

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!

Weak-Axis Moment Connections

When designing a weak-axis moment connection (a beam connecting into the web of a column), where can I find information or an example calculation for this type of connection?

The Appendix of AISC's *Design Guide 13—Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* contains a paper that outlines design considerations for weak-axis moment connections, along with a discussion of tests conducted by Driscoll and Beedle and recommended details.

Those tests were conducted to determine proper stress transfer, as explained in the *Engineering Journal* paper titled "Suggestions for Avoiding Beam-Column Web Connection Failure" by Driscoll and Beedle (first quarter, 1982).

You can download a copy of this design guide and the paper from www.aisc.org/epubs (free for AISC members and ePubs subscribers) or purchase a copy at www.aisc.org/bookstore.

Sergio Zoruba, Ph.D.

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Re-Entrant Corners

Is it required or recommended that the inside corners of blocked flanges (where beam flanges must be trimmed back to fit between column flanges) have radii similar to vertical web coping?

Refer to Section M2.2 of the 1999 *LRFD Specification*, as it contains provisions for thermal cutting. It states that re-entrant corners, except those from beam copes and weld access holes, shall meet the requirements of AWS D1.1 Section 5.16. The AISC *Manual* recommends that an approximate minimum radius of $\frac{3}{8}$ " is acceptable. AWS D1.1 requires re-entrant corners of cut material to be formed to provide a gradual transition with a radius of no less than 1".

Bill Liddy

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OCBF Lacking Diaphragm Action

The question is concerning an OCBF system as specified in the AISC *Seismic Provisions* (May 2002). We are designing a building with horizontal braced frames (OCBF) as well as vertical braced frames, and there isn't a continuous diaphragm on which to rely for lateral shear transfer to the vertical elements. The diaphragm is made up of horizontal braced frames, and frequently there is just a single member making up the diagonal of the horizontal braced frame that serves as the load path.

In designing a horizontal braced frame, is it a requirement to still design the connection of the brace for the full tensile capacity of the brace ($R_y F_y A_g$) or is it possible to consider the member a collector and just design for Ω times the force in the brace?

In this case, the bracing is what makes the horizontal elements into a diaphragm. A metal deck and concrete system does the same thing without the explicit diagonal element (using a shear panel, which actually does have an effective compression strut in it). So, the horizontal bracing and its connections should be designed for omega-level forces (that is, to remain nominally elastic). $R_y F_y A_g$ -type calculations only apply to vertical systems, which have elements expected to deform and thereby dissipate energy.

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Anchor Rods and Load Path

I am designing a six-story braced-frame building with an eccentric brace. I have very large loads. I am using 2"-diameter anchor rods of ASTM A449 material, but I have also used ASTM F1554 anchor rods on a previous project. What is the recommended material anchor-rod specification? What design criteria are recommended for designing the base for these forces?

To answer the first part of your question, ASTM F1554 is the preferred material specification for anchor rods since it puts all anchor rod material requirements together in one place. It contains three grades, namely 36 ksi, 55 ksi and 105 ksi. Each grade is color-coded for easy identification and the 55 ksi grade can be welded using Supplementary Requirement S1 (see ASTM F1554 for details). These are some of the benefits of placing anchor rods under a single material specification umbrella. Read on for the answer to the second part.

Sergio Zoruba, Ph.D.

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Regarding how shear forces actually get from the column or brace into the foundation, there are several possible mechanisms:

1. friction between base plate and supporting grout or concrete.
2. bearing between base-plate holes and anchor rods.
3. shear keys.

Mechanism 1 is probably the initial load path, especially if the anchor rods have been pretensioned. Unless the shear force is accompanied by enough tension and/or overturning moment to completely "uplift" the base plate, this mechanism will probably resist the entire shear force. However, friction cannot be considered when resisting code earthquake loads and another design calculation method must be used.

Mechanism 2 usually is considered in design and is probably sufficient consideration for light shear loads. It represents the shear limit state if the base plate has overcome friction and has displaced relative to the anchor rods. The anchor rods are usually checked for combined shear and tension. You also could check the anchor rods for bearing, but usually the base plates are so thick that this is not a problem.

Mechanism 3 should be considered for heavy shear loads, although welding and construction issues are raised. If a shear key is used, it is probably both the initial load path and the

shear limit state. If tension and/or overturning loads are present, anchor rods also need to be provided to resist tension forces.

There also might be some load transfer by bearing between the far edge of the base plate and the supporting grout or concrete. This requires base-plate bending and/or resisting to mobilize and should not be relied upon unless the base plate is specifically designed for such resistance.

*Rick Drake, S.E.
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Another possible mechanism for shear resistance is the shear-friction method described in AISC's *Design Guide 7—Industrial Buildings: Roofs to Column Anchorage*, which can be downloaded for free by AISC members and ePubs subscribers at www.aisc.org/ePubs or purchased at www.aisc.org/bookstore.

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HSS and Single-Plate Shear Connections

The *HSS Connections Manual* indicates that a single-plate shear connection can be used if the following HSS wall slenderness ratios are satisfied:

1. Rectangular tubes: $b/t < 253/F_y^{0.5}$
2. Round tubes: $D/t < 3300/F_y$

Are these limitations alone enough to satisfy the requirements or must one also evaluate the wall for punching shear resistance? Do the HSS requirements also apply to ASTM A53 steel pipe? How about to large-diameter round and large-sized rectangular HSS?

As noted in the *HSS Connections Manual*, the slenderness limits assure that local distortion caused by the shear tab will be insignificant in reducing the column strength of the HSS. However, even when the slenderness limits are met, there are HSS wall limit states that should be checked. See Table 4-2 for the applicable limit states (shear at weld and punching shear.) Table 4-3 gives the rationale behind the limit states.

The *HSS Specification* (Section 1.2.1) specifically lists ASTM A53 steel pipe as approved for use under the *HSS Specification*. As such, the HSS equations are applicable to ASTM A53 steel pipe. The *HSS Specification* also applies to the limits of ASTM A500—a maximum periphery of 64". For example, a rectangular HSS 20x12 is right at the limit. Also keep in mind that as you get bigger, it does become more difficult to meet the slenderness requirements about which you asked.

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Bolt Tests

What testing should be performed on high-strength bolts upon delivery from the manufacturer?

Refer to AISC FAQ 6.3.2 at www.aisc.org/faq:

To ensure that delivered fasteners comply with the purchase requirements and furnished documents, upon receipt, the purchaser should verify that:

1. The bolts and nuts are marked as specified.

2. The manufacturer's mill test report shows a chemistry that meets the requirements for the type of bolts and nuts specified.
3. Certification numbers appear on the product containers and correspond to the certification numbers on the mill test reports for the fasteners.
4. Mill test reports are supplied to both the purchaser and the testing laboratory responsible for quality control.
5. If the fasteners are galvanized, rotational-capacity test results and nut lubrication should be verified to be in compliance with the appropriate ASTM specification requirements.

A bolt-tension calibration device (see 6.9.1 and 6.9.2) should be available in the shop and at the jobsite at the beginning of bolting start-up. From RCSC Educational Bulletin No. 1 (see www.boltcouncil.org), regardless of project installation requirements, whenever high-strength bolts are to be installed, not less than three bolt, nut, and washer assemblies from each lot supplied should be tested in a tension measuring device. Such testing must demonstrate that the bolts and nuts, when used together, can develop tension not less than that provided in the 2000 RCSC *Specification* Table 8.1. The bolt tension should be developed by tightening the nut. A representative of the manufacturer or supplier should be present, if required by the SER, to assure that the fasteners are properly used, and to demonstrate that the fastener assemblies supplied satisfy the specification requirements. Additionally, the inspector should be present.

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