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Bracing Connection Work Point

I am studying the effect of moving the Work Point (WP) on a vertical brace connection. I have noticed that when moving the WP from column web to column flange, the gusset plate dimension is reduced. Is it a good practice to specify the WP at the column flange? This will cause an additional moment (force) in the column due to the eccentricity. Do we have to consider this additional moment for design? Which is the most economical solution?

Moving the WP to the face of the column often results in a more economical solution. It may reduce the dimensions of the gusset, which will help prevent fouling with mechanical systems. It also reduces the forces at the gusset-to-column interfaces, which may result in smaller welds.

This arrangement, as you mentioned, will add a moment to the column, which will have to be considered in the design. Though the additional moment may increase the column size somewhat, the cost savings in the gusset design will often more than offset the additional column weight.

In his "24 Tips for Simplifying Braced Frame Connections" Victor Shneur (*MSC* May 2006) lists this as tip 20:

"When a brace and girder frame into the column flange, the work point doesn't always need to be at the column and/or girder center line. If the column or girder can accommodate additional bending moment due to eccentricity, the work point can be moved to the face of the column and to the top or bottom of the girder. This makes braced frame connections much simpler because connection eccentricity doesn't need to be considered for gusset and girder to column connections, and brace connections become more compact because of improved geometry."

Larry S. Muir, P.E.

Minimum Fillet Weld Size

Table J2.4 in the 2005 AISC *Specification* lists minimum sizes for fillet welds. Is this table limited to use with A992 steel only?

No. Table J2.4 is not related to any specific material grade, but rather to the quench effect of thick material on small welds. This effect is the same for all grades of steel covered in the AISC *Specification*. See the *Specification Commentary* Section J2.4 for discussion of the basis of this table.

Kurt Gustafson, S.E., P.E.

Shape Surface Area

Where can I find information to calculate the surface area of a 30-ft-long W24×68?

Appendix A of AISC *Design Guide 19* provides the surface area in ft^2/ft. Assuming that you have what the table calls "Case B," which is the entire surface area of both flanges and web, the surface area = (6.77 ft^2/ft)(30 ft) = 203 ft^2. The table also lists surface areas assuming that the shape is boxed-in or if the outside of one flange is not included. This publication is a free download for AISC members at www.aisc.org/epubs.

Brad Davis, Ph.D., S.E.

Minimum Connection Depth

Is there a minimum connection depth required for a beam framing to a supporting beam or column?

Chapter F in the AISC *Specification* states "The provisions in this chapter are based on the assumption that points of support for beams and girders are restrained against rotation about their longitudinal axis." That is, the *Specification* requires that rotational support must be provided at supports. It is common practice to provide a connection with depth at least equal to $T/2$, as recommended for all the framed connections in Chapter 10 of the AISC *Manual*, to ensure that the proper restraint is provided. It may be possible to provide the required restraint by other means, but the half-depth connection has become a de facto requirement.

Larry S. Muir, P.E.

Faying Surface Preparation

The faying surfaces of a slip-critical connection have been prepared to meet the slip coefficient around the bolt locations only. Paint overspray has occurred on areas of the faying surface away from the bolt holes. Is this permitted?

Figure C-3.1 in the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts* shows the applicable requirements. A distance of one bolt diameter (or at least 1 in.) around the perimeter of each bolt hole, plus all area within this outside perimeter, must meet the requirements for the qualified surface. The remainder of the faying surface outside this zone can be painted.

Larry S. Muir, P.E.

Beam Connection Location

When using a double-angle shear connection for a beam-to-column (or beam-to-beam) connection is there anything wrong with installing the angles in the lower portion of the beam? Is there a limit to the location of the angle placement relative to the beam depth?

There are no restrictions on where connections are placed along the depth of the beam in the AISC *Specification* or *Manual*. However, there are provisions in the *Specification* requiring torsional restraint at the supports. When you consider the importance of this in the erected condition, especially before the slab is placed, you can begin to see that this is the origin of the half-depth connection "requirement" in the *Manual*.

Often, due to such factors as cope depths, the connection will be placed lower and perhaps in the lower half of the beam. This usually is not a problem. However, if the copes are large, and/or the connection is less than half depth, the beam lateral bracing condition should be considered. In the completed condition with the slab in place, this is less of a concern.

A connection near the top of the beam will provide more stability during erection than one near the bottom, but both arrangements are allowed. It is common practice to place the connection as close to the top of the beam as is practical, keeping in mind some allowance for shop standardization.

Larry S. Muir, P.E.

steel interchange

Vibration Problem

I am experiencing vibration problems in a steel-framed floor. I see in *Design Guide 11* that the fundamental natural frequency is related to the deflection. If I camber the steel beam, the deflection would be less. In the calculations for frequency, what deflection should I use? Is camber going to help with vibration?

The deflection of a cambered member is no less than one that is not cambered. The only difference is that deflection in a cambered beam will tend to straighten the beam. So from a vibration standpoint cambering a beam does not typically ameliorate the problem.

The following equation may make this more obvious. The natural frequency of a simply supported beam is really

$$f_n = \frac{\pi}{2}\sqrt{\frac{EI}{mL^4}}$$

where m is the mass per unit length along the beam. This is Equation 3.1 in AISC *Design Guide 11* except substituting m for w/g. That the subsequent equations are written in terms of deflection, Δ, can be confusing, but dynamic properties are simply a function of stiffness and mass. The use of Δ is a measure of stiffness in this case.

Amanuel Gebremeskel, P.E.

Instantaneous Center of Rotation

Is there a formula for determining coefficient *C* for eccentrically loaded bolt groups? Where can I find it?

There is not a formula to determine the *C* value. However, the procedure is provided on page 7-6 of the 13th edition AISC *Steel Construction Manual*, under the heading "Instantaneous Center of Rotation Method." The procedure is somewhat complex—and iterative. You must first guess a location for the instantaneous center of rotation, then use statics to check your guess. If your guess was incorrect, then you take another guess, and repeat the process. Usually this is accomplished using a computer program, though it is possible to solve for *C* manually, albeit with several iterations.

Larry S. Muir, P.E.

Prequalified Connection

Is there a prequalified moment connection with a welded flange and bolted web available in ANSI/AISC 358-05? FEMA 350 includes a Welded Unreinforced Flange – Bolted Web (WUF-B) Connection. I do not see this connection included in ANSI/AISC 358-05 or Supplement No. 1.

The WUF-B connection is not listed in ANSI/AISC 358-05. This connection was included in FEMA 350, but only for Ordinary Moment Frame (OMF) applications. ANSI/AISC 358 is a standard of prequalified moment connection for Special and Intermediate Moment Frames (SMF and IMF). The AISC *Seismic Provisions* (341-05) does not require conformance demonstration for OMF systems, as is required for SMF and IMF systems.

Kurt Gustafson, S.E., P.E.

AISC *Seismic Design Manual* Table 4.2

Section 9.3a of the AISC *Seismic Provisions* references AISC *Specification* Section J10.6 for the panel-zone shear yielding strength, which has Ω = 1.5 for ASD. Table 4.2 in the AISC *Seismic Design Manual* provides a design aid for Special Moment Frames (SMF) that includes panel-zone shear parameters. However, the table only lists ϕ = 1.0, which is for LRFD. Are these tables only good for LRFD or can they be used for both LRFD and ASD?

Because only the value of ϕ is shown, the tabulated values are intended for use with LRFD. For a more detailed description of the table values please refer to page 4-77 in the AISC *Seismic Design Manual*. However, since ϕ = 1.0 in the specific case you questioned, it is easy to adjust these values on your own for ASD. To do so, you can either compare the table values to Ω times your ASD required shear strength, or divide the tabular values of R_v by Ω for comparison with your ASD required shear strength.

Heath Mitchell, P.E.

HSS Dimensional Tolerances

What is the tolerance for outside dimensions of an ASTM A500 HSS7½×3½×3/16?

ASTM A500 Section 11.1.2 and Table 3 covers "Permissible Variations in Dimension." This information is summarized in Table 1-27 of the 13th edition AISC *Steel Construction Manual*. For this case, the permissible over/under variation is given as:

Large Flat Dimension	0.01 x 7½ in. = 0.075 in.
Small Flat Dimension	1.5 x Large Flat = 0.113 in.

Heath Mitchell, P.E.

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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:

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