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 Word 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 principles to a particular structure.

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

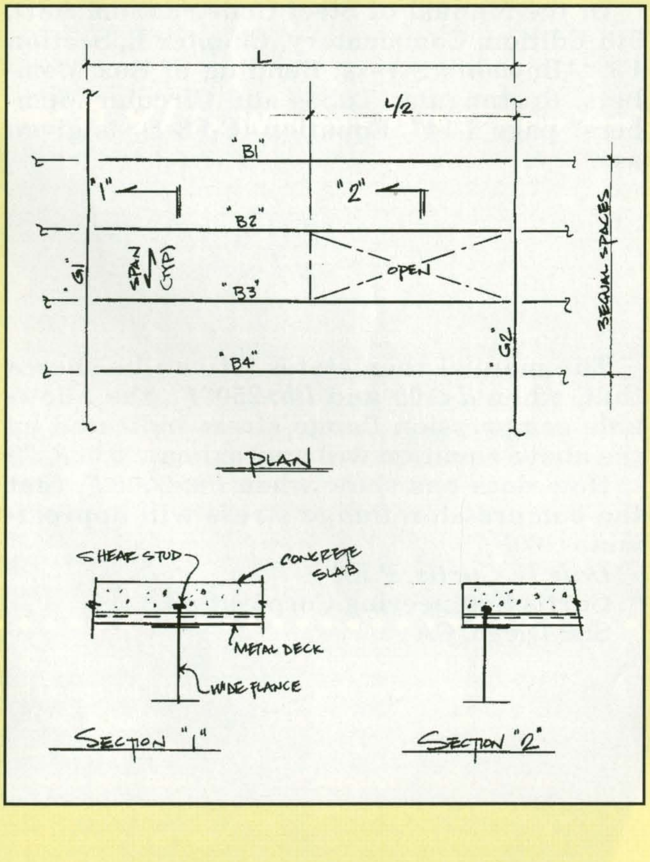
*** * * * Questions and answers can now be e-mailed to: rokach@aiscmail.com * * * ***

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

(From the May 1998 issue)

How is a composite beam designed when there is an opening in the concrete "flange" adjacent to the steel beam? Does the length of the opening affect the design, if the opening length is small relative to the beam length can it be neglected? Does the location of the opening, relative to the maximum moment, affect the design?

For example: If designing beams "B2" and "B3" (see sketch) as composite beams, is it too conservative to treat the beams as edge beams for the total span (only accounting for the concrete "flange" on the opposite side of the opening the full length of the beam)?



Selection of the composite beam is based on satisfying the maximum moment requirement and the moment along the opening. If the maximum moment occurs at the opening, the design is governed by the reduced "flange" width. Otherwise, the beam size considered should also be checked for the reduced capacity at the opening.

In view of the above, the length of the opening is irrelevant except for the deflection factor.

In the case shown in the sketch, with the apparent proximity of the opening to the point of maximum moment, the reduced "flange" width will control the design.

Isaac Gordon, P.E.
Ang Associates, Inc.
Philadelphia

(Another Answer)

This subject has been covered by Kenneth B. Wresner in his paper, "Composite Beams with Slab Openings," in the *Proceedings, 1996 National Steel Construction Conference*.

James Rongoe
Rongoe Engineers
Darien, CT

(From the May 1998 issue)

In my work, I often have to design reinforcing for existing steel beams that have cast-in-place concrete arches between the beam webs. The bottom flanges are usually exposed and if the concrete covers the top flange it is not enough to provide adequate cover for shear studs. Many times the top flange is below cinder fill and roofing which is not to be disturbed.

The procedure I use for design of the reinforcement for increasing the midspan moment capacity is as follows: First I calculate the existing stress in the beam top flange due to dead load only. I then size an inverted WT shape to be welded to the bottom flange of the existing beam. The WT is sized to limit the sum of the existing compressive stress in

the unreinforced beam top flange and the additional top flange stress due to new dead and live loads on the composite section to less than the allowable compressive stress. The beam is also checked for the increased shear stress and the higher end reactions.

Is this method overly conservative? Are there any references on reinforcement of non-composite beams? This method has its limitations when the existing dead load is close to the allowable.

In the 2nd Quarter 1996 *Engineering Journal*, John Miller, P.E., presents an article entitled "Strengthening of Existing Composite Beams Using LRFD Procedures." This method can also be used for non-composite beams. A non-composite beam can be thought of as a composite beam in which the concrete has a thickness of zero. Mr. Miller also uses an inverted WT section which is welded to the bottom flange.

I wrote a discussion of Mr. Miller's paper (3rd Quarter 1997 *Engineering Journal*) in which I presented another method of strengthening the same beam Mr. Miller had used. The method I used is post-tensioning by the use of cables applied below the composite sections, and is very easy to use. The simplest case is the non-composite. In my example, the cables are applied to the web, but they can be applied to the bottom flange if there is sufficient headroom.

In my discussion, there is a printing error. The length of weld required is:

39/(2x2.39) = 8.2 inches on each side.

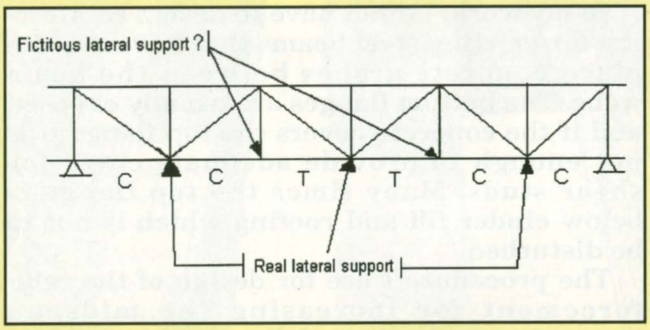
The expression for T prime did not print clearly:

The additional tension is T' (T prime)

T' = NUM/DEN

Peter Kocsis, S.E., P.E.
Barrington, IL

For a specific load combination, some bottom chord members of a continuous steel truss are in tension and others are in compression.



Is it possible to consider that the node, located between a tension and compression member, can behave like a lateral support to evaluate the compression strength with respect to the weak-axis of this member (by analogy to the inflection point of a bending moment diagram acting as a lateral support from beam lateral-torsional buckling)?

Professor Joseph A. Yura of the University of Texas at Austin, and probably others, have demonstrated analytically and through the use of finite element analysis that the use of inflection points as braced points in solid web I beams is not appropriate. An open web beam is probably sufficiently analogous to a solid web beam to support the assumption that this practice would not be prudent in either.

Steven J. Thomas, P.E., S.E.
Manager, Product Design
VP-Buildings, Inc.

New Question

Listed below is a question 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.

In the Manual of Steel Construction, ASD, 9th Edition, Commentary, Chapter F, Section F3 "Allowable Stress: Bending of Box Members, Rectangular Tubes and Circular Members" page 5-147, Equation (C-F3-1) is given as:

$$\left(\frac{l}{r}\right)_{equiv.} = \sqrt{\frac{5.1\, l\, S_x}{\sqrt{J\, I_y}}}$$

The manual then states "It can be shown that, when $d<10b$ and $l/b>2500F_y$, the allowable compression flange stress indicated by the above equation will approximate $0.60F_y$."

How does one show when $l/b>2500/F_y$ that the compression flange stress will approximate $0.60F_y$?

Dale H. Curtis, P.E.
Curtis Engineering Corporation
San Diego, CA