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.

Galvanized Slip-Critical Connections

Section 7.2 of the AISC *Seismic Provisions* indicates that bolted joints must have Class A faying surfaces. Section 3.2.2(c) of the RCSC *Specification for Structural Joints using ASTM A325 or A490 Bolts* indicates that galvanized faying surfaces are designated as Class C. Does this mean that we are unable to use steel members that are galvanized in the vicinity of the connections, for high-seismic applications?

The classes of faying surface finish requirements have been revised in the 2005 AISC *Specification*, now only including Class A and Class B requirements. The 2004 RCSC *Specification* was based on the three Class distinctions. The Commentary to Section J3.8 (page 349) of the 2005 AISC *Specification* discusses this revision. The previous Class A and Class C categories have now been consolidated into one Class A, which includes hot-dip galvanized and roughened surfaces.

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

Floor Plate

A note at the bottom of the Floor Plate Bending Capacity table on p. 2-145 in the 9th edition ASD *Manual* indicates that the loads are based on an extreme fiber stress of 16 ksi and simple-span bending. The 16 ksi allowable stress seems to be very conservative, assuming that the plates would likely have a yield strength of not less than that for A36 steel. What is the 16 ksi allowable based on?

You are right that the tables published in the *Manual* for simple-span flexure of floor plates may be conservative. However, these tables are merely design aides based on the conservative assumptions that are stated. Floor plate is commonly specified as ASTM A786, which is generally a commercial grade steel with no defined strength level, and this table allows for a very low strength level product. The responsible design professional always has the option of making their own analysis based on known parameters of the material they are working with, rather than use what they may deem to be conservative design aides. However, floor plate design is usually controlled by deflection anyway.

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

Rotational Restraint at Support

AISC *Specification* Section J10.7 requires full-depth stiffeners at the "unframed ends of beams and girders." What does this mean? Would an example be a girder bearing on a column with no beam framing into it at the column?

Yes, this section addresses situations such as the end of a beam that bears on column cap plate. Unless the column top is restrained, the beam might twist or the web might distort, allowing the bottom flange to move transversely. This creates a dangerous situation, because the column below was designed assuming a pinned-pinned condition with its top is restrained against lateral displacement. If a brace is provided to restrain the top of the column, the beam end is framed. If not, stiffeners can be used as required in Section J10.7. Note that the concern for column stability also exists when girders frame continuously over the top of the column. See Section 2 of the 13th edition AISC *Steel Construction Manual* for further information.

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

Bolting for High-Seismic Applications

Are slip-critical connections required for seismic connections? And if so, for what seismic design category are they necessary?

In high-seismic applications, slip resistance is required, but the connections are designed for bearing values. According to Section 7.2 of the 2005 AISC *Seismic Provisions* (AISC 341) (a free download at **www.aisc.org/2005seismic**), "All bolts shall be pretensioned high-strength bolts and shall meet the requirements for slip-critical faying surfaces … with a Class A surface." Also, "The available shear strength of bolted joints using standard holes shall be calculated as that for bearing-type joints…" This is applicable for high-seismic applications where the requirements in AISC 341 must be met.

Amanuel Gebremeskel, *P.E*

Nut Engagement

We have a situation where bolts have been installed too short (the bolt tip is below the top of nut) in a steel-to-steel joint. Is there a way to assess the reduced capacity based on the percentage of thread engaged?

The 2004 RCSC *Specification* (a free download at **www.boltcouncil.org**) requires that "the bolt length used shall be such that the end of the bolt extends beyond or is at least flush with the outer face of the nut when properly installed." Thus when the bolts are "short," the installation is non-compliant. There is no reduced value permitted by the specification. The bolts should be replaced with bolts of the correct length.

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

Hole Sizes for Galvanized Bolts

An engineer designed the structural steel connections using standard holes in all plys for ASTM A325-N bolts such that the connections need not consider slip-critical limit states. The steel is to be hot-dip galvanized. The galvanizer is requesting that the standard holes be increased by an additional tolerance of 1⁄16 in. to account for the coating thickness. I'm hesitant to grant approval for a hole size that would require slip-critical limit states to dictate connection design. If the hole size is increased, would the connection design need to be reevaluated for slip-critical conditions?

Increasing the hole size to account for the galvanizing in a bearing condition is not an accepted practice and is not allowed by the AISC *Specification* or the RCSC *Specification*. If the holes are oversized the connection must be designed as slip-critical.

Larry S. Muir, *P.E.*

steel interchange

Stiffeners for an EBF Link

Commentary Section C15.3 in the AISC *Seismic Provisions* indicates that for EBF links that are less than 25 in. deep, the stiffener need be on one side only. What is the interpretation of "need be?" Does it mean "must be" or "may be?" Many practicing engineers are interpreting this as "may be." When EBFs were tested, what was the protocol? Have they tested intermediate stiffeners on one side only? Is there any detrimental effect on the inenelastic rotation of link beam due to increased rigidity of link beam when stiffeners are used on both sides?

Use of either one-sided or two-sided intermediate stiffeners is allowed for beams up to the indicated depth. Both one-sided and two-sided specimens have been tested, with similar loading protocols to what is used for moment connections (increasing the displacement incrementally until failure). The drift limits in the AISC *Seismic Provisions* were based on these tests. No difference in performance was noted between one-sided and two-sided specimens. The key item is that the stiffener must be stiff enough to force the link web to buckle in the panels between stiffeners, rather than over length of the link. The thickness requirements for stiffeners in the AISC *Seismic Provisions* are adequate to make this happen for the single-sided cases.

James O. Malley, *Senior Principal*
Degenkolb Engineers

Tensile Strength of Anchor Rods

Where does one find values for f_{uta} (specified tensile strength of anchor steel) as used in Appendix D of ACI 318? Also, is the f_{ya} (specified yield strength of anchor steel) equal to F_{nt} given in Table J3.2 of the 2005 AISC *Specification*?

Generally, the required minimum tensile stress for the material type can be found in the applicable ASTM Standard. The value of F_u for many types of ASTM materials used for anchor rods covered by the AISC *Specification* also are listed in Table 2-5 of the 13th edition AISC *Steel Construction Manual*.

The answer is "no" to the second question. The values of F_{nt} in Table J3.2 in the 2005 AISC *Specification* provide the nominal tensile stress for use with ϕ or Ω in design, and are adjusted to account for the difference between nominal bolt body area and threaded area. This is not the yield strength. Some anchor rod materials have a defined yield point, while others do not. The tensile stress as used in the ACI 318 Appendix D approach is based on the tensile stress area at the thread. Therefore, one needs to be careful when comparing the two approaches to anchor rod evaluation.

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

Punching Shear

Why is there a requirement to check punching shear on the wall of an HSS column with a single-plate shear connection, but no similar check when connecting to the web of a W-Shape?

Punching shear can occur at a W-shape column or girder web. However, it is not usually a consideration, because W-shape column and girder webs are usually thick enough that punching shear won't control. Using Equation K1-10 of the 2005 AISC *Specification*, for a ⅜-in. ASTM A36 shear tab and an ASTM A992 web, the web would have to be less than 0.208 in. thick for punching shear to control. It would be very unusual for a W-shape column to have a web thickness less than this.

There are three W-shape beam sections that have a web thickness less than 0.208 in. These are rarely, if ever, used as girders. Even if one of these sections (W8×10, W10×12, or W12×14) is used as a girder, it will not have adequate torsional stiffness to allow punching shear to be a realistic limit state. If a shear tab is on both sides of the web, then the supporting member won't rotate much, but punching shear also may not occur in that case, because shear tab on the opposite side will be trying to rotate in the opposite direction. (Note that a single-sided shear tab is always the case for an HSS column.)

So, punching shear is possible for a web supporting a shear tab, but it is only realistic for HSS columns, because those columns provide significant rotational restraint and some of those sections have extremely thin walls.

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

Shear Lag Factor

Table D3.1 of the 2005 AISC *Specification* gives shear lag factors (U) for various cases of tension connections. I have a situation like Case 4, where two plates are transmitting tension through longitudinal welds only. The U-factors are based upon the length of the weld (l) and the width of the plate (w). No U-factors are tabulated for the condition where $l < w$. The plate I have is 4 in. wide and the weld can only be 2 in. long. What U-factor is appropriate for this situation?

Such a weld connection configuration does not meet the AISC *Specification* requirement as stated in Section J2.2b: "If longitudinal fillet welds are used alone in end connections of flat-bar tension members, the length of each fillet weld shall not be less than the perpendicular distance between them." Thus there is no U-factor listed as appropriate for this detail because it represents a condition in which the shear lag effect is likely to cause rupture to occur in a manner that is not well predicted by the methods we use in design.

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

The complete collection of Steel Interchange questions and answers is available online. Find questions and answers related to just about any topic by using our full-text search capability. Visit Steel Interchange online at **www.modernsteel.com**.

Kurt Gustafson is the director of technical assistance and Amanuel Gebremeskel is a senior engineer in AISC's Steel Solutions Center. Larry Muir and Brad Davis are part-time consultants to AISC. James O. Malley is Senior Principal at Degenkolb Engineers.

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, 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 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 Solutions Center
One East Wacker Dr., Suite 700
Chicago, IL 60601
tel: 866.ASK.AISC • fax: 312.803.4709
solutions@aisc.org