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.

Member Stiffness Reductions

The direct analysis method per the 2005 AISC *Specification* (Appendix 7 and its Commentary) requires the consideration of reduced axial/flexural stiffness and notional loads during analysis and design of any structure. How appropriate would it be to use this reduced stiffness and notional loads for serviceability conditions such as deflections/drift calculation? Is it necessary to run a separate analysis without any geometrical property reductions for applicable members to calculate the deflections?

Appendix 7 states that the stiffness reductions should not be used for serviceability checks. Nominal values should also be used when calculating the available strength of members. Reduced stiffness values are only used to determine the required strengths for which the structure must be designed.

Amanuel Gebremeskel, P.E.

ASTM A490 Bolts Subject to Tension

Section 4.2 of the RCSC *Specification* states that "pretensioned joints are required…for joints with ASTM A490 bolts that are subject to tension or combined shear and tension, with or without fatigue." This is reiterated in footnote "c" of RCSC *Specification* Table 4.1. Can you please explain the reason for this provision in a connection without fatigue? Why are ASTM A325 bolts not required to be pretensioned under tension or shear and tension, unless subjected to fatigue loading?

High-strength bolts are sensitive to variations in tension loads. Fatigue is usually considered in structural steel design above 20,000 cycles, but bolts can suffer degradation at fewer cycles. Pretension has been shown to reduce the magnitude of the change in applied tension loads to about 10% of the full applied load, effectively eliminating the degradation possible due to few cycles of high loads.

It used to be a requirement for all high-strength bolts. Testing was performed to show that ASTM A325 bolts have sufficient ductility to allow the elimination of the requirement to pretension them when used in tension applications as a default.

Tom Schlafly

Delayed Construction

We have a project that was shut down during construction due to the economy. It is ready to restart now. When the project was shut down, the structural steel was erected but no decking installed. Now the steel is rusted. Are there any industry standards related to dealing with rust on steel?

There is no industry standard with regard to the amount of surface rust that is permissible. This really comes down to engineering judgment, but here are a few things to investigate that may help in your decision. Look to see if the rust is tight and can withstand a vigorous wire brushing. Also make sure there is no appreciable loss of cross section. Determine if the steel will be enclosed such that future deterioration is prevented. If spray fireproofing is to be applied, one should consult the applier as to their requirements for adherence of the fireproofing.

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

S_x or Z_x?

I have recently been using the ASD option in the 2005 AISC *Specification* to calculate combined stress using square HSS and am confused with the use of Z_x for flexural design, versus the use of S_x in the older ASD specification. Is the use of Z_x limited to LRFD load combinations?

Use of the plastic section modulus, Z_x, is not restricted to when the LRFD load combinations are used in the design process. This is equally applicable to when ASD load combinations are used. The only restriction is that the use of the plastic section modulus is only permitted for compact shapes.

The older ASD specifications, back to the 1963 version, included a plastic distribution effect in disguise, with the 10% increase ($0.66F_y$ versus $0.6F_y$) in allowable stress permitted for compact shapes. This additional 10% was reflected in the use of a lower bound for the shape factor (Z_x/S_x) of 1.1 for W-shapes. In effect, you've "always" been using something closer to Z, even when the bending equation had S in the ASD formula. The difference is that the 2005 *Specification* permits the use of the actual Z_x for the section.

Kurt Gustafson, S.E., P.E

HSS Properties

Why are the thicknesses of HSS different in the 9th edition AISC ASD *Manual* compared to the 13th edition AISC *Manual*?

The differences in the listed HSS wall thickness stem from the common practices of HSS manufacturers. HSS producers in the U.S. most commonly use the ERW process for ASTM A500 products, and have the capability to produce HSS from plate products that have an actual wall thickness close to the lower bound permitted in ASTM A500 (10% less than the nominal value). To account for this consistent with tolerances for other structural shapes, Section B3.12 of the 2005 AISC *Specification* defines the design wall thickness for such shapes as 0.93 times the nominal wall thickness. This was not done 20 years ago when the tabulations in the 9th edition ASD *Manual* were published because the production practice was not known until after that time.

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

Bolt Entering Direction

Using a single-plate shear connection with short-slotted holes, and with one washer; does it matter which direction the bolt passes through the hole, as long as the washer is on the slotted side?

The entering direction of the bolt does not affect the performance of the connection. The only exception would be if the bolts are assumed to have the threads excluded from the shear plane. In this case it is possible for the entering direction to cause the threads to be included in the shear plane, resulting in a lower shear value.

Larry S. Muir, P.E.

steel interchange

Stiffened Extended Single-Plate Shear Connection

Based on my review of research papers (Muir and Hewitt 2009, Ghorbanpoor and Sherman 2003) and extended configuration design methodology provided in the 13th edition AISC *Manual*, it appears that test results indicate behavior is much different when an extended single pate is stiffened compared to a similar unstiffened connection. The *Manual* does not provide a stiffened extended single-plate connection design methodology. Is there such a design procedure?

The procedure provided in the 13th edition AISC *Manual* can be adapted for use with stiffened extended single-plate connections as well as unstiffened. The only changes I would make are:

(1) I would relax the $\frac{5}{8}t_p$ weld ductility requirement and instead check the C-shaped weld formed by the weld to the column web and the stiffeners to develop the flexural strength of the plate. I suspect in most cases a minimum sized fillet will suffice.

(2) I would not check the plate for buckling.

In my opinion, the seemingly large difference in behavior between the unstiffened and stiffened configurations reported in the Sherman and Ghorbanpoor tests is due to the fact that they were able to mobilize more of the reserve strength of the column in the stiffened case. Because the column cannot safely be assumed to have significant reserve capacity in all cases, the *Manual* procedure does not rely on any resistance from the column. The *Manual* does however specifically allow other rational design methods. One such method would be to take some of the eccentricity in the column when providing stiffeners. However, this additional eccentricity would have to be accounted for in the main member design.

Larry S. Muir, P.E.

Column Tier Height

Are there any specification requirements or OSHA regulations that limit the maximum length of columns that one can use as a single field section for steel erection? What is normal practice for location of column splices in multi-story construction?

One controlling factor in selecting column length between splices often stems from the OSHA requirement that a fully decked or planked floor shall be maintained within two stories or 30 ft, whichever is less, directly under any erection work being performed. The alternative provided by OSHA—installation of netting to restrict fall distance to 30 ft—means that tiers commonly are limited to 30 ft so that nets do not have to be installed as a common practice. This leads to common tiers being in two-story heights.

Regular readers of *Steel Interchange* may remember a previous answer that two-story and four-story tiers are common, but three-story tiers are not preferred. The three-story tier has built-in inefficiency because two floors can be erected at a time with the OSHA 30 ft limit, and having three means alternating between erecting two, then one. The two- and four-story tiers allow erection of two floors at a time at all levels.

OSHA also requires that splices for exterior columns must be at least 4 ft above the floor to facilitate attachment of safety cables. The 4 ft dimension is also a good working height for the ironworker to make the splice for the column above, and thus has become preferred location for column splices in general. A technical benefit of the 4 ft dimension is that it places the splice near the mid-story inflection point for lateral frame action.

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

Tension Control Bolts

What is the difference between an F1852 TC Bolt and an A325 TC Bolt? Which of these is superior in strength or more economical?

The answers are *none* and *not applicable*. These designations are different ways of referring to the same bolt. An ASTM F1852 bolt is a "Twist Off" Type Tension Control Structural Bolt/Nut/Washer Assembly, which is an ASTM A325 equivalent product. Before ASTM F1852 existed, TC bolts with an A325 strength level were called A325 TC bolts. Now that ASTM F1852 exists, it is just a matter of time before everyone is aware that these are essentially the same product. Note also that ASTM F2280 now exists and is essentially the same as an A490 TC bolt.

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

Vertical Bracing Connection

When using the general case of the uniform force method with a moment produced from $V_b(\alpha-\bar{\alpha})$, where does this moment get resolved? Does the column take it or does the beam take it?

It is common to resist this moment at the gusset-to-beam interface. There are two primary reasons for this.

First, it is common to use a bolted connection at the gusset-to-column interface and a welded connection at the gusset-to-beam interface. With this arrangement the gusset-to-beam interface usually will be stiffer and therefore will tend to attract more of the moment.

Second, the welded beam-to-gusset connection will usually be easier to design to resist the moment than the bolted gusset-to-column connection, which may have to consider prying and other effects.

It should be noted that neither the beam nor the column will see any moments beyond the limits of the connections. Even with a ΔV_b or α not equal to $\bar{\alpha}$, the moments are internal to the connections and the beam and column remain only axially loaded.

Larry S. Muir, P.E.

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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.

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