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:

In designing composite steel girders in accordance with the LRFD Method, it has been well established that significant reductions in beam sizes can be achieved. However, in my experience, I have found, in some cases, the most economical girder sizes may be unsafe during unshored wet concrete construction. This can occur when the metal deck runs parallel to the girder and, in my judgement, does not afford significant lateral restraint to the top flange of the girder in compression. For this condition, the unbraced length is the spacing between the beams supported by the girder. Significant reduction in the non-composite moment capacity can occur due to lateral torsional bucking which may not be adequate for the unshored wet concrete construction.

No criteria for this serviceability problem or guidance appears to be given in the LRFD specification. I would like to know whether there has been any testing or research to demonstrate that metal deck, parallel to the girder does indeed provide adequate restraint or that checking the beam size for the temporary construction condition, should be carried out as outlined above.

The optimization of composite steel plate girder design increases the potential for both flange and web buckling to occur during construction. Higher-than-allowable compressive stresses and resultant buckling are almost certain to occur in these girders if stability during construction is ignored by the designer. This problem is not limited to those girders designed under the LRFD Specification. Since the use of composite construction became prevalent, engineers have identified the problem of instability in steel girders during construction.

In failure cases observed in Pennsylvania, metal deck pans provided insufficient bracing against lateral flange buckling at critical sections. Evidence indicates that the failure of the welds connecting the pans to the flange can be expected to occur prior to flange buckling. Although increasing the capacity of this connection may be considered, the designer should be concerned about the quality control of such a critical connection, as well as the associated cost-effectiveness of this approach. More significantly, metal deck pans will do nothing to prevent excessive web buckling which is as likely to occur (but not as likely to be detected) in the compressive region of unstiffened webs.

AASHTO's Standard Specifications for Highway Bridges, 15th Edition, addresses the problem of construction instability in Article 10.50, by limiting both the compression flange shape factor (b'/t ratio) and the lateral-torsional buckling moment capacity (M_r) for composite girders subject to non-composite dead loads. In some instances, states have introduced their own criteria for checking this condition, which are typically more conservative than AASHTO. Pennsylvania, in particular, has gone to great lengths to develop their own design parameters. In general, the AASHTO criteria is a widely accepted check for the stability of girders during construction.

The AASHTO criteria can be met by reducing the length of deck pours, increasing the size of the steel girder section, reducing the distance between lateral brace points (i.e. diaphragms or cross beams), or by a combination of these methods. Limiting the length of deck pours is critical for continuous girders, where temporary positive moments from the wet concrete may be much larger than the final positive dead load moment that will exist after the entire deck has been placed.

The methods used to mitigate construction-related stresses should be determined by the designer after consultation with contractors and fabricators as to the economics of the various alternatives. When the methods chosen require control of the construction process (i.e. deck pouring sequence), this should be clearly indicated on the construction plans. As the question accurately illustrated, the need to address construction-induced stresses will become more critical as we continue to optimize and refine our design methods.

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The use of channel sections or other lightweight narrow flange sections as girts supporting non-bearing exterior wall assemblies against wind load is common practice. How is lateral instability of the unsupported compression flange accounted for when the wall is subject to outward pressure due to suction at the leeward face of the building? These outward forces are equal to or greater than the inward forces.

There are two basic approaches to this problem. The answer depends on the designers degree of conservatism in treating the role of sag rods that are often required to minimize the excessive girt sag under own weight.

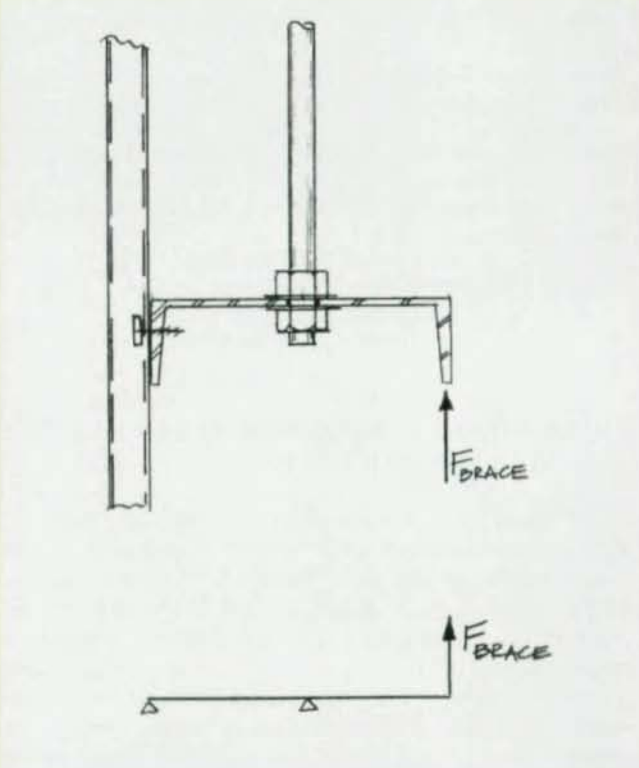
One approach, obviously a very conservative one, considers the interior flange as completely unbraced from column to column. The channel sections designed under this scenario are usually so heavy that it often makes sense to use wide flange girts instead.

Another approach recognizes a restraining action of sag rods and considers the channels laterally supported at each sag rod location. The number of sag rods may have to be increased to provide enough bracing points to maximize the allowable bending stresses in channels (AISC ASD Spec. Chapter F). This seemingly unconservative approach has been used for decades and withstood the test of time.

An attempt to rationalize this practice can be made as follows. For the channel girt to buckle under wind suction loading, its interior flange must move vertically. At the point of the sag rod attachment this movement is prevented, as it is at the exterior flange stabilized by the wall siding fasteners. It is widely recognized that the compression flange of a flexural member may be considered braced if the brace can resist a force of about 2% of the flange compression. If this force can be developed by the web cross-bending, the required bracing is present.

The figure illustrates the assumed model of the web acting as a cantilever beam. The width of the web effective in this action is determined by engineering judgement.

It is worth mentioning that cold-formed C- or Z-girts may often be more economical than structural channels. These members are usually continuous over the column supports, since lap splicing can be easily made, and the points of inflection are frequently assumed to be braced laterally.



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

Can threads on anchor bolts be either rolled or cut? Is one method better than the other?

What is the most efficient and cost-effective way to connect a steel wide flange girder to a concrete column?

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