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 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:

(From December 1998)

My question is regarding the shear friction design method used to transfer column shear forces to foundation systems. AISC Design Guide #1 (Column Base Plates) gives friction coefficients for use in determining frictional shear resistance. The friction coefficients are based on ACI 349-85. The design guide states that these friction coefficients are for limit state conditions and that a factor of safety of 2.0 should be used with these coefficients for ASD. In comparison, AISC Design Guide #7 (Industrial Buildings) also outlines the shear friction design method but makes no mention of the factor of safety nor is one used in the design example. Please clarify if a factor of safety on the friction coefficient for ASD is required or not.

William B. Kussro, P.E.
Giffels Associates, Inc.
Southfield, MI

There is no inconsistency between the examples in Design Guide No. 1 and 7. The commentary for each explains that friction can be developed by either column axial loads or by interaction with shear and tension in the anchor bolts. The example in Design Guide No.1 uses column axial load in the example problem while the Design Guide No.7 example has uplift in the column and the design uses anchor bolt shear and tension to develop the shear friction.

When using the column axial load method a factor of safety should be applied similar to what is done when designing footings for uplift. When designing for shear friction due to anchor bolt interaction as in Design Guide No. 7 the safety factor is already in the ASD anchor bolt design.

The design examples are for two different shear friction systems but have similar safety factors.

Lawrence A. Kloiber
Le Jeune Steel Company
Minneapolis

What is the weak axis effective unbraced length (K_yL_y) for a floor beam (W24x55 for example) subjected to axial load? The floor beam is loaded in compression along the neutral axis and is loaded in bending about the major axis via floor deck connected to the top flange. The beam has no perpendicular members framing into it.

Karl Menches, P.E.
Greenville, SC

In the weak axis, the beam-bending mode is analogous to a column since it's loaded in compression along its neutral axis. For a pin-ended column (whose ends can rotate but cannot translate) the points of zero moments are at the ends a distance L apart. Therefore, the weak axis effective length equals the beam length (Ky = 1.0).

Sam Babatunde, P.E.
Engineering Dynamics & Associates
Edgewood, PA

Treating the beam as a column with Ky = 1.0 is conservative. I would use rT, not ry, in calculating the slenderness ratio Kl/r. A more accurate approach is given for this case of a beam column having the top edge continuously braced by the floor slab in Section 12.8, p. 467 of "Guide to Stability Design Criteria for Metal Structures, 5th Ed." (Theodore V. Galambos, Ed., John Wiley and Sons, 1998)

Kenneth B. Wiesner, P.E.
LeMessurier Consultants, Inc.
Cambridge, MA

(From January 1999)

"Mill to bear" is a term often used in contract drawings and specifications. What precisely is the definition of "mill to bear", especially as it relates to *AASHTO Standard Specification for Highway Bridges* (16th Edition) and AWS D1.1 (1996)?

While our drawings do not call the parts "stiffeners", the closest we can come to the above question is paragraph 5.23.10 in D1.1. Because our contract does not reference AISC, Section M4.4 of the LRFD Specification (2nd Edition) is not being recognized by our customer.

Jim Tyvand, P.E.
ADDISON Corp.
Bend, OR

First of all, Section 11.1.1, Division II Construction, AASHTO Standard Specifications for Highway Bridges, 16th ed., notes that welding and weld qualification tests shall conform to the provisions of the current ANSI/AASHTO/AWS D1.5 Bridge Welding Code. WSDOT Standard Specifications permit AWS D1.1 for welding of other steel structures and not for steel bridges. AASHTO Standard Specifications for Highway Bridges also requires D1.5 for steel bridges.

"Mill to bear" is defined in Section 11.4.4, Fit of Stiffeners, Division II Construction, AASHTO Standard Specifications for Highway Bridges, 16th ed. "End bearing stiffeners for girders and stiffeners intended as supports for concentrated loads shall have full bearing (either milled, ground...)" Section 11.4.6 defines surface finish of bearing surfaces according to ANSI B46.1, Surface Roughness, Waviness and Lay, Part I: milled ends of compression members, milled or ground ends of stiffeners and fillers shall be ANSI 500.

John A. Van Lund, P.E.
Washington State Department of Transportation
Olympia, WA

The outdated term "mill to bear" comes from a time when cutting methods were not as advanced as they are today and cut surfaces required further treatment in contact bearing applications. As noted in AISC's publication, A Guide to Engineering and Quality Criteria for Steel Structures, Common Questions Answered, p. 7, all it really means is that the surface should be "finished" (modern terminology). This corresponds to an ANSI roughness height value not greater than 500. It is further noted in that publication that most modern cutting methods, including cold-sawing, will generally produce such a surface without further treatment.

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

What is the minimum concrete cover required above the head of a shear stud connector?

At one time, such a requirement did exist in the AISC Specification. However, the strength of a shear stud connector depends upon the strength of the connector itself and the cone of concrete under the head, not the concrete over it. Accordingly, the cover requirement was eliminated. The reader is cautioned, however, to allow sufficient slab thickness to accommodate possible variation in composite beam shape, such as that due to camber, which could otherwise cause shear stud connectors to protrude through the top of the slab.

NEW QUESTIONS

The AISC Manual indicates that design strengths tabulated for clevises and turnbuckles are calculated using f= 0.3 in LRFD or a factor of safety of 5 in ASD. The Manual indicates that this conservative reduction is used because these devices are most often used for temporary rigging which may be subjected to dynamic and impact loading. When these devices are used in permanent applications and not subjected to these considerations, e.g. as part of the permanent bracing system, is it justified to use f =0.5 in LRFD or a factor of safety of 3 in ASD?

In a large composite slab, construction joints may be required. Where should one put construction joints in composite floors?

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