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: 312.670.2400
fax: 312.423.4651
solutions@aisc.org

SEISMIC DESIGN OF BASE PLATES

Shall I use the load combinations (shown below) with amplified earthquake load (Eqs. 4-1 and 4-2 of the 1997 AISC *Seismic Provisions for Structural Steel Buildings*) for the design of column base plate and anchor rods?

$$1.2D + 0.5L + 0.2S + \Omega_0 Q_E$$
$$0.9D - \Omega_0 Q_E$$

Mike Ginsburg, P.E.
Leo A. Daly
Omaha, NE

The answer to the question depends on the expected system performance of the structure. In general the System Overstrength Factor, Ω_0, is prescribed to assure sufficient design strength to allow selected members to yield. When a Building Code requires design of a connection in accordance with Special Load Combinations that include the System Overstrength Factor, the intent is to assure that the connection is strong enough and stiff enough to allow yielding of the member.

In the case of moment frames, if the building system performance intends column yielding at the base plate, the connection between the column and the base plate should be designed for the System Overstrength Factor.

In the case of braced frames, if the building system performance intends brace yielding at the base plate, the connection between the brace and the base plate should be designed for the System Overstrength Factor.

In all cases, the design of the anchor rods should consider ductile behavior, without using the System Overstrength Factor. In other words, the system performance of all frames will be enhanced by designing anchor rods to yield before they "pull out" of the concrete. Designing anchor rods to yield requires sufficient concrete embedment to preclude concrete "shear cone" failure. The top half of the anchor rod should also have a bond breaker to increase the length of rod that will strain.

Rick Drake, S.E.
Fluor Daniel
Aliso Viejo, CA

SEISMIC DESIGN

What is the difference in design philosophy between a building structure that has been designed to meet the AISC LRFD *Specification for Structural Steel Buildings* and a building that has been designed to meet the AISC *Seismic Provisions for Structural Steel Buildings*?

Frequently asked question on AISC website, www.aisc.org/faq.html

A building designed to the *AISC LRFD Specification for Structural Steel Buildings* is one that possesses adequate strength to resist all design loads, primarily through nominally elastic behavior. A building designed to the AISC *Seismic Provisions for Structural Steel Buildings*, contains additional provisions for dissipating large magnitude seismic input energy through controlled inelastic deformations in discreet locations in the structure, such as through hinging of beams in moment frames, buckling of braces in concentrically braced frames, and shear (or flexural) yielding of the link in eccentrically braced frames to preclude structural collapse under high overload conditions that may occur. Obviously, a higher cost is associated with designing to the latter specification and achieving this level of ductility.

Answer given on AISC website, www.aisc.org/faq.html

CAMBERING EQUIPMENT

from March 2001 Steel Interchange

Who manufactures equipment for cambering steel?

Cheryl Vickroy
Madison, WI

Bay-Lynx Manufacturing of West Ancaster, Ontario, CANADA (Phone: 888.337.3331, www.bay-lynx.com) manufacturers the Cambercat Cambering Press. The distributor is Peddinghaus Corporation from Bradley, Illinois (Phone: 815.937.3800, www.peddinghaus.com). The contact people are David Hamann or Bud Panick. The Cambering Press capacity range is from 333 ton to 684 ton and there are over 45 presses in operation across North America.

Tim Verhey, M.Eng., P.Eng.
Walters Inc.
Hamilton, ON, Canada

Cambering hot rolled wide flange shapes is most commonly accomplished by one of 2 methods. The older method to camber is called heat cambering and is accomplished by heating triangular shapes on the section opposite the up side of the camber. The only equipment necessary is an oxygen acetylene torch with a "rosebud" tip. The most common method of cambering in steel fabrication shops today is hydraulic cambering. Companies like CAMBCO and Peddinghaus manufacture hydraulic cambering machines.

Harold Sprague
Black & Veatch
Overland Park, KS

I used to work for a company in Albany, GA. We custom built any type manufacturing equipment needed for the steel fabrication industry. The following is the information needed to get in touch with the company:

Fabrication Solutions, Inc.
1132 Gillionville Road
Albany, GA 31707
Contact: Vann Ditty, 229.483.0170

We custom built a cambering unit for Addison Steel of Orlando, FL. This unit could camber up to a 42" beam. It had two 300 ton hydraulic cylinders and could handle most anything you put in it.

Andy Hicks
Project Manager / E&H Steel Co.
Midland City, AL

DRAFTING GUIDELINES

from September 2000 and February 2002 Steel Interchange

Are there any standards or guidelines for structural steel drafting, particularly for the presentation and content of design drawings? I am interested in finding "rules" about showing the weights of beams on plans and column weights on elevations, etc. Each company seems to have developed its own drafting convention, but I would like to know if there are industry-wide standards.

Brian W. Bersch, P.E.

Beam size callouts are primarily shown on plans, i.e., W24x68. Beams between floor levels shown on the elevations and not reflected on plans are likewise labeled on the elevation, i.e., W21x57. Beams shown on elevations but have already been called out on plan are called out by only the depth designation, i.e., W21, and the duplication of a full callout of a member shown on the plans is generally avoided. On sidewall elevations column size callouts are typically shown, i.e., W14x109. On endwall elevations where a corner column is duplicated in that view, only the depth designation is called out, i.e., W14, and the duplication of a full callout of a member shown on a previous elevation is generally avoided. The general rule is that full callouts are shown in primary view (PLAN) and/or the first view (ELEVATION) that the member is shown. If the member is shown in subsequent views (other ELEVATIONS), only the depth designation should be used, i.e., W14, and any member having more than ONE full callout should be avoided. This is also true of details; members are identified only by depth designation, i.e., W12, C10, WT5, etc., thus not duplicating the full callout of the member. The rationale is to provide a consistent, uniform method for member callouts. Also, should member sizes change, only ONE callout for a member needs to be updated, thus saving time. If member callout existed for each member in multiple views and one of those callouts was missed during a member size change the drawings or views would immediately be in conflict and cause confusion.

Cal Graham
JHI Engineering
Portland, OR

Section 3 of the AISC *Code of Standard Practice* provides a "laundry list" of the information needed. It does not provide the conventions for showing such information, however. Any convention that works should be permitted. That said, the Council of American Structural Engineers (CASE) is currently working on improving the quality standards for design drawings. Some standard may result from that effort.

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

BACKING BARS

When should backing bars and run-off tabs be removed after welding?

Frequently asked question on AISC website, www.aisc.org/faq.html

To produce sound welds on many welded joint geometries, run-off tabs projecting from the finished member may be required to permit starting and stopping welds beyond the edge of the member; AWS D1.1 Sections 5.10 and 5.31 should be followed. Additionally, AISC LRFD *Specification* Section J1.5 addresses requirements for the removal of backing bars and weld tabs at complete-joint-penetration groove welded splices in ASTM A6 Group 4 and 5 rolled shapes and plates exceeding 2 in. thickness subject to primary tensile stresses. When such welding aids are required to be removed, the surface should be finished as indicated in 2.2.6 and 2.2.7.

Damage to welded beam-to-column-flange moment connections in the 1994 Northridge earthquake has raised several welding and seismic detailing issues and new criteria have been established. Explicit requirements for the removal of back-up bars and run-off tabs in seismic projects have been included in the AISC *Seismic Provisions* (AISC, 1997). An exception is included for tested assemblies that can be demonstrated to have acceptable performance with alternative treatments.

Answer given on AISC website, www.aisc.org/faq.html