steel interchange

IF YOU'VE EVER ASKED YOURSELF "WHY?" about something related to structural steel design or construction, *Modern Steel Construction*'s monthly Steel Interchange column is for you! Send your questions or comments to solutions@aisc.org.

Washers for Anchor Rods

What type of nuts and washers are required for ASTM F1554 anchor rods? Table 14-2 in the 13th Edition AISC *Steel Construction Manual* states that ASTM F844 washers may be used if the base plate is less than 1.25 in. thick, using a reduced hole size, with no limits on tension. Section 2.6 of AISC *Steel Design Guide 1* says to use ASTM F844 washers only for compression with a reduced diameter hole.

There is a table in ASTM F1554 Section 6.6.1 with the minimum requirements for nut specifications and grades. These vary by rod grade, diameter and galvanization requirements. You are correct that ASTM F844 washers should only be used for compression with reduced hole diameters. The part of *Manual* Chapter 14 that covers washer requirements for axial tension (page 14-10) confirms that the thicknesses given in Table 14-2 are minimums and that the "thickness must be suitable for the forces to be transferred."

Heath Mitchell, P.E.

Continuity Plate Welds

I recently designed a special moment frame using a prequailified RBS moment connection in accordance with AISC 358. The steel fabricator proposed to use an electroslag welding system for the moment frame continuity plates and submit a PQR for AWS D1.1 Electroslag Welding Process (ESW). Are the complete joint penetration groove welds of continuity plates to column flanges considered demand critical welds in accordance with AISC 341? If the welding of the continuity plates is not a demand critical weld, should ESW be permitted? If the weld of continuity plates is demand critical, what documents should be submitted by the steel fabricator to meet the criteria in accordance with AISC 341 Appendix W6?

The welds to the continuity plates are *not* demand critical per the AISC *Seismic Provisions*. However, if the ESW process were to be used for demand critical welds, the Commentary to Section W6.1 in AISC 341 states, "For processes such as ESW and EGW, the heat input level is considerably higher than that of the other four processes, and there has not been general testing proving the acceptability of these processes for *demand critical welds*. However, these processes may have had limited connection qualification tests performed for certain applications, and their use in such applications may be approved by the engineer."

Section 2.6 of AISC *Steel Design Guide 21* also has a good discussion of ESW. You can find this as a free download for AISC members at www.aisc.org/epubs. Non-members may purchase it there as well.

Also, for the sake of completeness, even though these welds are not demand critical, they still must satisfy the requirements of Sections 7.3 and 7.3a of the *Seismic Provisions*.

Larry S. Muir, P.E.

Structural Steel Utilization

I am trying to determine amount of steel used in the various construction sectors. I have the overall number of tons for construction, but am interested in getting the information at the sector level (lodging, commercial, bridge, etc.). Does your organization track this?

We do track structural steel utilization by building type through measuring the percentage of construction starts in a given year by project type and the structural steel market share in each category. We only track structural steel usage (hot rolled shapes and hollow structural sections). These numbers do not include other types of steel used in construction such as reinforcing bar, plate used to create girders, piping, cold formed steel used for studs and framing and such—these are *just* structural sections.

We break out the use of structural sections into three major categories: buildings; industrial construction not under roof (structures like petroleum refineries, power plants, industrial process plants); and non-structural uses (OEM use of structural sections for trailers, ship building, etc.).

The 2009 data are rather skewed as a result of the recession. In a more typical year we would have seen building construction represent closer to 60% of the structural steel market rather than the current 44%.

The breakdown for 2009 is:

	% of Building Construction Market	Structural Steel Tonnage
Buildings		1,892,000
Retail	11%	250,384
Warehouse	4%	75,691
Offices	17%	363,646
Parking/Auto	10%	63,076
Manufacturing	1%	15,906
Schools	19%	474,166
Health Care	6%	131,636
Government Services	5%	94,614
Religious	3%	46,895
Amusement	4%	76,788
Miscellaneous	3%	61,705
Hotel/Motel	2%	35,103
Dormitories	2%	44,976
Apartment/Condos	14%	157,415
Industrial		1,463,000
Other non-structural applications		946,000
Total Structural Shapes		4,300,000

John P. Cross, P.E.

For the bridge market, we estimate that the 2009 structural steel usage (shapes and plates) was 362,000 tons.

Brian A. Raff

Concentrically Braced Frame Design

Our office practice for concentrically braced frames is to include a redundant moment frame within the braced bay. We typically ask for the following with respect to the lateral system connections:

- **Braced connections to be designed for the full tensile capacity of the brace ($0.6A_gF_y$)**
- **That the beam-to-column connection to be designed for the full elastic capacity of the beam ($0.66S_xF_y$)**
- **That the scheduled beam shear be increased by ($0.66S_xF_y$) ×2/(length of girder)**

Can you please comment on this practice?

It is usually recommended that connections be designed for the actual forces rather than arbitrarily higher forces. This is recommended to promote economical steel designs, because it is the cost of connections that drives the cost of the structure. It is likely that your arbitrary requirements will increase the cost of the structure.

You may feel it is necessary or good practice to design connections for forces in excess of the required loads, but care should be used. Requiring connections to be designed based on member strengths can lead to numerous problems. You obviously will pay a premium for the connections because more material and labor will be required. Also, the members probably will have to be reinforced in order to carry your listed design loads. This is especially true if the connections are bolted or if shear lag is a factor. Unless this reinforcing is either clearly shown on the drawings or is clearly indicated as not required, you may incur extra charges for this work.

As far as increasing the shear, the shear from the moment should already be present in your analysis and reflected in your design loads. Including it twice does not make sense. It could also lead to unnecessary reinforcement of beam webs and larger shear connections. Again this could lead to extra charges if the typical details do not accurately reflect connections that will satisfy your requirements. It also could lead to RFI's trying to clarify your intent.

You should strive for the most economical structure possible, while maintaining safety. This means designing connections for the actual loads. If you are not comfortable with this, the key is to make sure all the requirements are clear during the bid stage. General statements concerning the design loads are probably not sufficient, because estimators are not engineers, and time will not permit the connections to be designed at the bid stage. Your details must also reflect your intent and should include reinforcement or unusual details necessary to satisfy the larger loads.

Larry S. Muir, P.E.

R = 3 System

We are designing a four-story building. Our understanding is that using an *R* of 3 with no special detailing is the minimum required by the code. Can you please confirm this for us?

Designing for $R = 3$ with no special detailing requirements per the *International Building Code* (IBC) will require higher design forces in some instances, but will usually result in a more economical structure. This option is only available in Seismic Design Categories A, B and C.

Larry S. Muir, P.E.

Fillet Welding of Studs

I am reinforcing an existing composite steel beam with additional ¾-in. diameter steel shear studs. Typically, I specify that the shear stud welding must be done with a stud welding gun. On this small project, the contractor would prefer to use fillet welding instead. Is a fillet weld a recommended equivalent substitute for stud gun welding? If yes, is reference to AWS D1.1 adequate to ensure quality, or are there some other specific recommendations which should be specified?

AWS D1.1 Section 7.5.5 does allow prequalified fillet welds to be used to attach shear studs. There are additional requirements, as well as minimum weld sizes given in this section. Additional qualification testing may be required for studs fillet welded through metal decking.

Heath Mitchell, P.E.

Unbalanced Loads on OCBF

I have a project where we are using V-type Ordinary Concentrically Braced Frames. We are using Section 14.3 of AISC 341 for the design. Section 14.3 (1) discusses the requirements for the design of the beam in an OCBF. These requirements include a check of the unbalanced force that will occur due to a tension brace force of $R_yF_yA_g$ and a compression brace force of $0.3P_n$. Does that requirement apply to OCBFs even if they meet the slenderness limits of Section 14.2?

The requirements apply whether or not they meet the slenderness limits. The actual load in the brace is likely to exceed the design load. By using the *R*-factor you are taking advantage of ductility to reduce the load below that which might otherwise be expected in an earthquake. When you consider this and the considerable drift expected, the compression brace is almost certain to buckle and cause the unbalanced load you are being asked to consider.

Larry S. Muir, P.E.

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John Cross is vice president of marketing at AISC. Brian Raff is marketing director for the National Steel Bridge Alliance. Heath Mitchell and Larry Muir are consultants to AISC.

Steel Interchange is a forum 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 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 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 700
Chicago, IL 60601
tel: 866.ASK.AISC • fax: 312.803.4709
solutions@aisc.org