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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.

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

Are there special requirements for the design of high-strength A325 or A490 bolts that are going to be in a high temperature area?

ASTM A325-89, paragraph 1.2 stated in part: "...where elevated temperature applications are involved, Type 1 bolts shall be specified by the purchaser."

However, this statement was removed from the 1993 update of the specification. This statement was all too encompassing , and not knowing the actual application for the bolts, it was determined that this statement should be removed. It is possible that the user could place these bolts under temperatures that may affect the physical properties of the bolts. An example being that they were not designed to be used in high-pressure, high-temperature service areas found in power plants.

ASTM A490 specifications do not address the use of these bolts in elevated temperature areas.

There are bolts manufactured for use in elevated temperature areas.

Not knowing the specific application or design requirements the bolts are being used for; look into the possibility of substituting these bolts with ASTM A193/A193M "Standard Specification for Alloy Steel and Stainless Steel Bolting Materials for High-Temperature Service" and ASTM A449 "Standard Specification for Quenched and Tempered Steel Bolts and Studs"

Tom Slovick
Industrial Steel
Mims, FL

If a W-shaped column is made up of three welded plates, how does one design the welds connecting the plates together?

In general it is best to weld along the full length of the web so one may ignore buckling of the individual elements, i.e. the web and flange. One must still consider buckling according to Table B5.1, Limiting Width-Thickness Ratios for Compression Elements, from the LRFD Manual of Steel Construction. Assuming there is no moment introduced into the column one should use the minimum weld size given in Tables J2.3 and J2.4 of the LRFD Manual.

If the column is subjected to bending stresses which may be due to eccentricities or a lateral load the weld must be designed to transfer the load between the web and the flange like one would do for a beam. The shear flow can be calculated using the formula:

$$\text{Shear Flow} = \frac{V_u Q}{I_x} \frac{kips}{in.}$$

The capacity of the weld is then calculated as:

Continuous Weld Design Strength = ϕR_{nw}

An example of this type of design can be found in Chapter 11 of Salmon and Johnson, Steel Structures. In most cases the minimum weld size will control.

James D. Palmer
Butler Manufacturing Company
Grandview, MO

For a continuous trolley beam with multiple spans and cantilevered ends what is the lateral unbraced length for the bottom flange?

One answer published in the July 1995 issue suggests that the lateral unbraced length is simply twice the cantilever distance. A more complete and exact solution to this problem is offered in a 3rd Quarter 1985 Engineering Journal paper by N. Stephen Tanner "Allowable Bending Stresses for Overhanging Monorails." By considering the ratio of the overhanging span to the adjacent interior span in developed equations, a less conservative answer may be computed.

Nestor Iwankiw
AISC, Inc.
Chicago, IL

When considering a point load on the standing leg of an angle, what provisions are there for determining the effective allowable member width?

The following two articles will give a practical approach including finite element study and the latter is a theoretical method for solving the problem of how to distribute a point load on a shelf angle.

Tide, R.H.R and Norbert V. Krogstad, "Economical Design of Shelf Angles," Proceedings, Symposium on Masonry: Design and Construction, Problems and Repair, Miami, FL, December 8, 1992, STP 1180, May 1993, American Society for Testing and Materials, Philadelphia, PA

Jaramillo, T.J., "Deflections and Moments Due to a Concentrated Load on a Cantilever Plate of Infinite Length." Journal of Applied Mechanics, March 1950, American Society of Mechanical Engineers, New York, NY

R.H.R. Tide
Wiss, Janney, Elstner Associates, Inc.
Northbrook, IL

Given a wall of sheet metal or plate subjected to fluid pressure and stiffened by same size parallel members spaced regularly, what section (or width) of the wall shall be used that contributes to the section of a stiffener? The stiffening member may be a flat bar, an angle, or a channel or any other section.

The section of wall that contributes to the section of a stiffener is defined by means of an effective width be which shall not exceed:

a) The geometric condition, that is the distance between center line of adjacent beams.

b) The shear lag condition, which may be estimated as $^1/_4$ of the effective beam span (length of positive moment area of the rib).

c) The stability condition of the plate between stiffeners (see "Specification for the Design of Cold Formed Steel Structural Members", AISI, 1986, B2.1).

With the effective width so defined a verification must be done of the resistance of welding between the sheet and reinforcement ("Coligon" stresses), and possible buckling due to weld spacing in the line of stress.

Miguel A. Dodes Traian
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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.

Is there any criteria, except direct field measurements by drilling holes, to determine the percent loss of capacity in steel bridge members due to weather exposure for the purpose of rating the truck capacity of bridges.

Mike Alomari
Wayne County
Sterling Heights, MI

Where should a control joint be located in a composite large composite floor? Should it be located over the top of either the girders or the beams? What happens to the strength of the shear studs if the concrete cracks over the shear studs? Does this crack go all the way through the slab to the top of the steel beam?

Can any beam be cambered without heating? Is there a slenderness limit for the web to prevent buckling of the web while cold cambering?

Is it acceptable to either mechanically galvanize or hot dip galvanize high strength bolts? Are there different requirements for the installation depending on how the bolt is galvanized?