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 at:

Steel Interchange
Attn: Keith A. Grubb, S.E., P.E.
One East Wacker Dr., Suite 2406
Chicago, IL 60601
fax: 312/670-0341
email: grubb@blacksquirrel.net

Welding Older Steel

I have an existing steel frame building (1962) to be retrofitted with steel braces. Some of the existing columns are A440 steel. I believe this grade was discontinued in the 1970's. The AISC steel manual sixth edition simply states that the steel is not recommended for welding. Most of the existing connections to the columns are bolted. How do I specify welding criteria for this steel?

You'll have to evaluate the modern-day weldability of this steel. It may be a question of whether it is possible and practical (or not). The following reference from AISC's Engineering Journal takes a comprehensive look at welding requirements for older structural steel: David T. Ricker, "Field Welding to Existing Structures," First Quarter 1988. Call 312/670-2400 and ask for *Engineering Journal* reprints to obtain a copy.

Slip-Critical Galvanized Surface Preparation

As required in RCSC Specification Section 3(b)(5), galvanized surfaces in slip-critical connections must be roughened by means of hand wire brushing. I have heard one definition of the proper degree of wire brushing required as "an amount to visibly alter the surface without disrupting the continuity of the galvanizing".

Can anyone provide a more quantitative description of the degree of wire brushing required to satisfy the RCSC specification for the use of slip-critical connections?

There is information on the subject in the AISC technical FAQ at www.aisc.org, but as you probably already know, the FAQ states essentially what you quoted already. There is more detailed discussion from a research perspective of the roughening required in the *Guide to Design Criteria for Bolted and Riveted Joints* by Kulak, Fisher and Struik.

The untreated galvanized surface would get you a slip coefficient on the order of 0.2. To get the higher value of 0.35 (recently changed from the 0.4 that was in the previous RCSC Specification), roughening is required. RCSC gives what I think is a fairly practical and performance-oriented requirement: hand wire brush the surface until it is visibly altered when compared to the untreated galvanizing around the area that's brushed.

Charles J. Carter, S.E., P.E.
American Institute of Steel Construction
Chicago, IL

Notional Loads

1. **In order to take column buckling lengths equal to unity and use AISC-ASD design procedures, can we use P-Δ analysis taking notional forces into account arising from imperfections?**
2. **Shall notional forces be included in the P-Δ combination in conjunction with dead and live loads?**
3. **If buckling lengths are taken as unity with an appropriate analysis, AISC requires max KL/r as 200 for columns, how can this be checked as taking $K = 1$, or is this requirement no longer valid?**

An excellent reference that will give you guidance on this is ASCE's *Effective length and Notional Load Approaches for Assessing Frame Stability: Implications for American Steel Design*. See more about it at www.pubs.asce.org. Although second-order analysis and column buckling are related, it does not necessarily follow that K can be taken as unity when a second order analysis is performed. Some cases where K can be taken as unity are outlined in the aforementioned publication.

Second-order analysis and ASD do not go well together, because the load level will affect the outcome. To properly address second-order effects in terms of frame deformations (P–Δ) and member deformations (P–δ) in ASD, I think you'd have to do your second-order analysis at the factored-load level and then unfactor the results to compare to the allowable stresses.

Question 2: Simplifying things far too greatly, the notional load method relies upon the use of notional or fictitious lateral forces on the frame in addition to the other loads present. Since you are trying to account for the increased column moments due to displacement of the frame and the vertical loads acting through those displacements, I believe the answer to your question is yes.

Question 3: The recommended *Kl/r* limitation is still applicable. From AISC LRFD *Specification* Commentary Section B7, this recommended limit is "based on professional judgment and practical considerations of economics, ease of handling, and care required to minimize inadvertent damage during fabrication, transport and erection." It is further indicated that this requirement is not strength related.

Charles J. Carter, S.E., P.E.
American Institute of Steel Construction
Chicago, IL

Slotted HSS Connections

For a slotted HSS/gusset plate connection...AISC recommends that the length of the weld shall be equal to or greater than the O.D. of the pipe. My interpretation is that this is a recommendation and not a definite requirement and that a shorter length may be used provided the numbers add up.

Also, Cheng and Kulak (AISC *Engineering Journal*, Fourth Quarter 2000) suggest that "...the stiffening effect caused by the gusset plate precludes shear lag fracture as long as the weld length is 1.3 times the pipe diameter..." Any chance that AISC will reexamine this?

There is a requirement in AISC *Specification* section J2.2b that longitudinal fillet welds for flat-bar tension members must be at least as long as the distance between them.

If you think of HSS and steel pipe as curved plates, you'll see the basis of our recommendation in the HSS Connections Manual. However, by the letter of the law, if you address shear lag effects in the tension rupture calculation, and all the other applicable limit states, you might be able to justify a shorter connection length. Note that the other general provisions for minimum and maximum weld lengths apply too.

Regarding the Cheng and Kulak paper, AISC is looking at that right now for the 2005 AISC Specification (the Unified Specification). Personally, I think the Kulak paper presents great work and a very positive and practical modification that can be made to our provisions for shear lag.

Charles J. Carter, S.E., P.E.
American Institute of Steel Construction
Chicago, IL

Lateral Support for Beams

For a beam, fully braced along its top flange (such as a roof or floor beam supporting a deck), subjected to axial compression and bending along its major axis, what is the unbraced length of the beam along its weak axis? I know its somewhere between zero and the full length of the member, but is there a standard rule of practice for weak axis unbraced length for this situation?

Robert Brodowski

As mentioned in Chapter F of the LRFD Specifications, the flexural strength of a beam bending about its strong axis is governed by such limit states as yielding, lateral-torsional buckling (LTB), flange local buckling, and web local buckling. For fully laterally braced compact beams, yielding is the only applicable limit state (since local buckling is not an issue for compact beams).

For beams bent about their minor axis, LTB simply does not apply. Thus, the "unbraced length" of a beam along its weak axis is not of consequence. For compact members, yielding is the only flexure limit state that need be checked for minor-axis bending.

For beam-columns, the story is somewhat different. For a beam-column that has one flange braced (by a floor system, for example) and the other flange unbraced, the design isn't so simple. However, this situation is covered in a paper by Joseph A. Yura called "Fundamentals of Beam Bracing" in the First Quarter 2001 *Engineering Journal* (soon to be available). Call 312/670-2400 for *Engineering Journal* reprints.

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