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.

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.

If you have a question or problem that your fellow readers might help you to solve, please forward it to us. At the same time, feel free to respond to any of the questions that you have read here. Contact *Steel Interchange* via AISC's Steel Solutions Center:

Steel SolutionsCenter
One East Wacker Dr., Suite 3100
Chicago, IL 60601
tel: 312.670.2400
fax: 312.423.4651
solutions@aisc.org

ROOF DECK PROVIDING BRACING

from August 2001 Steel Interchange

I am working with some 40- to 50-year old buildings (ASTM A7 steel). The roofs are metal deck about 1 to 1.5 inches deep. What parameters do I use to assume that the supporting purlins are laterally braced by the roof deck? Should I even consider this as neither the welding of deck is verified, nor is the weld size or spacing?

Question submitted anonymously

If there is no connection between the deck and purlin, then the purlin certainly is not braced. If there is a connection, then the adequacy of the deck as a brace has to be checked for strength and stiffness. Most engineering textbooks cover this. The old "rule of thumb" of 2% for the brace would mean the deck should have at least 2% of the purlin strength could be conservatively used. Stiffness of the deck should meet an appropriate KL/r limit.

Robert Lorenz, P.E.
AISC Alumnus
(Answer from August 2001 Steel Interchange)

To supplement Mr. Lorenz's comment, remember that the 2% rule is an approximate guideline and was developed under the assumption that one point of lateral load resistance is bracing the flange of a beam or column. When there are multiple points of lateral support, such as a deck attached to a joist or beam through welds, then that percentage drops dramatically at the point of lateral support.

Rick Ehlert
Colorado

Editor's Note: The 1999 LRFD *Specification* provides a means to assess bracing strength and stiffness in Chapter C3. Many typical configurations are covered.

SHEAR TABS

Are there any recommended procedures for designing single plate shear connections requiring multiple lines of bolts or a deep single line of bolts?

Question sent to Steel Solutions Center

Single plate connections must be configured to provide sufficient rotational flexibility and ductility to accommodate the rotational demands of a simply supported beam. AISC design tables are all for a single vertical row, because the method used in creating those tables is limited to one vertical row. The 3rd edition LRFD *Manual* now contains tables for single rows of up to 12 bolts, and for bolt diameters from $^3/_4$" up to $1^1/_8$".

If you are interested in the behavior of these connections (and research conclusions), the following two *Engineering Journal* articles would be useful:

- Abolhassan Astaneh et al, "Design of Single Plate Shear Connections." *Engineering Journal*, First Quarter, 1989, pages 21-32.
- Abolhassan Astaneh, Discussion of "Design of Single Plate Shear Connections." *Engineering Journal*, Third Quarter, 1990, pages 122-126.

Reprints are available at www.aisc.org/ejreprints.html.

The use of multiple vertical rows of bolts in shear tabs is not addressed by the *Manual* method. However, many such connections are designed using statics, strength of materials and engineering judgement. There is at least one University of Texas–Austin research report that was sponsored by AISC that might be helpful: "The Behavior and Analysis of Double Row Bolted Shear Web Connections" by J. M. Ricles and J. A. Yura. Reprints are available for a nominal cost through AISC's Steel Solutions Center at:

solutions@ aisc.org.

AISC is currently finalizing research and preparing a design guide on shear tabs that includes the extended configuration of this connection, which will simplify design and increase usage of this connection.

Charles J. Carter, S.E., P.E.
American Institute of Steel Construction
Chicago, IL

TRACEABILITY AND IDENTIFICATION

reprinted from www.aisc.org/faq.html

What is the difference between traceability and identification of material?

Traceability means the ability to identify a specific piece of steel in a structure, throughout the life of the structure, and its specific CMTR. As such, traceability requirements are significantly more expensive than the identification requirements in 2.1.1 [see question 2.1.1 at www.aisc.org/faq.html for these requirements]. The owner should clearly understand the differences, limitations, and relative costs involved.

Traceability is not a requirement in the AISC LRFD *Specification* and, when required, must be clearly specified in the contract documents prior to the ordering of material. The following elements of traceability should be selected only as needed:

1. *Lot traceability vs. piece-mark traceability vs. piece traceability:* Lot traceability means that the materials used in a given project can be traced to the set of CMTR's for that project. Piece-mark traceability means that the heat number can be correlated for each piece mark, of which there can be many individual pieces. Piece traceability means that the heat number can be correlated for each piece, which effectively demand separate piece marks for each piece.

   Each of these three successive levels of traceability adds significant costs. Piece traceability, the most expensive option, is necessary only in critical applications, such as the construction of a nuclear power facility. Piece-mark traceability is often specified for main members in bridges. Lot identification is most common in other applications where traceability is required.

2. *Main-material traceability vs. all-material traceability:* Main-material traceability means that beams, columns, braces, and other main structural members are traced as specified above. All-material traceability means that connection and detail materials are also traced as specified above. All-material traceability, the more expensive option, is necessary only in critical applications, such as the construction of a nuclear power facility. In other cases, main-material traceability is sufficient, when traceability is a requirement.

3. *Consumables traceability* means that lot numbers for consumables such as bolts, welding electrodes, and paint can be traced. This is necessary only in critical applications, such as the construction of a nuclear power facility.

4. *Required record retention* defines the level of detail required in documenting traceability (who, what, when, where, how, etc.)

5. *Fool-proof record retention vs. fraud-proof record retention:* Fool-proof record retention means internal verification of records. Fraud-proof record retention means external certification of records. Fraud-proof record retention is necessary only in critical applications, such as the construction of a nuclear power facility. In other cases, fool-proof record retention is sufficient, when traceability is a requirement.

Answer reprinted from www.aisc.org/faq.html

A325 AND A490 BOLTS

I have a situation where I would like to use some relatively small diameter high strength (A325 or A490) bolts. What is the smallest diameter bolt that I can specify?

Submitted anonymously

Both the ASTM A325 and the ASTM A490 Specifications covers diameters as small as 1/2 inch. Note that since 3/4 inch high strength bolts are the smallest bolts routinely used in structural steel applications, availability of the smaller diameter bolts should probably be confirmed prior to their specification.

Keith Mueller, Ph.D.
AISC Steel Solutions Center
Chicago, IL

NEW QUESTIONS

CAMBERING EQUIPMENT

Who manufactures equipment for cambering steel?

Cheryl Vickroy
Research Fellow
Madison, WI

WEB PANEL-ZONE SHEAR

In the 1992 and 1997 *Seismic Provisions*, for SMF, the resistance factor for panel-zone web shear is 0.75. The *Seismic Provisions* are somewhat silent for panel-zone web shear in OMF. LRFD Specification Section K1.7 using a resistance factor for panel-zone web shear of 0.90. For OMF, do we default to Section K1.7 and use a resistance factor for panel-zone web shear of 0.90? Or is the resistance factor always 0.75 in OMF and SMF if the loading is non-static?

Stephen Crockett
D. M. Berg Consultants, P.C.

HEIGHT-THICKNESS RATIOS

Referring to LRFD Specification Sections F2.2, Appendix F2.2, and Appendix G.3:

For all the standard rolled W- shapes, is the h/t_w ratio always ≤ 260? In other words, if a standard rolled shaped is being considered, is it necessary to check for the limit states of web shear yielding or bucking? Also, for all the standard rolled W- shapes utilizing up to 50 ksi specified minimum yield strength, is it always true that:

$$h/t_w \leq \frac{418}{\sqrt{F_y}}$$

Stephen Crockett
D. M. Berg Consultants, P.C.