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. If you have a question or problem that your fellow readers might help you to solve, please forward it to *Modern Steel Construction*. At the same time, feel free to respond to any of the questions that you have read here. Please send them to:

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Chicago, IL 60601-2001

Answers and/or questions should be clearly presented. E-mail submittals and/or e-mail attachments are welcome.

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**** **Questions and answers can now be e-mailed to: grubb@aiscmail.com** ****

According to the August, 1999 Steel Quiz, plug welding a less-than-fully-threaded nut to an anchor rod is not an effective means of attachment. I would like to back up this recommendation with something more substantial than this reference from the Steel Quiz in *Modern Steel Construction*. Can anyone guide me to research, articles, codes, or books that deal with this topic in detail?

Frank S. Griffin, Jr., EIT
Fort Worth, TX

Item 7.1.5 on pp. 50-51 of AISC's *A Guide to Engineering and Quality Criteria for Steel Structures: Common Questions Answered* deals with anchors that come up short. The information in that publication is based upon the collective judgement of the AISC Committee on Manuals and Textbooks. We are not aware of any more specific requirements.

The recommendation that one not weld short anchor rods to nuts stems from a welding problem: there is no prequalified joint or welding procedure specification for making this weld. A possible indirect prohibition may be found in the surface condition requirements specified for surfaces onto which weld metal will be deposited in AWS D1.1: the exposed rim of a threaded nut won't pass. Also, into what category does the A563 nut material fall for filler metal selection and other welding issues?

The popularity of this "fix" stems from the assumption that "filling the hole" with weld metal either a) effectively extends the anchor rod and thus fully engages the threads of the nut; or b) prevents the rod from pulling out of the nut, thus developing the strength of the rod. No published research, test data, or analyses are available to substantiate these assumptions.

An ounce of prevention is worth a pound of cure: construction personnel must not wait until the steel is erected to concern themselves (and then the designer) with the issue of anchor rod placement.

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

In table B5.1 of the 9th ed. *ASD Manual*, footnote *e* defines a compression element restraint coefficient, k_c. This expression includes the ratio h/t. What definitions of h and t are used in this equation? While I think that h is well defined, the value of t is open to interpretation as either t_w or t_f.

I have been given different interpretations of both h and t by different people; however, I think that h is the clear distance between the flanges (height of the web) and t is the thickness of the web ($t = t_w$). This interpretation appears to be backed up by the Example 12 on p. 2-220 of the 9th ed. However, it does seem unusual that the local buckling capacity of a flange is related to the stiffness of the web. Can someone please clarify this?

Andrew Abbo
Formation Design System
Fremantle, Western Australia, Australia

The variable h is the clear depth of the web. Strictly speaking, this is the distance from toe of fillet at one flange to toe of fillet at the other flange. But h can be conservatively taken as the depth between flanges if you don't want to mess with the fillet radius. The variable t is the web thickness.

The web does affect the buckling strength of the flange for an I-shaped member. If the flange is to buckle, it does so in a rotational mode (when one flange tip goes up, the other has to go down.) The web is along for the ride and will provide some resistance, depending upon its relative stiffness.

Charles J. Carter, P.E.
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Chicago, IL

Is there a limitation on the thickness and type of coating on the faying surfaces of a connection for developing the full strength of a bearing type connections with A325 bolts? The issue is particularly critical when shop applied fire protection coatings must be

applied in two coats (for a thickness of 0.31 inches) to provide a two hour fire rating. The current practice appears to be to block out the connections from shop coating, and field coat them after the erection of the steel is completed.

Ray Krishnan
Bechtel Inc,
Houston, TX

Take a look at the 1993 LRFD Specification, Chapter J, Section J6 - Fillers. There is an equation given for reducing the shear strength of a bolt when "filler" material between the plies exceeds 1/4" but is less than 3/4". To me, this leaves no doubt that adding space in between the plies impacts the bolt capacity. Also, take a look at the RCSC Specification for Structural Joints Using ASTM A325 and A490 Bolts (1994). Section 3 (a) states that all material within the grip of a bolt shall be steel, with no compressible material. Although paint is allowed on the faying surfaces of the connection per Section 2 (g), I think your coating is far too thick to qualify as simply "paint."

Keith A. Grubb, P.E.
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We have been wondering whether the "ends" of simple shear tabs need to be wrapped with a fillet weld when the weld symbol shows a typical fillet weld on both sides of the shear tab. A similar question has arisen with WF column stiffeners located at the top and bottom of a connection shear tab (not part of the lateral system), which are welded with fillet welds as opposed to full penetration groove welds. Does the weld need to continue across the end or edge of the stiffener where it is flush with the column flange?

Ron J. Allen
Western Steel Manufacturing Co.
Boise, ID

There are several issues to consider. Grab your copy of these two references:

• AWS D1.1-98 (section 2.4.7; changed in '98 version)

• AISC LRFD Specification Supplement No. 1, 1998 (section J2.2b; agrees with AWS 2.4.7)

Because of concerns about leaving notches in the top of the shear tab or on the edge of a flange, the new specifications *recommend* that you stop the weld one weld size short of the edge. The 1989 ASD Specification required that the welds stop short, although there are cases where closing the weld may be appropriate. For example, if the job is painted or galvanized, you may need to seal the ends of the welds. In this case, the engineer may decide to wrap the weld in spite of the notching concerns. The "stop-short" requirement was relaxed in the newer specifications to be more flexible.

Two other issues come to mind in your column stiffener weld. First, the toughness in the "k-area" of the W-shape is a concern. It is generally recommended that you keep welds away from the fillet of the columns.

Another concern is that welds can be wrapped to hide poor fit-up. Poor fit can reduce the throat of a fillet to nothing very quickly, so wrapping is not encouraged.

Thomas J. Schlafly
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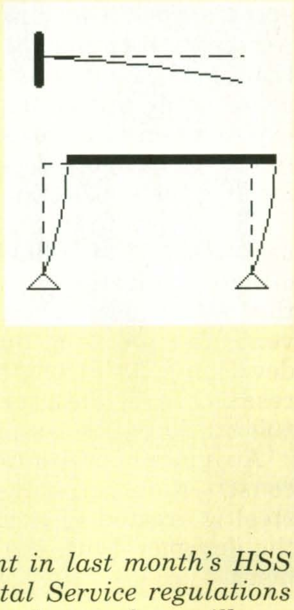
New Questions

1. Is the C_m value for a beam in a moment frame based on the sidesway behavior of the frame? Specifically, should the beam be considered subject to sidesway, resulting in C_m=0.85 per the 9th ed. ASD Manual, or does the sidesway only apply to the columns whose ends can translate relative to each other?

2. AISC's 1989 ASD Specification, Chapter F, states that C_b can (should) be taken as unity for cantilevers. Does this apply to columns of moment frames with pin supports? It appears as though the deformed shape, moment diagram, etc. are identical in the cantilever and the column (see figure). In one case the tip deflects, in the other case the support translates.



Raoul Karp
RAM International
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Editor's Note:
I missed an important point in last month's HSS Mailbox feature: U.S. Postal Service regulations generally require "break-away" posts for mailboxes in the event they are hit by a vehicle. Check with your local post office for specific requirements.