

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

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

A reader wrote to state a concern regarding one of the answers in a previous *Steel Interchange*. His concern was that the response interpreted the AISC Specification and only the AISC Committee on Specifications can officially interpret the Specification. This observation is correct; however, *Steel Interchange* is intended to be a forum for readers to discuss ideas and information on all phases of steel construction. As stated in each month's disclaimer, the opinions expressed are not endorsed by AISC nor do they represent official opinions of AISC, but are the authors own recommendations.

Are there any good connection details for a truss made up of all WT sections?

We have designed and/or detailed (as we are sure that a good number of others have) many welded trusses using WT chord sections which work very well with double or single angle web members.

We would suggest, if it be desired to use WT web members, that they be welded flange to the chord web all from one side. The eccentricity effect on the truss is usually less for all welded one side than alternate sides. Of course, double WT web sections would eliminate that. Shop welding costs would also be considerably less for welding one side only.

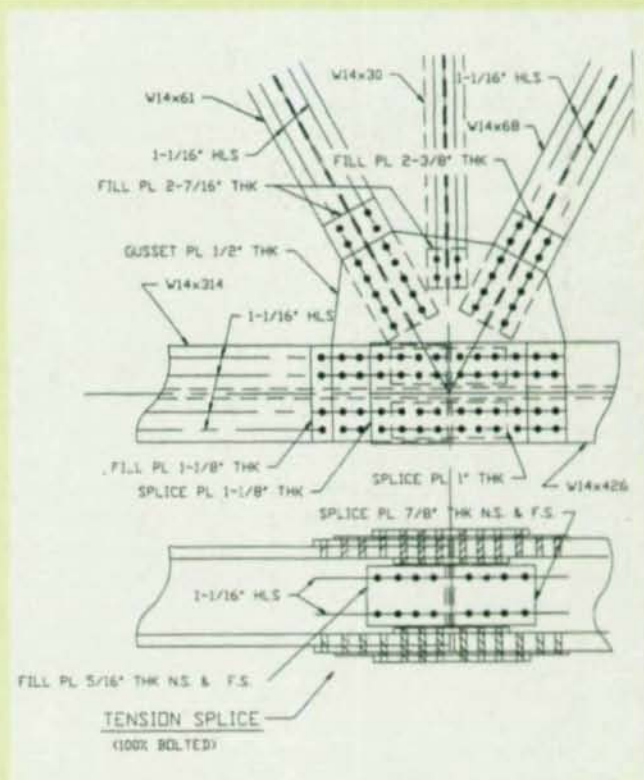
A variation of the above question, which arises frequently, is: "Are there any good connection details for a truss made up of all W sections?" A number of good details have been developed by many engineers using field welded connections. This question was recently explored in an ASCE publication and apparently some engineers feel as we do that bolted connections are preferred for reasons of fit-up, shock resistance, and overall costs. In the case of high corrosion vulnerability, field welding may be the best choice; however:

1. Where fit-up tolerances are critical, shop pre-

pared bolting is almost a necessity and fit-up bolting may be required, even for field welded connections.

2. Gusset plates for the heavier W sections can become unwieldy, need to be flattened or milled, or become unduly heavy and expensive unless thinner plates are utilized by laminating the gussets at chord splices.

Cost effective joints can be designed to fit this criteria by developing the required fill plates before adding the next layer of gusset plate so as to connect the web members. Make the thickness of the web member gusset no thicker than needed for connection. Where the main chord is also spliced, smaller sized splice plates can be added inside chord flanges, web, and on top of main gusset plates. Use slip critical type connections.



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The figure indicates typical details as designed by us for a large building roof support system.
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What is the origin of the Vierendeel truss?

Professor Vierendeel's development (a truss without diagonals, which maintains its shape by the rigidity built into its joints) was first used in 1893 for the construction of a church steeple in Flanders Belgium. Then, for the International Exposition of Brussels in 1897, Vierendeel designed a riveted steel bridge 103 feet long.

After the overcoming of initial opposition by many engineers to the concept, well over 100 Vierendeel truss railroad and highway bridges were built in Belgium, the Belgian Congo and elsewhere. In recent years, it seems to be used more in buildings than in bridges.

It is true that Vierendeel truss design by hand can be tedious. However, a method developed by Grinter and Tsao, presented in the October 1953 Proceedings of the American Society of Civil Engineers, simplified the procedure greatly in those days before computer solutions were commonly available. The system consisted of an adaptation of the Hardy Cross method of moment distribution by successive approximations, with the usual Hardy Cross stiffness factors being modified by simple multipliers. This method was useful to me some years ago, when I had to produce a design for the lower floor portion of several stories of a section of building spanning a city street, in which architectural requirements for windows prevented the use of diagonal web members. The solution was story-high Vierendeel trusses, made up of heavy 14 in. steel sections with welded joints.

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The use of channel sections or other light-weight 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.

I have found guidance in Galambos (*Guide to Stability Design Criteria for Metal Structures*, 4th Ed., 1988, Wiley & Sons, New York, pp.

172ff). In particular there is an expression for determining the critical moment of a symmetrical wide-flange beam with the tension flange provided with an infinitely stiff, continuous, lateral (not torsional) restraint. Most girts and metal studs are not symmetric wide flanges, but other work suggests that the equations for wide flanges are only about 5% non conservative for hot rolled channels. In many cases the critical moment is much greater than the yield moment, so the discussion becomes academic.

A lower bound for any section as proposed by Winter is also discussed. It essentially treats the entire compression area as a partially restrained column. The "truth" no doubt lies somewhere in between the two approaches. However, the methods converge fairly closely for very slender girts, and may prove useful. I have found that the majority of channel girts attached fairly frequently to metal siding have very high critical moments (even without considering the rotational restraint of sag rods), and thus are governed by yield considerations.

I am told that additional guidance may be found in Yu (*Cold-Formed Steel Design*, 1985, Wiley Interscience, New York), but I am unfamiliar with the work and have not used it.

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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.

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.

When, if ever, is it acceptable to consider an inflection point to be a bottom flange brace point in the design of a continuous beam? If an inflection point cannot be considered a brace point, then what values of L_b and C_b should be used?

Is it acceptable to use $K = 1$ for the design of moment resisting columns in an unbraced frame if a second order analysis is performed?

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