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.

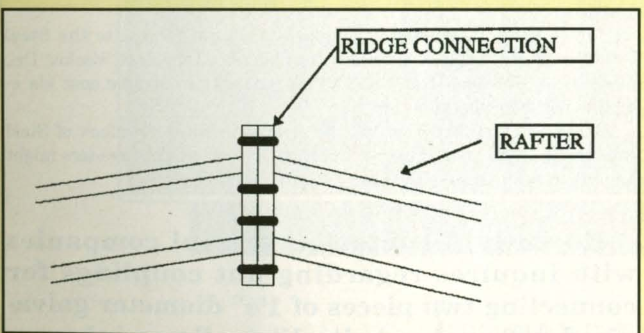
*** * * * Questions and answers can now be e-mailed to: newman@aiscmail.com * * * ***

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

Are there any published design aids or criteria for the design of a bolted moment ridge splice connection similar to the one shown? If not, would the tee stem analogy be an acceptable alternative to designing the plate thickness for the connection?

(Editors note: This question was also answered in the January 1997 issue of Steel Interchange)

When the angle between the rafters is relatively flat, the ridge connection can be treated as



an extended end-plate moment connection. Allowable Stress Design (ASD) procedure and Load and Resistance Factor Design (LRFD) procedure can be found in the AISC *Manuals*. Also AISC Steel Design Guide Series No. 4, *Design Guide for Extended End-Plate Moment Connections*, by Thomas Murray provides comprehensive information on this topic.

Wing Ho, P.E.
CU/H2A, Inc.
Princeton, NJ

Another answer:

There are several publications which could be used as references for the design of a bolted moment ridge splice, they are as follows:

Page 7-239 to 7-244 of *Structural Engineers (S.E.) License Review Manual*, Volume 3, 4th Edition published by PEDP (Professional Engineers Development Publications, Inc., ph: 714/898-3658; fax: 714/898-4635) has a solution to a problem which is similar.

The AISC Steel Design Guide 4, *Extended End-Plate Moment Connections*, by Thomas Murray; the 2nd edition of the LRFD *Manual of Steel Construction, Volume 2 Connections* on pages 10-21 to 10-35 for the design of end-plate moment connections; and, pages 856-860 of *Steel Structures: Design and Behavior*, 4th Edition by C.G. Salmon and J.E. Johnson, published by Harper-Collins.

The last three publications can be purchased through AISC.

Timothy M Young
Cumberland, VA

Is it permissible to accelerate cooling of structural steel after the application of controlled heat?

Because the maximum temperature permitted by LRFD Specification Section M2.1 for heat straightening, curving, or cambering is below any critical metallurgical temperature for the material being heated, the use of compressed air, water mist, or a combination thereof is permitted to accelerate the final cooling of the heated material, unless specifically prohibited in the bid documents. For members to be used in dynamically loaded (bridge) applications (i.e. where fatigue and toughness are design issues) it is recommended that such accelerated cooling not begin until the temperaure has dropped below 600 degrees F. This limitation is more historical than technical in nature. As a fair balance between the needs of the fabricator and the concerns of the owner, it provides an added safeguard to prevent the abuse of excessive cooling and undesirable residual stresses should accepted procedures not be strictly monitored.

When must high-strength bolts be ordered as a bolt/nut assembly from a single manufacturer?

(Editors note: This question was also answered in the December 1996 issue of Steel Interchange)

High strength bolts and nuts are not required to be manufactured by the same manufacturer, in fact, a number of manufacturers make only the nuts or bolts. The lubrication and testing of the assemblies noted in the December 1996 Steel Interchange answer are done after the bolts and nuts are brought together as assemblies for shipping to assure that the assemblies will provide the requried tension when installed.

Gerald E. Schroeder, P.E.
Federal Highway Administration
Columbia, SC

How can one evaluate the strength of a girder or column web or HSS wall with a single-plate connection or stiffened seated connection welded to it?

When such connections frame back-to-back on a girder or column web, the designer need only consider the total end reaction of the connections and ensure that the shear strength of the supporting material is adequate; any incidental eccentricity will be transferred through the support rather than into it, due to the higher relative stiffness of the framing beams. When framed to one side of the web, however, the concerns exist that the web may yield locally, reduce the column strength due to lacal deformations, or punching shear limit state may control.

Sherman and Ales in a 1991 National Steel Construction Conference presentation demonstrated that local yielding of the support was not a concern due to the self-limiting nature of simple connection end rotation and that member strength was unaffected by the associated local deformations. This same research indicated that punching shear may be of concern for relatively thin supporting material thicknesses. Accodingly, it is recommended that the minimum supporting material thickness be:

$$t_w \geq \frac{(F_y t)_{pl}}{1.2F_{uw}}$$

Thus, for A36 plate material, the minimum support thickness is then $0.52t_{pl}$ for A36 supporting material and $0.72t_{pl}$ for A572 Gr. 50 supporting material; for A572 Gr. 50 plate material, the minimum support thickness is $0.46t_{pl}$ for A36 supporting material and $0.64t_{pl}$ for A572 Gr. 50 supporting material. These minimum thicknesses would also be applicable to a welded plate tension connection (uniform stress distribution). However, for cantilevered bracket connections, which do not have self-limiting rotations; yielding must also be checked.

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 newman@aiscmail.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.

Recently, I contacted several companies with inquires regarding nut couplings for connecting two pieces of 1½" diameter galvanized A36 anchor bolts. Virtually no information pertaining to safe working load; catalog cuts showing size, shape and threaded dimension; or, ASTM material is available to the structural engineer from the nut and bolt industry. Please advise me of information sources.

Timothy E. Donovan, P.E.
North Weymouth, MA

Are special tolerances required to accommodate the cladding on structural steel frames.

When are notch toughness properties required for structural steel members.