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 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
1 East Wacker Dr., Suite 3100
Chicago, IL 60601

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. 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 principles 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 comments and responses to questions from previous Steel Interchange columns, as well as to other articles in this and other AISC publications, have been received:

What is the effect of designing with LRFD on deflection and vibration?

With the increasing popularity of LRFD, I expect that deflection and vibration related concerns will become a large part of claims of structural engineers.

Conclusions from deflection studies (such as those in MSC April, 1992 *Mega Mall Creates New Shopping Experience*, p. 20) may be helpful in keeping structural engineers out of court, where the project size is not adequate to justify such studies.

Discussions of the deflection, vibration concerns may also reduce the hesitancy on the part of engineers to change from ASD to LRFD.

Roger W. McGarrigle, P.E.
Van Domelen/Looijenga/McGarrigle/Knauf
Portland, OR

What is a good "wind" connection for the top of a column?

In response to the question regarding a "wind" connection for the top of a column, I offer the detail shown in Figure 1. This is a slight variation or the "semi-rigid" sketches shown in the April Steel Interchange.

Donald J. Shurilla, P.E.
F & M Associates, Inc.
Allentown, PA

There are many different possibilities concerning a good "wind" connection at the top of a column depending on shop and field preferences. My favorite is shown in Figure 2.

Robert O. Disque, P.E.
Besier Gibble Norden
Old Saybrook, CT

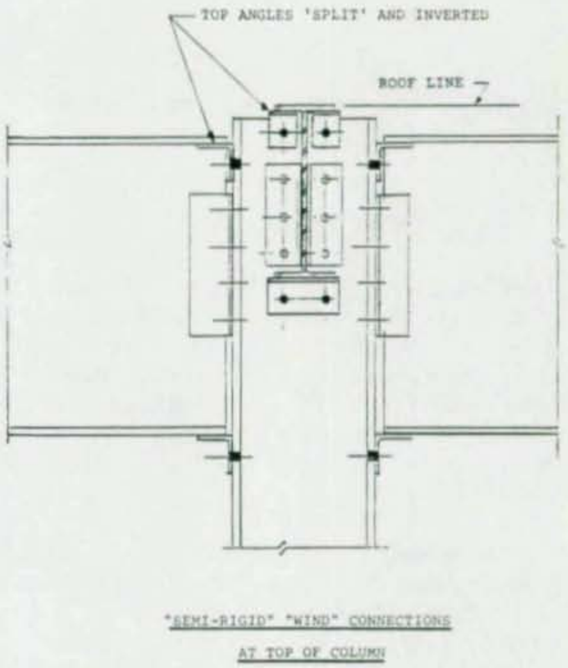

Figure 1: Semi-Rigid Wind Connection

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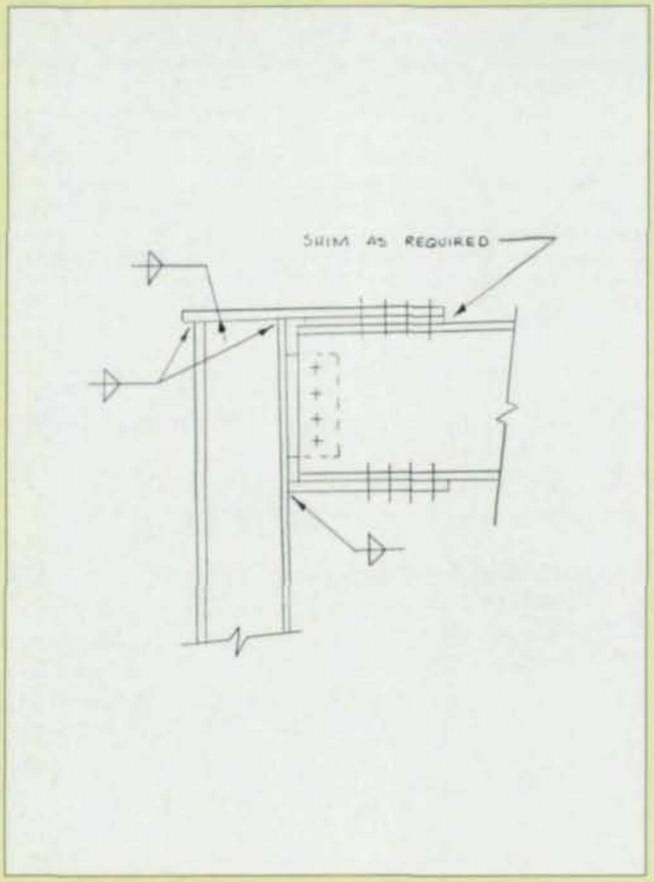

Figure 2: Wind connection at top of column

New Questions

Listed below are some 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. The question 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.

1. The following oddity exists when comparing 0.4Fy versus 0.3Fu shear stress values. The ratio of Fy to Fu for A36 and A572 Gr. 50 steel is not proportional, reflecting the lesser ductility of the higher strength steel. For applications based on Fv = 0.4Fy, the allowable shear stress for A572 Gr. 50 steel is 39% greater than A36 steel; however, for applications based on Fv = 0.3Fu, the allowable stress is only 12% greater for the A572 Gr. 50 steel.

For A36 steel there is an increase in going from $0.4F_y$ to $0.3F_u$. For A572 Gr. 50 steel there is a decrease.

Type of Steel	$0.4F_y$	$0.3F_u$
A36	14.4 ksi	17.4 ksi
A572 Gr. 50	20.0 ksi	19.5 ksi

Three questions arise from this paradox:

a)When a single round hole penetration is required in a beam web, is it proper to use $F_v = 0.4F_y$ or $F_v = 0.3F_u$ when calculating the beam shear capacity?

b)Would a row of bolt holes behave differently than one large round hole which resulted in the same net area?

c)Does the presence or absence of bolts in holes affect the shear capacity of the member?

David T. Ricker, P.E.
Payson, AZ

2. The AISC design procedure for end-plate moment connections is for static loading only. (See LRFD manual, 1st Edition, p. 5-143 and ASD Manual, 9th Edition, p. 4-116.) Why is this restriction made? What is the definition of static loading? Can this connection be used for a utility bridge that has wind loading? Can it be used on a frame that supports a crane runway?

Barry K Shriver, P.E.
Piedmont Olsen Hensley
Greenville, SC

3. How should I connect wide flange beams to all four faces of a structural tube column in such a way as to transfer wind moments as well as dead and live load reactions?

John W. Kush, P.E.
Zurheide-Herrmann, Inc.
St. Louis, MO