC *Code of Standard Practice for Steel Buildings and Bridges* states that the Fabricator must obtain approval from the Owner's Designated Representative for Design if they select or complete the Connection details, as specifically stated in Section 3.1.2. And the Code defines a Connection as:

"An assembly of one or more joints that is used to transmit forces between two or more members and/or connection elements."

Additionally, according to AWS D1.1-02 Section 6.5.1 *"The inspector shall make certain... that no unspecified welds have been added without approval."*

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

Back a few years ago, structural steel was delivered to two or three school projects with shop splices in main members that were neither on the approved shop drawings nor approved by the SER. At the time I found language in one of AISC's publications that said something like "shop splices in main members should not be made without the approval of the SER".

According to the AISC FAQ 2.5.2, found online at www.aisc.org/faq:

"When material is short of the minimum required length, welded splices or deposited weld metal, when applied with appropriate welding procedures and specified material, should be permitted with the approval of the SER."

It seems to me that this advice applies to any unauthorized shop splice in main members. In one of the above school cases, the erected spliced members were removed and re-fabricated. In another, the SER required field testing of all the welded splices to ensure proper quality.

Very simply, it is not good practice to add splices without the SER approval. SERs do not like surprises in the field!

Emile Troup, P.E., Consultant
Structural Steel Fabricators of New England
Canton, MA

EXTENDED END-PLATE CONNECTIONS

from November 2002

Symmetric tension bolt pitches are assumed in the published design procedures for this connection. However, due to ease of fabrication, we would like to use a different pitch above and below the top tension flange of the beam. Are there guidelines on this, or has this connection been prequalified for only symmetric pitches?

AISC's *Design Guide No. 16: Flush and Extended Multiple-Row Moment End-Plate Connections* contains design examples for cases of extended end-plates with unequal pitched tension bolt rows. See Chapter 4 of this particular design guide for additional information.

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

Steel SolutionsCenter
Your connection to ideas + answers
866.ASK.AISC
solutions@aisc.org