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

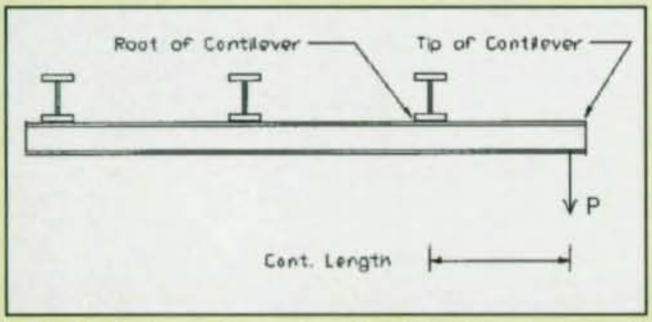
Answers and/or questions should be typewritten and double-spaced. Submittals that have been prepared by word-processing are appreciated on computer diskette (either as a Wordperfect file or in ASCII format).

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.

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

For a continuous trolley beam with multiple spans and cantilevered ends what is the lateral unbraced length for the bottom flange? Can the distance between points of inflection be considered an unbraced length?



We have been involved in the design of many monorail and bridge crane systems that had similar cantilever conditions. Since the number of the systems investigated was very large, we did some research in this particular problem, and we are preparing a paper for publication of the findings.

If the beams are doubly symmetric sections, such as wide flanges and S shapes, one suggested solution is given in the Guide for Stability Design Criteria for Metal Structures, edited by T.V. Galambos, section 5.2.4 page 168. The procedure gives the critical buckling moment, M_{cr}, for the cantilever to be:

where, M_{cr} is the theoretical critical buckling

$$M_{cr} = \frac{\pi}{KL}\sqrt{EI_yGJ}\sqrt{1 + \frac{\pi^2 EC_w}{(KL)^2 GJ}}$$

moment without any factor of safety, L is the cantilever length, K is an effective length factor, E is the modulus of elasticity, G is the shear modulus, I_y is the minor axis moment of inertia, and Cw is the warping constant, and J is the torsional constant. Both J and C_w are provided in the AISC Manuals for standard wide flange and S shapes. The value of K varies depending on the restraint conditions at the root and at the tip of the cantilever, as well as the location of the load with respect to the neutral axis (as indicated by figure 5.11 of the above reference). For the case in question, where the cantilever is continuous over the root with only top flange laterally restrained at the root, no lateral restraints at the tip, and bottom flange loading, the reference suggests a value of K of 3.0.

This will result in a value for the critical buckling moment. An appropriate factor of safety, typically in the range of 1.67 to 2, should be applied too obtain the allowable moment. In addition, this allowable moment should be limited to 0.66 of the yield moment for compact sections and 0.6 of the yield moment for non-compact sections.

In effect this method gives an unbraced length of 3.0 times the cantilever length, and there is no need to use the unbraced length to the inflection point. We should stress the fact that this method does not apply to singly symmetric beams, i.e. patented track, that are frequently employed for trolley support.

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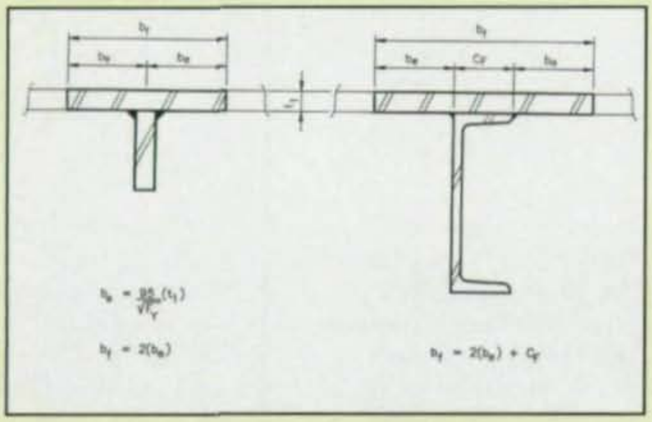
Given a wall of sheet metal or plate subjected to fluid pressure and stiffened by same size parallel members spaced regularly, what section or (or width) of the wall shall be used that contributes to the section of a stiffener? The stiffening member may be a flat bar, an angle, a channel or any other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



The effective width contributing plate section should be limited to the width thickness ratios for compression elements as found in Table B5.1 of the AISC Allowable Stress Design Specification for Structural Steel Buildings. As a bending member, the maximum b/t ratio should be limited to less than $95/\sqrt{F_y}$, to be considered as fully effective. A general rule of thumb is to consider a total plate width contribution of 32t for structures comprised of A36 steel, with a corresponding allowable bending stress of $0.6F_y$.

The figure depicts these recommended limits.

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ANOTHER RESPONSE:

The effective projection of the plate on either side of the stiffener being in contact with the plate should be 16 times the thickness of the plate: Thus, $b_e = t + 32t_2$.

This common practice to obtain a transformed section has been widely used in the design of tanks for liquid storage per standards by the American Petroleum Institute (API).

The American Iron and Steel Institute in its publication entitled Steel Tanks for Liquid Storage, revised Edition 1976 provides a table for the section for the section moduli of the stiffening ring sections based on the 16t effective projection on each side of the stiffener.

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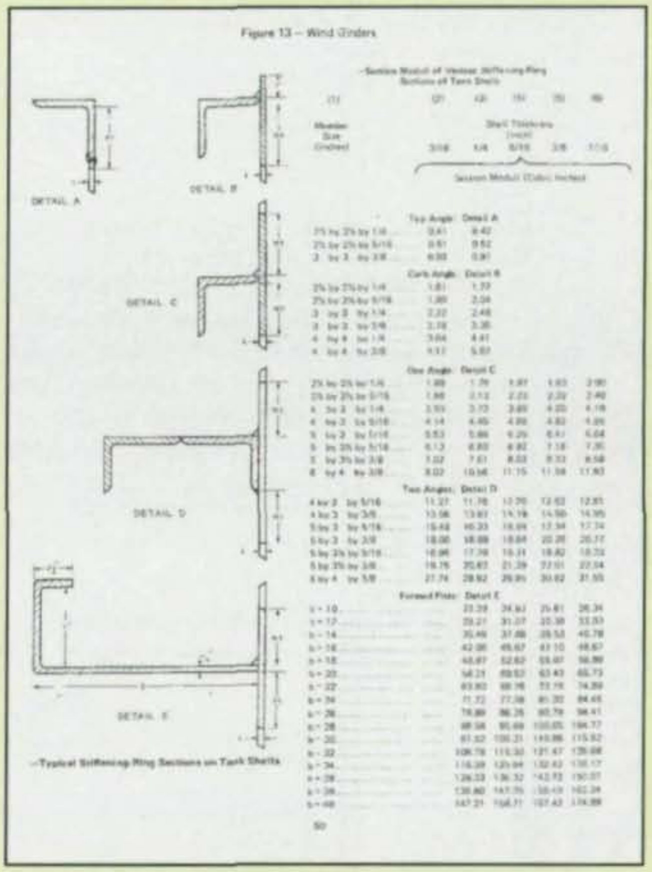

Questions:

Has any engineering firm ever designed a multi-story unbraced frame using mainly semi-rigid (partially restrained) connections? Which, if any, computer programs were used to assist in the analysis and design? What are some of the major pitfalls in using partially restrained moment connections?

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