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

Snug-tightened Bolts Make Sense

From AISC 7th edition *Manual*, the preferred method of tightening structural bolts for bearing connections was the turn-of-nut method.

Looking at current references, the method for bearing connections is called "snug tight." I prefer the turn-of-nut method and wonder if requiring it is an undue expense for the erector. What kind of cost difference is there between the two methods?

A lot has changed since the early 70s when the 7th Edition was current, especially in the area of bolted design and construction.

When high-strength bolts were first introduced in the 1950s, it was required that they be pretensioned regardless of the application in which they were used. This had more to do with protectionism than science. Rivet manufacturers were afraid bolts would eliminate the use of rivets. The concession that got bolts in the code at the time was that they always be pretensioned. Apparently rivet manufacturers had reason to worry. Anybody still know where to find a rivet installation procedure today?

Slowly and steadily, the industry has liberalized the unilateral requirement of pretension. Snug-tightened installation is now allowed for the majority of bolted joints. In very specific applications, pretension is required. This is all explained very well (I think, anyway) in the 2000 *RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts*, posted for free download on the RCSC web site:

www.boltcouncil.org

There is definitely a cost advantage to specifying snug-tightened joints when you can. It makes design easier. It makes installation easier. And it makes inspection easier. And snug-tightened joints have withstood the test of time. I don't have a dollar figure to quote you on the cost difference, but it is worth it to use snug-tightened joints.

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

Surface Finishes

What is the difference between calling out "tight fit," "grind to fit," and "grind to bear." They all sound the same to me, but I've seen details where more than one is called out.

Michael LeComte
Washington Group International
Deerfield Beach, FL

First, let's assume that the details where these notes occur are bearing details of some sort. The AISC LRFD *Specification* Section M2.6 states, "compression joints which depend on contact bearing ... shall have the bearing surfaces of individually fabricated pieces prepared by milling, sawing, or other suitable means." The commentary to AISC's 2000 *Code of Standard Practice* Section 6.2.2 states that "Most cutting processes, including friction sawing and cold sawing, and milling processes meet a surface roughness limitation of 500 per ANSI/ASME B46.1.

Because grinding and/or milling are seldom necessary, a clearer note on the drawings would be "finish to bear" and leave the options up to the fabricator.

Keith A. Grubb, P.E., S.E.
Chicago, IL

Second Order Analysis

Consider a three dimensional four story building model with some unbraced frames in both directions (the rest are leaning columns) in which all the gravity load per floor is considered in calculating the second order (destabilizing) effects. Building codes require designers to include second order effects (structure instability and local instability) in the analysis of steel and concrete frames. ACI-318 accepts second order analysis for the structure instability (many software packages can do this) and provides a test for the designer when to test a member for local instability. AISC/ASD is silent on this topic. Researchers recommend to use $K = 1$ in the

interaction equations when second order effects are included in the analysis. However, engineers use $K = 1$ or the value obtained from the nomographs. What are your thoughts on this subject?

Nicholas Constantine
Korda/Nemeth Engineering
Columbus, OH

AISC Allowable Stress Design is not silent on the issue of second order analysis. Section A5.3 states "Selection of the method of analysis is the prerogative of the responsible engineer." However, Section C1 states, in part, "frames...shall be designed to provide the needed deformation capacity and to assure overall stability." The ASD column interaction equation, H1-1 includes, in the bending term, the amplification factor for second order moment effects. It is permissible to use a second order analysis, carried out at factored loads, to determine the second order moments in place of the amplification factor used in the interaction equation.

On the question of effective length factor, the use of a second order analysis does not remove the need to determine the critical buckling capacity of the axially loaded column and thus, the effective length factor. Nor does the second order analysis account for the reduced stiffness of the restraining columns due to the existence of any "leaning" columns. Since there are numerous approaches to carrying out a second order analysis, designers should use care and not simply accept analysis results without understanding the method being used.

A useful reference might be the 2000 T. R. Higgins Lecture, "A Practical Look at Frame Analysis, Stability, and Leaning Columns" by Louis F. Geschwindner, Proceedings, NASCC, AISC, Las Vegas, NV, 2000.

Louis F. Geschwindner, P.E.
Professor of Architectural Engineering
Penn State University

Special Truss Moment Frames

I am looking for design aids for special truss moment frames. Can you recommend any recent publications?

Stephan Schambeck, P.E.
PDC Consulting Engineers Inc.

A very good reference is called "Special Truss Moment Frames Design Guide" by S. Goel, D. Rai, and H. Basha, research report UMCEE 98-44, from the Department of Civil and Environmental Engineering at the University of Michigan, Ann Arbor, MI.

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

Technical Note: Specifying Anchor Bolts

ASTM A325 and A490 are for steel-to-steel structural bolting only, not steel-to-concrete anchorage, because the y have special heading and threading requirements and are generally available in lengths up to 8 in. only. ASTM A354 is the strength level equivalent of ASTM A490 in a rod specification with more general heading, threading and other requirements.

Take a look at ASTM F1554, which is a relatively new material specification that covers hooked, headed and threaded and nutted anchor rods in three strength grades: 36, 55 and 105.

Grade 36 is most commonly specified, although grades 55 and 105 are normally available when higher strength is required. ASTM F1554 grade 36 or, if its availability can be confirmed prior to specification, ASTM F1554 grade 55 with weldability supplement S1 and the carbon equivalent formula in ASTM F1554 Section S1.5.2.1 can be specified to allow welded field correction should the anchor rods be placed incorrectly in the field. ASTM F1554 grades 36 and 105 are essentially the anchor-rod equivalents of the generic rod specifications ASTM A36 and A193 grade B7, respectively. ASTM F1554 grade 55, when specified with the weldability supplement, is similar to an ASTM A572 material that is intermediate between grades 50 and 60.

Several other ASTM Specifications can also be used. For applications involving unheaded rods, ASTM A36, A193, A307, A354, A449, A572, A588 and A687 can be specified. For applications involving headed rods, ASTM A307, A354 and A449 can be specified.