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 responses to questions from previous Steel Interchange columns have been received:

Can one weld to an existing structure? How does one determine if the steel is weldable?

The answer to the first part of the question is a resounding yes, provided that certain condition are met. These conditions are enumerated in an article which appeared in the AISC *Engineering Journal*, Volume 25, No. 1, 1988. Interested readers are advised to access this publication.

The answer to the second part of the question (also available in the above mentioned literature) is summed up briefly as follows:

If the chemical properties of the steel to be welded are known, either by valid mill certification or laboratory testing of samples, the weldability of the steel can be predicted by use of a carbon equivalent formulae, of which there are many available.

An obvious test of weldability is to examine the existing steelwork for evidence of original welding. If one is still in doubt it is suggested that an on-site test be performed which will address weld ductility and hardening of the base metal. (Refer to AWS Spec. D1.1 Chapter 5).

In addition to determining the weldability of existing steel there are other factors which should be considered: the past history of the structure, nature of the loads, weather conditions, whether the welding involves members carrying stress, etc. The aforementioned article addresses these and other issues and provides useful references.

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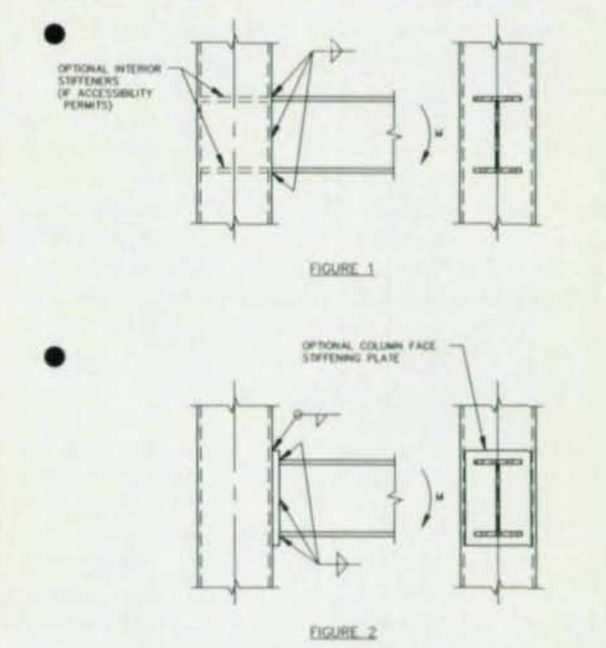
How should I connect wide flange beams to all four faces of a structural tube column in such a way as to transfer wind moments as well as dead and live load reactions?

There are a number of possible solutions for connecting wide flange beams to the face of a structural tube column to transfer moments. The type chosen depends on varying parameters. Economical determination of the proper connection will consider such things as the magnitude of the end moment



WF BEAM TO TUBE COLUMN CONNECTIONS

(flange forces), thickness and width of the column wall, and width of beam flange in comparison to the width of the connecting face of the tube.

The simplest of connections is to weld the beam directly to the column (see Figure 1). The static moment strength of this beam to column connection can be related to that of a gusset plate welded to the face of a square or rectangular tube. The moment capacity checks for this type connection will vary based on geometry.

Examples:

1. Connections with beam flanges which are the same width as the tube face should be checked for chord wall bearing or buckling failures.

Note: General collapse of the tube can be particularly severe in "cross" connections (connections with members attached on opposing tube faces) and should be fully investigated.

2. All connections should be checked for an effective flange width based on the width to thickness

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ratio of the column face.

3. Connections with flange widths less than the chord face width minus two times the chord wall thickness need be checked for "punching shear" criteria (severe in connections with low beam flange width to column width ratios).

Of course the beam web to column connection must be adequate to carry any beam end shear. I would recommend to readers that they obtain a copy of the book "Hollow Section Joints" by Jaap Wardenier, published by Delft University Press as a design guide for this type of connection.

If the preceding connection design will not work for members sized by classical analysis and design procedures, possible remedies may include thicker column walls, interior stiffeners if accessibility permits (see Figure 1), stiffening the column walls with face plates (see Figure 2), or changing beam size (depth or flange width). Another possible solution is to weld plates from flange-to-flange of the beam to form a "box" section at the connection, then use a design approach similar to that which is outlined for tube-to-tube connections in Chapter 10 (Tubular Structures) of the 1990 Structural Welding Code (AWS D1.1-90) to check the beam-to-column connection.

Tubes provide very economical structural compression members and we at Continental Bridge utilize them almost exclusively in design and fabrication of our pedestrian bridges; however, their economical use is greatly affected by connection details (tube to tube or wide flange to tube) and connection design must be thoroughly investigated to insure a safe, economical structure.

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Are there any design requirements that an engineer can follow when designing lateral bracing?

As indicated in the response to this question in the October 1992 issue of *Modern Steel Construction*, there is little information available on how to design bracing. Even in the publications cited, criteria are presented for only a few basic applications. Hopefully, this will soon change. On April 6-7 1993 there will be a 1½ day conference in Milwaukee on the theme "Is your Structure Suitably Braced?" The emphasis of the conference is on the determination of forces and stiffness required for bracing systems of beams, columns, frames and structural systems. The conference is sponsored by AISC, AISI, MBMA and SSRC. A call for papers appeared in the September issue of *Modern Steel Construction* and more information will appear in a future issue. One session of the Conference will be devoted to the discussion of questions raised by the participants.

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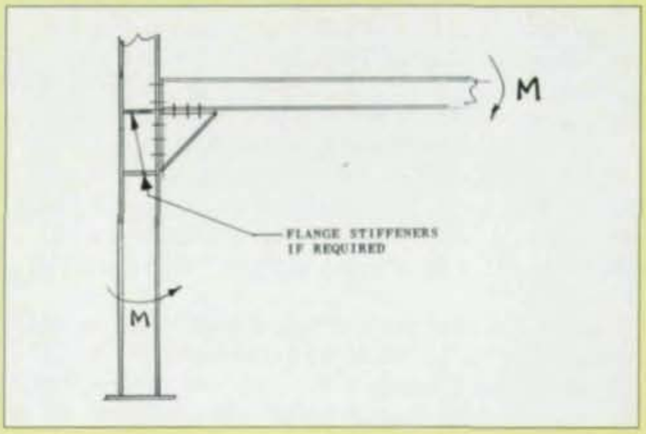
New Questions

Listed below is a question 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. 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.

What fatigue catagory is a full penetration weld between a tube column and a base plate?



The AISC and various authors present a multitude of solutions to the connection problem of a horizontal beam transmitting moment to a column. However, one solution I cannot find is the use of a heavily flanged gusset plate used below the beam to connect it to the column. This method would be very handy with industrial type structures for elevated hoppers,m bins, storage silos, as for stone aggregates, coal, cement, flyash and the like, which must make provision for trucks or railcars passing underneath, and where the moment is from wind or seismic forces, for the most part. Is such a concept not a proper or practical solution? If the concept is practical, what is a sensible calculation method, particularly for a bolted connection?