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 at:

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Crane Design

I would like to know if there's a reference similar to what could be called "Crane Design for Dummies." I am looking for a handbook or manual on cranes for industrial buildings. Subjects mainly needed are bridge beams, runway beams, columns, bracings, brackets, load considerations, load combinations, etc. Crane information is not relevant because the supplier or manufacturer supplies it all. I would like a reference that focuses on the structure that supports them.

Question from September 's Steel Interchange

Based on my experience a very key item was left out of the crane girder design considerations answered in September's Steel Interchange. Most engineers can properly size crane runways. What they do not do is provide connection details that provide for both the proper force transfer and the alignment adjustment needed for the erection of the girder and installation of the rail to the required tolerances. The steel frame is normally erected to AISC tolerances or in some cases to AISI tolerances which may be less but these are both substantially larger then the rail alignment tolerances. If the girder is erected to AISC tolerances, there is a very real possibility that the rail will be offset enough from the girder centerline to cause flange bending and torsion. The girder support connections must be adjustable enough to allow the girder to be erected to a more restrictive tolerance then the steel frame in order to allow proper installation of the rails.

Larry Kloiber
LeJeune Steel
Minneapolis, MN

Welding Through Metal Deck

Can headed studs be welded thru 18 ga. metal deck without having the deck pre-punched?

Question from SEAINT list server (www.seaint.org)

Sure, it's done all the time. I believe the issue is addressed in ICBO approvals for composite structural decks, which indicate that such studs can be assumed to replace on a one for one basis 3/4" nominal puddle welds that might otherwise be required for shear transfer from the deck to the supporting framing. Check with your deck supplier.

Answered by an engineer on SEAINT list server
Pasadena, CA

Biaxial Bending in Beams

When calculating biaxial bending in beams, we generally use ASD equation H1-3: $f_a/F_a + f_{bx}/F_{bx} + f_{by}/F_{by} <= 1.0$ which in many cases, allows us to use $0.66F_y$ as F_{bx} and $0.75F_y$ as F_{by}. However, we've recently reviewed calculations where F_{bx} and F_{by} are $0.60F_y$, no matter what the compact length attributes of the beam are. Is the $0.6F_y$ for biaxial bending standard?

Question from SEAINT list server (www.seaint.org)

No. The ratios in the interaction check represent the percentage (fraction) of the total member strength used up by each component of the load. The strengths (F_a, F_{bx} and F_{by} in this case) used in this check must each be calculated as though the member were subject only to that individual component of the total load. So the unbraced length will very much affect which column-only or beam-only equation must be used.

Answered by an engineer on SEAINT list server
Chicago, IL

IBC 2000 and AISC Seismic Provisions

In South Carolina we are using the provisions of IBC 2000 and most of the state's structures are now under seismic design category D. Section 2212.1.2 requires the use of AISC's *Seismic Provisions for Structural Steel Buildings*, dated April 15, 1997. If we design using Part III of this document (page 135, Part III Section C4.1), it encourages the use of the governing code or standard for the load factor on *E*.

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This governing code is now IBC 2000. If we use the alternate IBC 2000 load combinations of section 1605.3.2, is the intent that the combinations of formula 16-17 and 16-18 be used now instead of the equations 4-a and 4-b in AISC's *Seismic Provisions*? Please verify if the nominal strength per section 4.2a is to be used for equations 4-a and 4-b and 4-1 and 4-2 that it is ok to use this nominal strength with formulae 16-17 and 16-18 of the IBC 2000 (and not use any other increase factors). Paragraph 1605.3.2 of IBC 2000 states that stresses are permitted to "be increased where permitted by .... the referenced standard." What increases do the steel standards (seismic) allow for the formulae in section 1605.3.2? The IBC 2000 says in section 1617.1.2 to use an increase of 1.7 and ϕ of 1.0 in ASD for E_m in equations 16-30 and 16-31. Do these take the place of formulae 4-1 and 4-2 in AISC's *Seismic Provisions*, or should this be checked and also 4-1 and 4-2 with the actual resistance factors for steel as per section 4.3 on design strengths?

Question sent to AISC's Steel Solutions Center
Greenville, SC

I presume that the reference to equations 4a and 4b is to 4-1 and 4-2. If that is the case then the intent of 4-1 and 4-2 is to require the use of an amplified load on seismic Ω_0. These equations are intended to be the same as IBC formulae 16-19 and 16-20, respectively. Section 4.2.a Part III of AISC's *Seismic Provisions* allows the user to establish the nominal strength and gives the appropriate ϕ factors that must be used to establish the design strength required by Part I of the AISC Specifications. The design procedure necessary to comply with the loads in formula 16-17 and 18 of the IBC are those found in the AISC ASD *Specification*. IBC formula 16-30 & 31 establish the maximum seismic load effect. These can be used with either IBC formula 16-19 and 20 or AISC formulae 4-1 and 4-2. There is not need to check both of the IBC 16-19 & 20 and AISC 4-1 & 2 either one will suffice.

Answered by a consulting engineer in California.

Some Still-Unanswered Questions

Cambering Galvanized Members (August 2001)

Are there any special considerations that need to be made in specifying camber in a beam that is also to be hot-dip galvanized?

Question submitted anonymously

Braced Frames and 1997 UBC (August 2001)

Can someone refer me to a design guide for the connection of steel braced frames to footings using the 1997 UBC? I am looking for some recommended connection configurations. I am particularly interested in what load factors I should be using and how to treat the grout space and oversized anchor rod holes.

Chris A. Hasse, P.E.

Anchor Rods too Short (September 2001)

Are there any guidelines or recommendations concerning the repair of anchor rods without adequate projections? This question applies particularly to applications in rigid frames and braced frames where tension is a limiting design condition. Also these are applications where epoxy anchors are not applicable. We know of several methods of repair - couplers or cutting and welding bolt projections. Could you supply some information on the applicability of each repair—minimum/maximum size of anchor, minimum/maximum projection, minimum/maximum plate size?

Kurt Swensson
KSI Structural Engineers
Atlanta, GA

Stiffener Requirements (September 2001)

Regarding Chapter K of the ASD Manual, reference page 5-82, Section K1-8, Paragraph 3:

If Sections K1.4 or K1.6 require stiffeners, the stiffeners shall be designed as axially compressed members (columns) in accordance with the requirements of Section E2.

If Section K1.4 requires the stiffener, I would design the stiffener as a compression member with an axial load of R from section K1.4. If Section K1.6 requires the stiffener, should the stiffener be designed for an axial load of P_{bf} from section K1.6 or from the computed force delivered by the flange? If P_{bf} is used, often the stiffener (assuming the same width as the flange) will be thicker than the flange and this appears odd to me.

I would appreciate any information you could supply me concerning this information.

Paul Howell