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 312/670-2400 ext. 433.

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

For fire wall construction, building codes say the wallshall have sufficient stability to allow for collapse on either side of the wall without collapse of the wall. What is the optimum detail for this type of connection.

The sketch provided with the question submitted is applicable for a specific type of tied fire wall; non-loadbearing, constructed between two double-column lines, structural members on each side of the wall at the same elevation and primary framing members parallel to the wall. For a scenario such as this the recommendations of Factory Mutual Loss Prevention Data Book 1-22 are very specific and are as follows.

The anticipated horizontal component of the force resulting from the collapse of the structural frame on one side of the wall should be resisted by the remaining structure on the opposite side of the wall. This is accomplished through the use of through-wall ties. The ties are designed based on the horizontal pull "H" calculated from the formula provided in Recommendation #3 of the referenced FM Data Book, using an allowable stress of not more than 10 ksi. A detail of the recommended installation of the through-wall tie at each column line is shown in Figure 12 of the same FM Data Book. For the situation indicated by the sketch in question, it may be necessary to also install ties more often than every column line. In either case, enough slack should be provided in the tie connection to allow for normal building movement.

While the through-wall ties insure the continuity of the opposing framework at the fire wall, flexible masonry anchors should be provided at approximately 2 to 4 feet on center to brace the wall laterally (see Figure 13 of the referenced FM Data Book). It is important to note that enough slack should be provided in the anchors to compensate

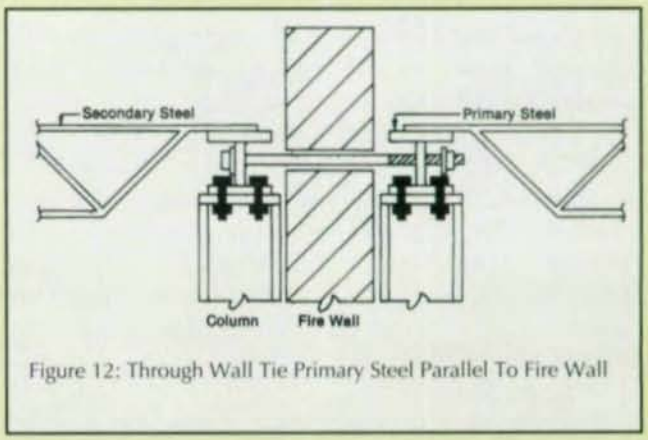

Figure 12: Through Wall Tie Primary Steel Parallel To Fire Wall

for the slack provided in the through-wall ties. This slack insures that the collapsing frame on the fire side of the wall will not pull on the wall before there is resistance provided from the frame on the unexposed side of the wall via the through-wall ties.

The Factory Mutual recommendations also include provisions for adequate separation between the double-column line and the fire wall to prevent damage to the unexposed structure during the initial stages of the fire.

D. Matthew Stuart, P.E.
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What is the most efficient way to enlarge an existing footing, when new loading conditions are applied?

There is a good article dealing with this problem. It was published in the fourth quarter AISC Journal 1980 and was authored by Agrawal and Stafiej. The parameters required to solve the problem include ratios of the respective moments of inertia of the two column sections, ratios of axial loads applied at the top of the column to loads at the lower section, and ratios of the upper length to the lower length. Using these

ratios, one then uses a chart which gives equivalent effective lengths factors for the composite column for six different end condition cases, pin-pin, fix-free (Steel Interchange question case), fix-pin, fix-slider, fix-fix, fix-pin, and pin-slider. From the determined effective length factors, the effective lengths of the upper and lower column sections are easily obtained for use in the Euler buckling formula.

James F. McCarthy
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Is the method of determining the flexural design strength of a single angle given in the Manual appropriate for unequal legs not loaded through the shear center?

A good reference for this question is a paper by Tide, Raymond H. R. And Norbert V. Krogstad, Economical Design of Shelf Angles, Masonry: Design and Construction, Problems and Repair, ASTM STP 1180, John Melander and Lynn R. Lauersdorf, Eds., American Society of Testing and Materials, Philadelphia, PA, 1993, p. 60.

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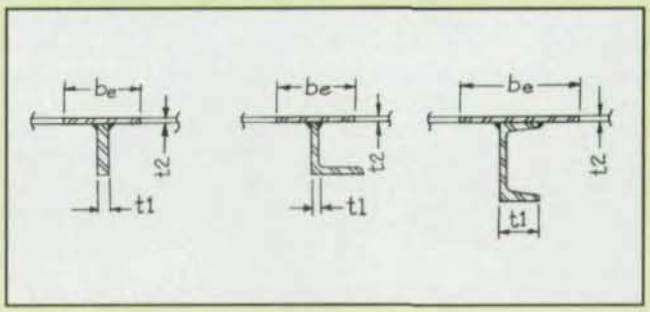
New Questions

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 Co nstruction, One East Wacker Dr., Suite 3100, Chic ago, IL 60601-2001.

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.

Given a wall of sheet metal or plate subjected to fluid pressure and stiffened by same size parallel members spaced regularly, what section (or width) of the wall shall be used that contributes to the section of a stiffener? The stiffening member may be a flat bar, an angle, a channel (see figure) or any other section.



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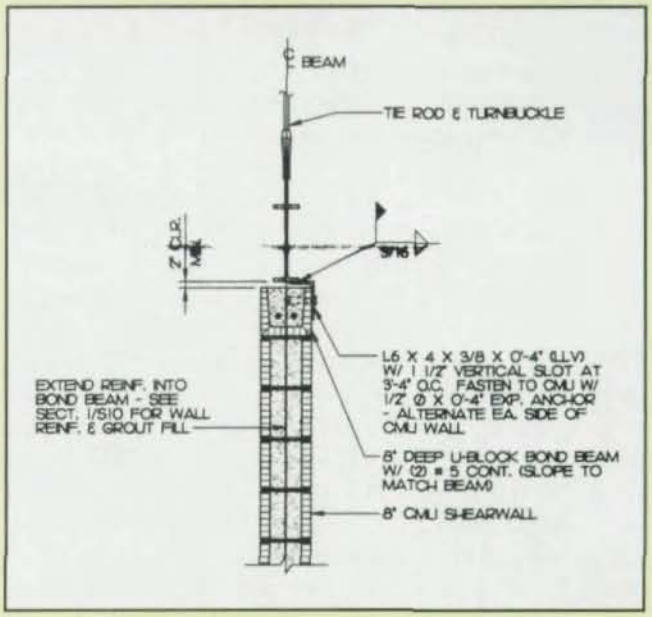
Is there a more efficient and cost-effective way to connect a masonry shear wall to structural steel framing? The most common problem with the following detail is that once the masonry is built up to the bottom flange of the beam, there is not enough room to install the grout and continuous reinforcing bars in the bond beam at the top of the wall. If the bond beam is dropped a course in elevation, the masonry to steel beam connecting angle vertical leg or bent plate vertical leg becomes excessively long.



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