STEEL INTERCHANGE

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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SEISMIC DESIGN OF BASE PLATES

Question reprinted from April 2002 Steel Interchange:

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

Answer reprinted from April 2002 Steel Interchange:

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

I was surprised (and slightly concerned) to see in the April 2002 *Steel Interchange* a recommendation from Rick Drake that "The top half of the anchor rod should also have bond breaker to increase the length of rod that will strain". I am not familiar with this practice. Is this something new? This was part of a discussion on the seismic design of base plates. Is this practice only related to the seismic design and only recommended in high seismic areas—or should my 2½" diameter stack anchors that were installed last year have been debonded at the top? Is this "should" a "good idea" or a code requirement?

Kenneth S. Peoples, P.E.
Lehigh Valley Technical Associates
Northampton, PA

The concept is very common in structures that routinely require pretensioned anchor rods as is done in the tank and petrochemical industry. Rick Drake has a lot of experience with the tank and petrochemical industry, and he understands the principles of basic bolt pretensioning well.

A good parallel can be drawn to anyone familiar with the concrete post-tensioning industry. Back in the 1970's when unbonded PT was first being used here in the US, they used a greased strand with kraft paper. It would bond at various points along the length that was unintentional. When they started tensioning the strands the inadvertent bonds would break. The jacks would blast off the seats about 2 or 3 feet then slam back against the seats breaking kneecaps and soiling shorts. This gave rise to two things: An OSHA requirement to not stand behind the jacks and a plastic sheath in lieu of the kraft paper on the strand. The lesson here is that the bond along the shaft of an anchor rod is not reliable.

If the anchor rod is NOT bonded along the shaft, the tension load is more reliable because you are totally relying on the head of the anchor bolt in bearing. With the shaft unbonded, the entire length of the shaft can be pretensioned more uniformly.

Harold O. Sprague
Black & Veatch

RESTRAINED CONDITION

One of the issues in determining fire resistive properties per ASTM E 119 is whether the members are considered restrained. It is my understanding that the steel

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industry has decided that all moment connections result in a restrained condition. Does anyone know if there is any documentation on this?

Question posted on SEAINT list server

This issue has been treated similarly, but with differing extents of information throughout the various building codes, ASTM documents and the UL *Fire Resistance Directory*. In some documents, ASTM E119 Table X3.1 has been incorporated directly, in others it is referenced. AISC has included it (reprinted with ASTM permission) in the 3rd Edition *LRFD Manual* (see page 2-47) with other fire resistance information in part to bring some consistency to the available information. ASTM E119 Table X3.1 classifies all steel construction (except some wall-bearing joist systems, which may not have resistance to thermal expansion) as restrained. Note that, in this application, restrained means resistance to thermal expansion, not connection restraint in the flexural or rotational sense.

There is also a paper in the 2nd Quarter 2001 AISC *Engineering Journal* that goes into more depth on this subject authored by Emile Troup, Consultant, Canton, MA and Dick Gewain of Hughes Associates, Baltimore, MD on the subject. It is available on the web at:

www.aisc.org/ContentManagement/ContentDisplay.cfm?
ContentID=2796

and a paper from the 2002 North American Steel Construction Conference by John Ruddy and Socrates Ioannides of Structural Affiliates International in Nashville, TN here:

http://www.aisc.org/Content/ContentGroups/Modern_Ste
el_Construction3/April_2002/2002_04_fire.pdf

and a transcript of an AISC online chat session with them here:

http://www.aisc.org/Template.cfm?Section=Online_Chat
&Template=/ContentManagement/ContentDisplay.cfm&
ContentID=3282

Prescriptive fire protection will probably always be with us — hopefully with less confusion and misinformation out there. You have to watch out for fire protection salespeople and people with other competing interests because they would like to cloud the issues significantly on steel fire protection, particularly on the restrained/unrestrained issue.

The future of fire design is likely in performance-based design as outlined here, however:

http://www.aisc.org/ContentManagement/ContentDis-
play.cfm?ContentID=626

Charles Carter, S.E., P.E.
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SHEAR LAG

Question #6 in the March 2002 *Steel Quiz* stated that the reduction coefficient U for shear lag is applied to the net area of bolted tension members and to the gross area of welded tension members except for HSS members with slots for gusset plates. What is meant by this exception?

Question submitted anonymously

Generally speaking, shear lag must be considered for HSS with slots and welded to gusset plates. The "except" in the *Steel Quiz* statement is that HSS strength is calculated using net area (deducting for slot widths) instead of using gross area of the member.

Tom Ferrell
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Birmingham, AL

Editor's Note on Shear Lag:
The reader is encouraged to review "Gusset Plate Connection to Round HSS Tension Members" by Cheng and Kulak (Engineering Journal, 4th Quarter 2000). This article discusses when the shear lag coefficient U *can be taken equal to 1.*

NEW QUESTION

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.
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