steel interchange

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SCBF X-Brace

Is a one-story X-braced frame permitted for a Special Concentrically Braced Frame?

X-braced frames are permitted for use in SCBF systems; however, each brace must be able to accommodate both the tensile and compressive modes. Tension-only bracing systems are not permitted in SCBF systems. See Section C13.1 in the AISC *Seismic Provisions Commentary* (page 177) for discussion.

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

Fillers

In Section J5 of the AISC *Specification*, one of the alternatives is that "The fillers shall be secured with enough bolts to uniformly distribute the total force in the connected element over the combined cross section of the connected element and the fillers." What does this mean? What are the design criteria to compute the number of bearing bolts in the filler plate outside of the primary connection?

The intent of this requirement is to try to eliminate uneven stress distribution due to bolt bending as the filler is developed in the joint. Obtaining the uniform distribution on the bolt is usually accomplished based on a consideration of the relative thicknesses of the elements involved. The number of bolts required can be calculated as

$$n = \frac{P\left(1 + \frac{t_f}{t_f + t_p}\right)}{\phi R_n}$$

Where P is the required strength (load), t_f is the thickness of the filler, t_p is the thickness of the plate, flange, or element being connected, and ϕr_n is the design strength of the bolt. The term in parentheses represents the increase in the number of bolts due to the presence of the fills.

Larry S. Muir, P.E.

Evaluation of Existing Structures

Is it permissible to use the ASD provisions of the 2005 AISC *Specification* to analyze an existing structure designed using the *Specification* of the 8th edition era? We're currently involved in the renovation of a structure designed and built in 1987/1988, and will need to slightly increase the load on several floor beams. Using the 8th edition steel manual several beams will be overstressed, from 8% to 13%. If the ASD provisions of the 13th edition are used, then these same beams are not overstressed.

Yes, you can use the current *Specification* to evaluate existing structures. You can use either the ASD or LRFD load approach to evaluate a structure originally designed using an older ASD *Specification*, as long as you use it consistently on both the load and resistance side of the design equation. Also, you can find provisions that may be helpful specifically when you are doing evaluation and/or repair in Appendix 5—*Evaluation of Existing Structures*—in the 2005 AISC *Specification*.

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

Bolt Hole Sizes

If a fabricator has detailed ⅞-in. diameter holes for ¾-in. diameter bolts can this still be considered a bearing-type connection? The loads are small—less than 10 kips per connection. It is for a pipe rack. Also, can you use slip-critical connections with galvanized steel?

Section J3.2 of the AISC *Specification* states, "Oversized holes are permitted in any or all plies of slip-critical connections, but they shall not be used in bearing-type connections." and "Short-slotted holes are permitted in any or all plies of slip-critical or bearing-type connections. The slots are permitted without regard to direction of loading in slip critical connections, but the length shall be normal to the direction of the load in bearing-type connections."

The first statement prohibits the use of oversized holes in bearing connections. The intention of both statements is to prohibit bearing-type load transfer in a direction where the hole clearance is greater than 1⁄16 in. From this the ⅞-in. holes would not be permitted in a bearing connection.

Galvanized material is allowed within the faying surface of slip-critical connections. Section J3.8 includes "hot-dipped galvanized and roughened surfaces" as a Class A surface. Section 3.2.2.(c) of the RCSC *Specification* (The Bolt Spec.) states, "Galvanized Faying Surfaces: Galvanized faying surfaces shall first be hot-dip galvanized in accordance with the requirements of ASTM A123 and subsequently roughened by means of hand wire brushing. Power wire brushing is not permitted."

This is an extra step required in the field or possibly at the galvanizers. For one connection or one beam this should not be a problem, but for an entire project this could represent a significant cost and schedule impact. There would probably also need to be additional people in the field to ensure compliance, at least for a while when the requirement was first introduced. I usually recommend avoiding slip-critical connections on galvanized material whenever possible.

Larry S. Muir, P.E.

Class A and Class B Coatings

How are Class A and B coatings qualified for slip-critical connections?

Coatings are qualified using the procedures contained in Appendix A of the 2004 RCSC *Specification*. This document is a free downloaded at www.boltcouncil.org. In my experience most coatings available on the market and qualified, are qualified for use as Class B. Since blast cleaning is required before coating, this is probably because the lower slip resistance of Class A would tend to make it less economical.

Larry S. Muir, P.E.

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Second-Order Analysis

I attended a seminar on second-order analysis, where I heard that the loads must be multiplied by the alpha value of 1.6 when using ASD for the member design. Do the analysis results get divided by the same value of 1.6 for member design, or are they calculated? I have been using the analysis results as calculated, and not dividing by 1.6.

Yes, if you're using ASD, multiply the loads by 1.6 going into the analysis and then divide the resulting member moments and other force effects by 1.6 for comparison with M_n/Ω, etc. This is stated in the last sentence of Section 7.3(a) in AISC *Specification* Appendix 7, if you're using the Direct Analysis Method. If you're using the Effective Length Method, this is Section C2.2a(2).

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

Panel Zone Shear Strength

1. Based on AISC 341-05 Section 9.3a, panel zone shear strength is calculated per *Specification* Section J10.6. In J10.6, there are two sets of equations; one assumes panel zone is elastic, the other considers the inelastic overstrength. My question is when to use the inelastic equation.

2. After comparing panel zone shear demand with the column web shear capacity, we may need to provide a doubler plate. To calculate the required thickness of the doubler plate based on the additional strength required, what is the length of doubler plate that can be used? Do you suggest counting the full column depth or using the actual length of the doubler plate, which is (Column depth – 2 × column flange thickness)?

My thoughts are as follows:

1. AISC 341-05 Section 9.3a refers to AISC 360-05 Section J10.6, which provides two options. In the first option, one can do the frame analysis with panel-zone deformations not modeled; in this case the basic form of panel zone shear strength (Equations J10-9 and J10-10) is used. Alternatively, when a more sophisticated analysis that considers the effect of panel-zone deformations is performed, a higher shear strength can be used (Equations J10-11 and J10-12). This higher strength is based upon the deformations (inelastic action) of the panel zone. So, the inelastic equations can be used when you include the deformations in the analysis.

2. The calculations in AISC *Design Guide 13* (and other examples in AISC literature) implicitly use the full column depth when selecting the web doubler plate thickness. That is, the required thickness is calculated based upon the full depth of the column, and then the additional thickness required is determined by subtracting the column web thickness. The edges of the doubler plate along the column flanges are welded to develop the shear strength of the doubler plate, so I think this is appropriate.

Charles J. Carter, S.E., P.E., Ph.D.

Conventional Configuration Single-Plate Shear Connections

Design limitations for conventional configuration single-plate shear connections imply that long-slotted holes are not permitted. Why are these not permitted? Also, for the extended configuration, are long-slotted holes permitted? The limitations for the extended configuration refer to AISC *Specification* Section J3.2 requirements, which imply that long-slotted holes may be used. If these are permitted, would they need to be slip critical?

Yes, the procedure presented in the Manual for the conventional configuration of single-plate shear connections is specific to the use of standard or short-slotted holes. This procedure is based upon testing to define a simplified approach that can be used for the majority of cases. No tests were run on connections with long slots, so I would not apply the procedure to connections with long slots.

Long-slotted holes can be used with the extended configuration procedure, however. When using long slots, I would design the connection as slip-critical. I do not think the eccentrically loaded bolt group *C*-values in the Manual can be obtained in bearing-type connections using long slots.

Larry S. Muir, P.E.

Nut Type for Anchor Rods

We have a project with anchor rods specified as ASTM F1554 Grade 105. ASTM A194 Grade 2H nuts were substituted for ASTM A563 Grade DH nuts. Are the A194 nuts considered equivalent in this application.

The ASTM F1554 Standard permits the use of either ASTM A194 or ASTM A563 nuts having a proof load equal to or higher than the minimum tensile strength specified for the anchor rod. Table 4 in ASTM F1554 lists the axial tensile strength of the anchor rod based on the diameter and grade. Table 3 in ASTM A194 provides the proof load for the diameter and type of nut. Comparison of these values shows that ASTM A194 Grade 2H nuts are acceptable with ASTM F1554 Grade 105 rods.

Kurt Gustafson, S.E., P.E

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