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:

Steel Interchange
Modern Steel Construction
One East Wacker Dr., Suite 3100
Chicago, IL 60601-2001

Answers and/or questions should be typewritten and double-spaced. Submittals that have been prepared by word-processing are appreciated on computer diskette (either as a Wordperfect file or in ASCII format).

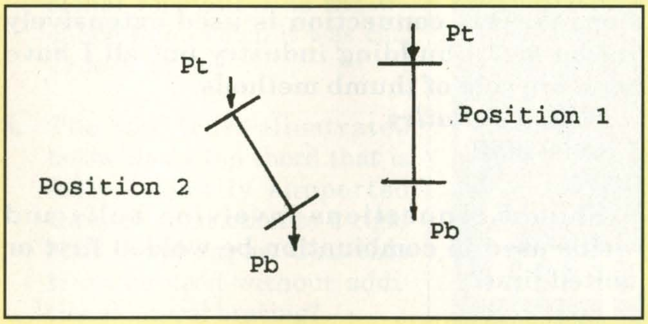
The opinions expressed in *Steel Interchange* do not necessarily represent an official position of the American Institute of Steel Construction, Inc. and have not been reviewed. It is recognized that the design of structures is within the scope and expertise of a competent licensed structural engineer, architect or other licensed professional for the application of principals to a particular structure.

Information on ordering AISC publications mentioned in this article can be obtained by calling AISC at 800/644-2400.

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

The following responses from previous Steel Interchange columns have been received:

Does an unbraced trolley beam that is loaded on the bottom flange have the same buckling characteristics as an unbraced beam loaded on the top flange?



The answer is no. A beam loaded on the bottom flange has a greater buckling capacity than one loaded on the top flange. This is true for both concentrated loads and distributed loads. A laterally unsupported beam will (when the load is sufficiently large) move as shown in the figure from position 1 to position 2. The load P_b applied to the bottom flange exerts a restoring moment pulling the beam back to a vertical position, thus increasing the load required to cause buckling. The load P_t applied at the top flange twists the beam even more, thus decreasing the buckling load. P_t and P_b may be concentrated or distributed loads.

Lateral stability is discussed in the following publications:

"Design of Steel Structures", 2nd ed. Gaylord & Gaylord, pages 266 to 275. McGraw-Hill, 1972. The 1st ed. derives F1-8 in the AISC *Allowable Stress Design Manual*, 9th ed.

"Theory of Elastic Stability", 2nd ed. Timoshenko & Gere, pages 251 to 277. McGraw-Hill, 1961. (Also 1st and 3d editions.)

"Formulas for Stress & Strain", 5th ed. Roark and Young, pages 203 to 208 and chapter 14. McGraw-Hill, 1975.

"A Survey of Literature on the Lateral Instability of Beams", Lee,G.C. Bulletin 63, Welding Research Council, Aug. 1960

Peter Kocsis
Barrington, IL

Another response:

The formulas in the AISC codes for predicting the buckling resistance of beams were developed with the assumption that the load is applied through the shear center. When the load is applied above the shear center of the beam, the buckling strength is reduced. When the load is applied below the shear center, the strength is increased. Reference #1 provides the following formulas that can be used to account for the effect of load height.

For top flange loading:

$$C_b' = \frac{C_b}{B}$$

For shear center loading:

$$C_b' = C_b$$

For bottom flange loading:

$$C_b' = C_b B$$

For beams with a point load at midspan:

$$B = 1 - 0.180W^2 + 0.649W$$

Where

$$W = \frac{\pi}{L}\sqrt{\frac{EC_w}{GJ}}$$

It should be noted that these formulas become unconservative for the rare cases where W exceeds 1.75. For these cases, the procedure in reference # 2 could be used with greater accuracy.

Reference #1: Galambos, T. V., Ed.,1988, Structural Stability Research Council, “Guide to Stability Design Criteria for Metal Structures”, 4th Edition, New York: John Wiley & Sons, Inc.

Reference #2: Johnston, B. G., Ed., 1976, Structural Stability Research Council, “Guide to Stability Design Criteria for Metal Structures”, 3rd Edition, New York: John Wiley & Sons, Inc.

Bo Dowswell
Birmingham, AL

What is the minimum concrete cover required above the head of a shear stud connector?

At one time such a requirement did exist in the AISC Specification. However, the strength of a shear stud connector depends coupon the strength of the connector itself and the cone of concrete under the head, not the concrete over it. Accordingly, that cover requirement was eliminated. The reader is cautioned, however, to allow sufficient slab thickness to accommodate possible variations in composite beam shape, such as that due to camber, which could otherwise cause shear stud connectors to protrude through the top of the slab.

What cautions are required when cold bending material with sheared of flame-cut edges?

When cold bending plates or performing other operations involving cold bending and a sheared or flame-cut edge, caution must be exercised to preclude the initiation of cracks. LRFD Manual Table 9-13 indicates minimum radii for cold bending plates of various steel grades. Additionally, the corresponding text indicates that “Flame cut edgesw of hardenable steels should be machined or softened by heat treatment. Nicks should be ground out and sharp corners should be rounded.

New Questions

Listed below are questions that we would like the readers to answer or discuss.

If you have an answer or suggestion please send it to the Steel Interchange Editor, Modern Steel Construction, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001. Questions can also be sent via e-mail to newman@aiscmail.com.

Questions and responses will be printed in future editions of Steel Interchange. Also, if you have a question or problem that readers might help solve, send these to the Steel Interchange Editor.

Are there any design aids or accepted procedures for designing a bolted tapered haucnh connection with a diagonal haunch? All bolts are inside the column and rafter flanges. This connection is used extensively in the metal building industry but all I have seen are rule of thumb methods.

Wilson J. Bailey
via e-mail

Should connections involving bolts and welds used in combination be welded first or bolted first?

How is a welded double angle connection designed when the double angles are connected to the flange of the column and welded on the back side of the double angles? (See figure below.) This may be necessary when the column flange is short.