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 at:

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Single Plate Connections

Volume II of the LRFD *Manual* (Metric Conversion of the 2nd edition) states on p. 9-150 that the limit states for Bolted/Welded Single-Plate Connections are bolt shear, bolt bearing on (connection) plate, shear yielding of the (connection) plate, shear rupture of the (connection) plate, and block shear rupture of the (connection) plate. There is no mention of flexural yielding of the connection plate.

My calculations verify that flexural yielding of the connection plate is not included in the Table 9-10 values. For instance the Table 9-10 value for two A325M(N), M22 bolts through a 6mm connection plate is 82.9kN. The limiting shear value for this plate based on flexural yielding is 72.1kN. On p. 8-226 of the same reference, flexural yielding is listed as an applicable limit state for the end of the supported member where the top and/or bottom flange has been coped.

My question is why does the flexural yielding criteria apply to the coped beam end but not to the connection plate? They are seeing equal and opposite forces and have nearly equal geometry-assuming both top and bottom flanges are coped.

Robert C. Tedrick

The answer is that in tests of single-plate connections of the geometric configurations mentioned in the *Manual*, flexural yielding was not demonstrated to occur. In other words, if you meet the geometric requirements of the analysis method outlined in the *Manual*, flexural yielding is not a controlling limit state. Keep in mind, however, that in plates longer than those considered in the *Manual*, it very well might be a controlling limit state.

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

Column Splices

I am currently working on a job that has several column splices. The splices are made with outside flange plates only. The upper column is bearing directly on the lower, I am assuming, via a note that reads "finish columns to a column plane." The EOR has called out ½" flange splice plates, ⅞" SC bolts, and "fillers as required." What is the largest tolerable gap between splice plate, and upper column flanges? Is this documented somewhere?

Question from Steel-Detail list-server

I think that the information that you are looking for is found in Section M4.4 of the new LRFD Specification, available as a free download at www.aisc.org/lrfdspec.html.

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

Galvanizing Limits

I am working on a project that will have 35' long pipe columns. Does anyone know what sort of lengths of steel can be hot-dip galvanized in the Seattle area? I can cut the pipe back to 29'-6" or so, if the limit is 30'.

Question from Structural Engineers Association International email list-server

Try checking out www.galvanizeit.org. You can find a list of galvanizers in the Seattle area with a list of their kettle sizes. You can also find a state-by-state listing of galvanizers with their capacities in the March 2001 issue of *Modern Steel Construction* (www.aisc.org or www.modernsteel.com).

Scott Melnick
Black Squirrel Communications
Chicago, IL

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A36 Plate

Does anyone have a "ready" number for the proportional limit for A36 plate, from 3/16" to 1/2" thick?

Question from Structural Engineers Association International email list-server

The F_y for ASTM A36 steel plate is 36 ksi up to and including a thickness of 8 inches. Above 8 inches thickness, it is reduced to 32 ksi.

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

Roof Deck Providing Bracing

I am working with some 40 to 50 year old buildings (ASTM A7 steel). The roofs are metal deck about 1 to 1.5 inches deep. What parameters do I use to assume that the supporting purlins are laterally braced by the roof deck? Should I even consider this as the welding of deck is not verified, neither is the weld size or spacing?

Question submitted anonymously

If there is no connection between the deck and purlin, then the purlin certainly is not braced. If there is a connection, then the adequacy of the deck as a brace has to be checked for strength and stiffness. Most engineering textbooks cover this. The old "rule of thumb" of 2% for the brace would mean the deck should have at least 2% of the purlin strength could be conservatively used. Stiffness of the deck should meet an appropriate KL/r limit.

Robert F. Lorenz, P.E.
AISC Alum

Old Beam Sizes

I am analyzing a structure to support a replacement chiller, and found on the original structural framing plans a beam designation of 18B35. The plans are dated 1970, and I am only familiar with W, C, M, and S shapes for steel beams. Can you help me identify what type/size beam this is?

Question submitted to solutions@aiscmail.com

It sounds to me as if you need a copy of the 6th or 7th edition steel manuals. These references can often be found in a used book store (such as www.powells.com). Additionally, I have seen some great bargains on old AISC publications through some auction sites such as www.ebay.com.

If you ever work with REALLY old structures, AISC has a publication that covers section properties for iron and steel beams from 1873 to 1952. This can be ordered through our pubs warehouse, 800/644-2400.

All that being said, I would guess that your member is an I-shape with properties very similar to that of today's W18x35.

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

New Questions

Cambering Galvanized Members

Are there any special considerations that need to be made in specifying camber in a beam that is also to be hot-dip galvanized?

Question submitted anonymously

Braced Frames and 1997 UBC

Can someone refer me to a design guide for the connection of steel braced frames to footings using the 1997 UBC? I am looking for some recommended connection configurations. I am particularly interested in what load factors I should be using and how to treat the grout space and oversized anchor rod holes.

Chris A. Hasse, P.E.

1929 Properties

I am performing a structural analysis on a spillway radial gate constructed in 1929. Do you know the strength values for steel used at that time? Also, if there is any difference in strength values for the following members: angles, channels, plates, rivets, bolts. Could you report them to me?

Any other information you might have on steel from this era would be appreciated, such as, unique failure modes, oddities, etc.

Question submitted anonymously