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 800/644-2400.

* * * * **Questions and answers can now be e-mailed to: aiscpmn@interaccess.com** * * * *

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

Are the Rules in the AISC Specification for Structural Steel Buildings appropriate to use when designing in a foreign country? How about when the material follows a foreign structural steel specification?

Typically, international contracts indicate which codes are acceptable and may allow for the use of alternate codes where they can be demonstrated as being the equivalent to those specified. As a general rule the AISC Specification is among those allowed or is quickly proven as acceptable. From my own experience in Southeast Asia the Japanese JIS, the German DIN, the British BS and the American AISC codes are preferred. When a designer chooses to complete designs based on AISC, the design will need to clearly indicate the code being used and specify that fabrication and erection practices must conform as appropriate. The designer should be cognisant that designing in AISC does not necessarily mean that all materials will conform to ASTM requirements and that erection will follow the specified AISC requirements. For example, two areas of concern for the designer and client are material strength and field bolted connections.

Before starting the design, confirm the specification of the material to be used. As an example, in Indonesia the majority of the available steel conforms to JIS G 3101 for strength and JIS G 3192 for dimensional tolerances. It is typical practice to request confirmation of available member sizes, material strengths and fabrication tolerances from the fabricator prior to design and to request mill certificates after fabrication. In this case the fabricator will use JIS G 3101 SS41 which is roughly equivalent to ASTM A36 but with lower yield strengths. As such, the designer will need to account for the strength differences in the design and realize that many of the AISC design aids will not apply when they are based on different material strengths. Additionally, the designer should note that the AISC code recognises material conforming to ASTM standards as listed in AISC Specification A3.1.a and the use of alternate materials should be evaluated and confirmed prior to use by the owner.

In contrast to the available steel members, ASTM A325 high strength bolts have proven to be readily available. The problem occurs during erection when many contractors will use ASTM A325 high strength bolts but fail to apply bolting guidelines of the Research Council on Structural Connections (RCSC) *Specification for Structural Joints Using A325 or A490 Bolts for slip-critical connections.* In several cases contractors have shown their preference to utilize the DIN code which allows for direct installation of bolts using a set torque value and therefore fail to satisfy the AISC and RCSC Specification requirements. To avoid this conflict many designers have chosen to specify tension indicating devices such as tension control bolts or tension indicating washers.

The validity of the design process presented by AISC is technically sound for the international marketplace provided the engineer ensures the commercial, or contractual, acceptability of this standard and recognizes and incorporates the actual material properties and construction practices which will be used.

J.H. Schnieders P.E.
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What is the maximum eccentricity allowed for a crane runway rail and girder (centerline of rail to centerline of girder)?

AISE Technical Report 13 (*Guide for the Design and Construction of Mill Buildings*) section 5.18.6.4 states "Crane rails shall be centered on crane girder webs whenever possible. In no case shall the rail eccentricity be greater than three-fourths of the girder web thickness." Section 5.18.6 also covers center to center of crane rail tolerances and maximum permissible misalignment in both the horizontal and vertical directions.

It would not be prudent to design a new crane runway with the rail eccentric from the girder web since fabrication and erection tolerances (horizontal sweep in the girder, column plumbness, etc.) could also contribute to rail/girder web eccentricity. Also, if a floating rail clip system is used, the rail is permitted to move laterally up to .25", further contributing to the eccentricity. Rail/girder web eccentricities induce torsional forces on the crane girder and will reduce the fatigue life of the girder web to top flange weld for plate girders.

If, during an inspection of an existing crane runway, the rail is found to be eccentric, the web to top flange weld should be inspected for signs of distress. There are numerous conditions which may lead to girder/rail eccentricities, including fabrication and erection tolerances mentioned above, failed top lateral tie connections, bowed girder top flanges due to overstress, failed rail clamp connections etc. Efforts should be made to determine the cause of misalignments, and the problems corrected. The rail should be realigned within AISE Technical Report 13 guidelines and, if eccentricity cannot be eliminated, an analysis of the girder should be performed accounting for torsional stresses due to eccentricity.

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ChemTech Consultants, Inc.
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There was a typographical error in the August issue of Steel Interchange:

The answer concerning re-entrant corners of beam copes correctly stated that the AISC *Manual of Steel Construction Load and Resistance Factor Design, Volume II* recommends that an approximate minimum radius to which the re-entrant corner of a beam cope must be shaped is $^1/_2$-in. radius. However, the error occurred where the answer stated that there is nothing magical about a $1^1/_2$-in. radius. This should have stated that there is nothing magical about a $^1/_2$-in radius.

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 can also be sent via e-mail to aiscpmn@interaccess.com.

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.

Under What Conditions, if any, is it acceptable to flame cut bolt holes, and what references substantiate this?

George R. Hoppen, P.E.
Feeco International, Inc.
Green Bay, WI

What design criteria is recommended for a circular bolt pattern in a moment connection, such as a splice joint in a circular column? I have used the Pressure Vessel Design Handbook, section 4.4.

Cindy Green, P.E.
Salt Lake City, UT

Does an unbraced trolley beam that is loaded on the bottom flange have the same buckling characteristics as an unbraced beam loaded on the top flange?

Robert W. McKenzie, P.E.
Baton Rouge, LA