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!

Reduced Coefficient of Slip Resistance

I have a building that was designed as a special concentrically braced frame (R = 6). It is a one-story building, and most of the vertical braces were detailed to be shop or field welded. However, there are some horizontal connections that are to be field bolted. The AISC *Seismic Provisions for Structural Steel Buildings* require that the surfaces be prepared for slip resistance (Section 7.2), which we did. However, the contractor has accidentally painted over these holes, and the building is now erected. Would it be possible to use primer paint in place of a reduced coefficient of friction (lower than 0.33) to justify that the bolts have a lower than class A coating, but still satisfy the need to be slip-critical for the building?

Question sent to AISC's Steel Solutions Center

The slip-coefficient requirement is included in the seismic provisions to minimize damage in more moderate seismic events. There are no provisions given where reduced slip coefficients are justifiable where the faying surfaces are primarily subjected to shear. If the faying surface is primarily subjected to tension, such as in bolted end plate moment connections, the requirement on preparation of the faying surfaces may be relaxed. See the commentary to Section 7.2 of the 2005 seismic provisions.

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Cruciform Columns and RBS

Is there any concern relative to using a cruciform column (symmetric column in both directions with one column web cut and welded to the other column web) in an SMF? If so, are there any design criteria or is there research regarding the use of this type of column? The column and beam combination in either direction are pre-qualified per FEMA 350.

Question sent to AISC's Steel Solutions Center

This question appeared a few months ago in Steel Interchange and is being revisited now that a new standard has become available. With the recent release of ANSI/AISC 358-05 *Prequalifed Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (a free download from **www.aisc.org/aisc358**), cruciform columns are now prequalified per Section 2.3.2b(4). Reduced beam section (RBS) moment connections are now prequalified per Section 5.3.2 on either axis of cruciform columns. Hence there is no longer the need to qualify these items, as both are prequalified under ANSI/AISC 358-05.

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Rod vs. Nut Strength

I understand the capacity of a threaded rod in tension is equal to the gross (nominal) area times $0.33F_u$. Does the 0.33 factor include the effects of the potential for stripping of the threads of the rod through the nut placed at the rod end?

Question sent to AISC's Steel Solutions Center

I believe you are referring to an older AISC specification. The current 2005 AISC specification (**www.aisc.org/2005spec**) provides for an allowable tensile capacity of $0.375F_u$ for a threaded rod rather than $0.33F_u$.

The 0.33 or 0.375 coefficients do not address the material strength of the rod nor the type of nut. The nut grade is always selected from an acceptable list in ASTM A563 based upon the rod (or bolt) material used such that the nut is stronger than the rod. That is, a fully engaged ASTM A563 heavy hex nut of the appropriate grade will develop the tensile capacity of the bolt or threaded rod with which it is used. The design therefore is based on the allowable tensile capacity of the rod, and thread stripping will not occur with a properly selected nut.

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Combined Loads on Anchor Rods

How do you design anchor rods for combined loads of tension, shear, and bending? What equation do you use to combine the forces?

Question sent to AISC's Steel Solutions Center

AISC has historically recommended that taking shear in anchor rods should be avoided. If friction between the base plate and foundation is insufficient to resist the shear force, AISC recommends that shear lugs, embedments, or other restraining elements be used to resist these shear forces. There are several problems involving the engagement of rods bearing against the enlarged base plate hole: the movements that must occur for the rod to bear, the number of rods that may go into bearing simultaneously, the vertical locations of the bearing against the plate, and the point of resistance in the concrete causing the eccentricity of shear resulting in the bending force. There are no standard assumptions for these variances. These unknowns must be combined with the relatively small capacity of anchor rods in bending as compared to tensile and shear capacities. If all of the tensile, shear, and bending forces are rather small, the engineer will have to use engineering judgment as to the method of combining the forces.

The AISC specification does not have an equation for combining the effects of tension, shear, and bending on anchor rods. Anchor rods are grouped with bolts in Table J3.2 for determining the nominal tensile and shear stress for bolts and threaded parts. Combined tension and shear of bolts in either bearing or slip-critical connections is covered in Sections J3.7 and J3.8 respectively. These sections do not specifically address anchor rods as there is no definition of an anchor rod connection as being either snug-tightened or pre-tensioned. However, many engineers will use the bearing-type connection equations of Section J3.7 when checking the effects of combined tension and shear. ACI 318 Appendix D has a similar but more conservative straight-line equation approach for checking this interaction of tension and shear. The bending component in a bolted steel-to-steel connection is not addressed in the AISC specification, as this is neglected due to the tendency of clamping pretension to resist the bending.

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Strength of Truss Connections

Section J1.5 of the 9th edition ASD *Manual of Steel Construction* states: "The connections at ends of . . . members in trusses shall develop the force . . . not less than 50% of the effective strength of the member . . ." Chapter 13 of the 13th edition *Steel Construction Manual* seems to indicate this is no longer required (see the paragraph titled "Minimum Connection Strength"). Is this correct?

Question sent to AISC's Steel Solutions Center

The answer is yes. While the *Steel Construction Manual* is simply a tool to aid the engineer in applying the specification, the specification no longer stipulates the 50% requirement.

There was not much of a basis for this requirement in the old AISC specifications, other than judgment. It was included in these specifications as an attempt to eliminate connections that would make handling, shipping, and/or erection difficult. This requirement was removed in more recent AISC specifications because it really did not ensure anything. The issue was then still addressed in the manual text, which highlighted the issues for handling, shipping, and erection, and recommended (but did not require) a 50% minimum, unless one knew better.

The old provision provided an out. For example, if you knew how the truss would be picked, you could provide less than 50% if it was adequate for the criteria you knew. So yes, you can provide less than 50% when you know what the effects of handling, shipping, and erection are for the truss.

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Shear Connection Capacities

I just received my copy of the 13th Edition *Steel Construction Manual*. Am I misreading these tables or have all of the connection capacities for ASD connections really been beefed up this much? For example, a shear tab connections with a ⅜ plate with two ¾ bolts now can carry 21.2 kips.

Question sent to AISC's Steel Solutions Center

Yes, connection capacities per the 2005 AISC *Specification for Structural Steel Buildings* and the 13th Edition *Steel Construction Manual* are higher in many cases than for the 1989 ASD specification and 9th edition manual. It depends, though, on which limit state controls, as some went up and others went down. Also, it depends upon what changes were made in design procedures given in the *Manual*.

For the specific example you cited—a single-plate connection—the loads for the plate and bolts are higher than they used to be. We streamlined and simplified the design procedure for the conventional configuration based upon some testing that was done to assess strength, eccentricity, and rotation questions. The testing showed that eccentricity requirements for the conventional configuration could be significantly reduced (eliminated in most cases). This accounts for most, if not all, of the strength increase you see in Table 10-9.

The details of how the current procedure works are provided on page 10-101. If you compare to the previous versions of the *Manual*, I think you will see significant simplification. Also, an extended procedure is provided in case you have something that doesn't meet one or more of the limitations of the conventional procedure.

Note also, however, that the beam web limit states may still limit you to a lower strength. A thin web may have a lower block shear or bolt bearing strength, for example. You can check this easily using the bottom part of Table 10-1.

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Steel Interchange is a forum for *Modern Steel Construction* readers 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.

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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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