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:

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.

It is instructive to determine the force required to prevent buckling. A C8 x 11.5 is commonly used as a girt. For simplicity, assume the full flange is stressed to 20 ksi. The force developed by the flange is 17.6 kips (0.081 in.^{2} x 20 ksi = 17.6 kips). Invoking the 2 percent rule gives a required resisting force to prevent buckling of 350 pounds. This is small compared to the forces structural engineers usually consider.

I note that girts are generally held level with one or more sag rods spaced along the girt's span. The sag rods commonly pass through the girt web and are secured with a single nut placed beneath the girt.

If the girt is installed with a slight bow downward, the girt flange can only laterally buckle downward since it is not possible to reverse initial curvature. Provided the sag rod is placed close enough to the compression flange, any tendency to buckle will be resisted by the sag rod in tension. The relevant question is how close to the unbraced flange must the sag rod be? I do not know of any authoritative guidance on this matter.

However, if the girt is initially set with an upward deflection and supported by a single nut below the web, the girt may laterally displace upward unhindered except by gravity. this problem may be overcome by providing a double nut connection - one below and one bolt above the web. This would place the sag rod in compression. Although the sag rod's capacity in compression is small, the force required to resist flange buckling is also small.

In any event, thousands if not hundreds of thousands of industrial buildings have been constructed with metal siding supported by girts and the failure of girts under wind loading does not appear to be a serious problem. I believe that in many of these buildings little thought was given to the unbraced girder flange. I have personally observed "pre-engineered" buildings where light gage Cee or Zee girts are stayed between columns by the exterior siding only.

I speculate that girt supported walls perform satisfactorily for several reasons, regardless of the potential for flange instability. One, code dictated wind loads are conservative and rarely achieved. When achieved, it is likely that the wall diaphragm is breached relieving some of the loading. Also, wind loads are short-lived and the inertia of the girt-wall system results in a time-lag before the girt is fully deflected and stressed. During that delay the gusting wind may slow or change direction. Should the girt flange buckle, additional load carrying capacity may be developed through catenary action. In other words, the girt becomes a tension member restrained by its supports. And of course, the girt to column connection may afford substantial continuity.

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When erecting steel beams on a brick wall, could the non-shrink grout be omitted under a proper bearing plate if the surface of the brick is smooth, clean of any and all debris and leveled?

Even though the author of the question describes the brick bearing surfaceas "smooth, clean...., and leveled", the degree of perfection commonly found in brick masonry is no match for the flatness of a steel bearing plate.

Without grout the load will not be uniform but rather will be concentrated at the high points of the brick. This may lead to fragmentation of the brick masonry. One of the purposes of grout is to help distribute the load uniformily. It should not be omitted.

David Ricker, P.E.
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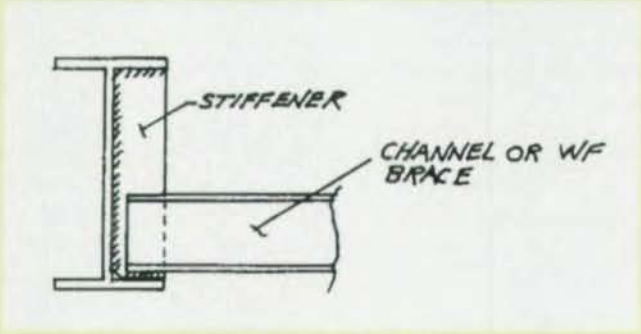
What type of framing is considered bracing the compression flange? Does the member bracing the flange have to be attached to the flange? If a 4 inch deep member frames into mid-depth of a 10 inch deep beam is that considered bracing the compression flange (centerlines of each member at same point)?



As discussed in the ASD 9th Edition Section C-F1, strong-axis-bending lateral-buckling can be prevented either by bracing the compression flange directly or by preventing the section from twisting. The first of these methods can be done using a concrete slab, properly attached deck or steel framing. Due to coping or framing considerations, steel braces often are not directly attached to the compression flange. A rule of thumb is that the connection for a brace should extend at least into the upper $^1/_3$ of the beam depth to consider it as a lateral support for the top compression flange. Where this is not possible, the detail shown could be used to assure lateral support.

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

Anchor bolt threads may be cut or rolled, depending on the project specification requirements. Allowable stresses published in AISC and other codes account for any differences in strength which may result from one method or the other.

A good article on this subject is titled, "Rolled Threads vs. Cut Threads," dated June 20, 1966. This article was published in Fastener Facts and was released in the mid 1970's by the Bowman Products Division of the Associated Spring Corp. of Cleveland Ohio. To summarize, the article states that rolling and deforming the grain structure, rather than cutting through it , results in additional strength or resistance to thread shear.

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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.

In regard to single plate shear connections, Table X of allowable loads in ASD 9th Ed. is based on conservative assumptions such that in many cases the bolt diameter, type, or quantity must be increased to satisfy the required loads. Is the method used in Engineering for Steel Construction (University of Arizona research) still acceptable, particularly when the job specifications call for using the "latest AISC standards"?

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Specifications currently exist which require minimum pretensioning loads for slip critical connections. There is, however, no guidance regarding minimum pre-loading of anchor bolts which occur at column bases. While in most situations this issue is academic since the anchor bolt nut and thread projection are below the plane of the concrete slab on grade and are eventually embedded in concrete at the slab isolation joint, there are instances where the nut and thread projection remain exposed. Is tightening the nut to "snug tight" and tack welding the nut to the bolt thread the only solution in preventing the nut from backing off?

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