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

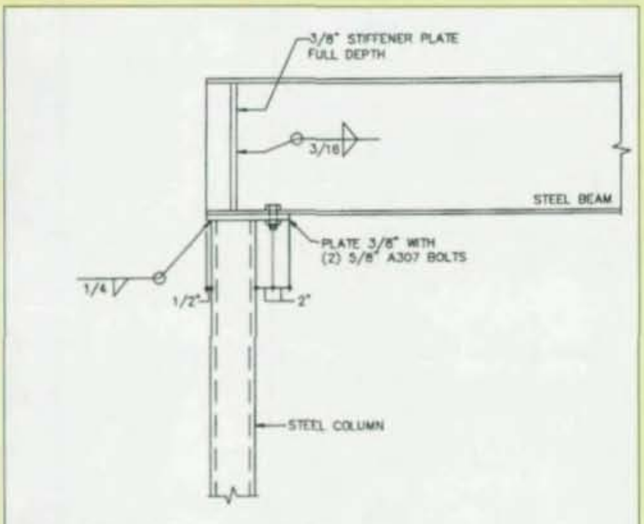
Answers and/or questions should be typewritten and double spaced. Submittals that have been prepared by word-processing are appreciated on computer diskette (either as a Wordperfect file or in ASCII format).

The opinions expressed in *Steel Interchange* do not necessarily represent an official position of the American Institute of Steel Construction, Inc. 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 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 responses to questions from previous *Steel Interchange* columns have been received:

When designing a horizontal beam resting on columns with an unbraced compression top flange, may full-height web stiffeners at the bearing ends provide bracing to the compression flange without any intersecting beams? (See Detail)



COLUMN TO BEAM CONNECTION

This is in response to the answer by Mark W. Cunningham that appeared in the July 1993 *Steel Interchange* column. That answer apparently approves of a seated beam connection with no lateral support for the web or top flange. It has been my belief that some type of support for the upper part of the beam should always be provided at seated connections. This belief is buttressed by comments in almost any text on steel design as well as by statements in AISC publications, e.g., the first line on page 4-35 of the 9th Edition, *Manual of Steel Construction - Allowable Stress Design and Plastic Design*. The purpose is to provide some lateral stability so that the beam can not "roll" on its support with prevention of web buckling as a secondary consideration. I, too, have sometimes wondered if full height web stiffeners at the beam seat could be considered to serve the same purpose.
Frank C. Hartzell, Jr.
Wynnewood, PA

This response is intended to offer an opinion on the above question as well as the response by Mark W. Cunningham in the July 1993 *Steel Interchange* column. The July answer downplays the significance of a top flange restraint at the end of the span.

I believe Mr. Cunningham misunderstood the question. The inquiry specified that the top flange of the beam was unbraced. Common sense suggests that if the top flange is unbraced and the ends are unrestrained the possibility of the beam "rolling over" is significantly greater than if the beam were restrained at the end. To extend the column buckling analogy given in the July answer, consider the classical pinned-end column. The pinned end of a column is a restraint from lateral displacement while allowing rotation. If the end of a column to be tested were placed on a roller it would simply fall out of the testing device. In a similar sense, the end of the compression flange needs to be restrained. The matter is one of boundary conditions, not the magnitude of compressive stress.

Regarding the original question, I believe the stiffeners are required for end restraint of the beam. In typical clip angle framing to the side of a column, top flange lateral restraint is provided by connection of the clip angle to the upper one-third portion of the beam web. For the beam seat detail, even with stiffeners, a large beam placed on a relatively light column would not be adequately restrained since the stiffeners derive their restraining capacity from the bending stiffness of the column below.

I am not aware of applicable code requirements or experimental or theoretical studies on this subject. It would be worth a literature search. If the information is not already available, a study of relative beam, column, and stiffener properties required to provide the required restraint would be worthwhile. As stated by Mr. Cunningham, the derivations I have seen require axial stiffness of a lateral brace to be only a small percentage of the top flange stiffness. A relationship relating the equivalent rotational resistance of

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the column and the stiffeners to the axial stiffness of an adequate lateral brace would be easily applied in practice.
Gordon C. Glass, P.E., S.E.
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Lexington, KY

The 9th Edition ASD Manual states on page 4-84 that, when using single angle connections, "Where possible, the distance between the centers of the top and bottom connecting bolts should equal or exceed one-half the T-distance of the supported member to guard against overturning of the beam." Alternatively, Volume II - Connections of the Manual says, on page 3-96, "To guard against overturning of the beam, it is recommended that the distance between the centers of the top and bottom connecting bolts be equal to or exceed one-half the T-distance of the supported member when possible. This is not a Specification requirements and the fabricator may elect to satisfy T/2 by using the more traditional length of the connecting angle."

This is somewhat confusing. Why is there a difference in the two publications?
John Simon, P.E.
Chantilly, VA

When single angle connections were introduced into the 9th Edition of the *Manual of Steel Construction - Allowable Stress Design and Plastic Design*, the requirement that T/2 be met, if possible, by using the distance from the centers of the top and bottom bolts was arbitrarily included in the design aid. When the AISC Committee on Manuals, Textbooks, and Codes was developing Volume II, it was called to our attention that this is more restrictive than any other one sided connection where T/2 is satisfied by the more traditional method of using the length of the connection. It is believed that the clamping action of the bolts in the connection, even when snug tight, approximates the length of the connection material, making the more traditional method of satisfying T/2 acceptable.

To be consistent, the Committee has now revised this statement and T/2 for single angle connections may be met using the dimension of the connection angle. References to the centers of the bolts will be deleted in future printings of both publications. T/2 is not a Specification requirement and is violated by connection designers as joint geometry dictates such as in a deeply coped beam. When this is done, it is important to be sure that the beam is laterally restrained by struts, bracing, metal deck or other means to guard against overturning.
Barry L. Barger
Vice Chairman
AISC Committee of Manuals, Textbooks, and Codes

AWS D1.1-92 Section 8.8.5 states, "Fillet welds deposited on the opposite sides of a common plane of contact between two parts shall be interrupted at a corner common to both welds." Is this necessary?

This is a comment on Richard W. Mudd's response (*Steel Interchange* August 1993) to a weld detail which showed an all-around weld symbol (*Steel Interchange* April 1993). He states that this violates Section 8.8.5.

AWS D1.1-92, paragraph 8.8.5 is ignored in the offshore industry in the North Sea and also Southeast Asia. Of my 20 years in steel construction supervision of probably 50,000 short tons of above water level steel Offshore structures all fillet welded members are always continuously welded around the perimeter. The reason is to seal the overlapping surfaces. With respect to paragraph 8.8.5, it shall continue to be ignored in the offshore industry unless qualified to allow seal welding.
Roger Steele
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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.

In designing the connection of a tubular beam to a tubular column for a required moment, the provision of AWS Chapter 10 were not met because the beam width was only a fraction of the column width. Can this connection be made by simply adding a plate to the end of the beam (larger in dimensions than the beam), and if so, what is an appropriate design approach to size the plate and the welds between beam to plate and column to plate.
Howard Epstein
The University of Connecticut
Storrs, CT

When welding to AWS D1.1 requirements what is a "seal" weld and what are the applicable inspection criteria for same?
Roy Hogan
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Knoxville, TN