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:

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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 Word 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 800/644-2400.

* * * * **Questions and answers can now be e-mailed to: grubb@aiscmail.com** * * * *

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

I have received our client specifications and they indicate that they require all bolt holes that are in galvanized steel to be 3/32" over sized and not the standard 1/16". Our fabricator has indicated that this is a very costly request when the codes indicated that only 1/16" is required, and the 3/32" would require retooling the fabrication line which in turn delays the fabrication. Is there any economic justification on the constructor's side that the holes be oversized 3/32" for mechanically galvanized bolts? Hot dipped galvanized bolts? In addition to the higher cost of fabrication all engineering standard detail drawings would have to be updated to indicated this over sized hole.

There is no technical reason for changing the hole clearance from 1/16" since this provides sufficient clearance for the coating on the part and the coating on the bolt. The coating thickness can be as much as 8 mils each, which would give a total of 16 mils of coating. The hole clearance of 1/16" is 62.5 mills which gives a large allowance for these coatings. There is no need to increase the clearance to 3/32" or 93.75 mils. The added costs to change to this higher clearance hole are another reason to avoid this change. The bearing surface under the bolt head will also be reduced with a larger clearance hole.

Tom Langill
Technical Director
American Galvanizers Association

What is the process and/or requirements for welding to A7 and/or A9 steel? Our office is working with a number of existing buildings of this era and would appreciate an answer, idea or reference.

An excellent reference on welding to existing structures is "Field Welding to Existing Steel Structures" by David T. Ricker (*Engineering Journal*, 1st quarter 1988). Where extensive or more difficult welding is contemplated, material samples should be taken and tested for chemical analysis so that a carbon equivalent can be determined. Such testing is not expensive. The carbon equivalent can then be used to help establish welding requirements such as required preheat and postheat and electrode type. Some erectors will weld a small tab plate to the existing steel, strike it with a sledgehammer, and see whether it bends or the weld fails. While simple and of some value, this is a crude test at best. A review of both the old ASTM standards covering A7 and A9 steel as well as older AWS welding codes which included provisions for their welding is also recommended.

Michael J. Garlich, S.E, P.E.
Collins Engineers, Inc.
Chicago

I am analyzing a structure which was fabricated in 1924. Am I correct in assuming that the material is ASTM A7? Where can I find the section properties for this material? In particular, I need the design properties for ship building channels, standard channels and angles from this era. Also, what is a good reference to use for determining the strength of riveted connections?

For a structure fabricated in 1924, it is likely that the material is either A7 or A9 steel. A "must have" reference when dealing with old structures is AISC's *Iron and Steel Beams 1873 to 1952*. It contains information on steel types as well as extensive tables of member shapes and properties. The strength of riveted connections closely follows that of bolted connections and is presented in many text books, particularly older texts such as *Structural Steel Design* by Lambert Tall, ed.

Michael J. Garlich, S.E, P.E.
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The 23rd edition of the Carnegie Steel "Pocket Companion," dated 1923, provides section properties for ship building channels, standard channels, and angles, as well as other shapes, from that era. It also gives steel material properties and has a section on stresses in rivets and pins.

The Carnegie "Pocket Companions," as well as the handbooks from other steel companies like Jones and Laughlin, are an excellent source for section properties. The AISC publication *Iron and Steel Beams 1873 to 1952* is a "must have" for those working with older structures. It is also very helpful to have a set of AISC Manuals starting with the first edition through the present.

Geoffrey H. Goldberg, E.I.T.
A. G. Lichtenstein & Associates
Monroeville, PA

Another response:

A good reference for riveted connections is *Guide to Design Criteria for Bolted and Riveted Joints*, 2nd edition, by Kulak, Fisher, and Struik, 1987.

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

Editor's notes:

Iron and Steel Beams 1873-1952 (publication number M003, $30 + s/h) and *Guide to Design Criteria for Bolted and Riveted Joints* (publication number P633, $79.95 + s/h) are available from AISC Publications at 1-800-644-2400.

Reprints of articles from the AISC *Engineering Journal* are available by calling AISC at 312-670-2400 and asking for *Engineering Journal* reprints.

LRFD Manual (Vol. II) on page 9-194 states that "If provision is made for ductility and stability, it follows from the lower bound theorem of limit states analysis that the distribution which yields the greatest strength is closest to the true strength." Can someone help to explain just how one "provides for ductility and stability" with an extended shear tab?

There are several factors which influence the ductility and stability of an extended shear tab (shear plate) connection:

1. Considerations for the degree of stiffness of the support, which may vary from flexible on one extreme to rigid on the other.
2. Whether the supported beam is or is not laterally supported.
3. The manner of loading—whether gravity or a combination of gravity and axial loading.
4. How the extended shear tab is laterally stabilized—usually, in the case of a column weak axis connection, by top and bottom transverse stiffeners.
5. A minimum height requirement for the shear tab.
6. A minimum plate thickness requirement to prevent lateral buckling.
7. A maximum plate thickness to assure that adequate ductility exists in the proximity of the extreme outer bolts.
8. The type of HS bolt used: snug-tight or fully-tensioned. Most column connections require fully-tensioned bolts, assuring the plate will yield before the bolts will fracture.
9. Whether standard holes or short horizontal slots are used.
10. The edge distance at the bolt gage.
11. The type of steel material used for the shear tab.

Other considerations include bolt shear, bolt bearing on the plate, shear yielding of the plate, shear rupture of the plate, block shear rupture of the plate, weld size and strength (the weld develops the plate).

These considerations are covered on pages 9-147 through 149 and 9-192 through 195 of Volume II of the LRFD Manual. Refer also to Astaneh et al, "Design of Single Plate Shear Connections," *Engineering Journal*, 1st quarter 1989.

David T. Ricker, P.E.
Consulting Engineer
Payson, Arizona

NEW QUESTIONS

Are there any references on the maintenance of weathering steel wall panels or structural members? Can weathering steel be painted?

For calculating the allowable bending stress on beams (ASD 9th edition, Chapter F) can I assume a pair of brackets welded from the top flange to bottom flange on both side of the beam serves as a lateral support to reduce the laterally unsupported length of the compression flange?

Emha Antariksa