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.

Backstepping

I just saw the word "backstep" in reference to a welding note on a structural drawing. What exactly does this mean?

Backstepping is a welding technique where the direction of welding is reversed over segments of the weld length to help avoid rotational distortion. For example, butt-welding two pieces of plate along a seam may induce rotational distortion as a result of transverse shrinkage, causing the joint to either open up during welding or close tight, and may possibly cause overlapping in thin plates.

While the backstepping technique can be successful in controlling rotational distortion, it provides little or no assistance in minimizing other types of distortion. The subject of backstepping is discussed in AISC *Design Guide 21: Welding Connections – A Primer for Engineers*. Design guides are available as free downloads for AISC members at **www.aisc.org/epubs** or can be ordered from the AISC Bookstore at **www.aisc.org/bookstore**.

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

Slip-Critical Bolt Values

Has Table 3, "Allowable Load for Slip-critical Connections," which appeared in the 9th edition AISC manual, not been included in the 13th edition?

Slip-critical bolt values are covered in Part 7 of the 13th edition AISC manual. There you will find two tables, one for slip prevented at service loads, and another for slip prevented beyond the service-load range up to the strength-level loading. All previous AISC specifications were based upon slip in the service-load range as a serviceability criterion. The latter provision has been added for specific (and rare) cases in which the factor of safety against slip must be higher. These two options, including specific cases for which the latter criterion might apply, are discussed in the AISC specification in Chapter J and also in the Commentary, available as free downloads at **www.aisc.org/2005spec**.

Sergio Zoruba, Ph.D., P.E.

Holes in Base Plates

On one of my projects, it was reported by the inspector that holes in some of the column base plates were enlarged in the field to accommodate anchor rods that were misplaced. Some of the holes were enlarged significantly, and some of the plate edges were notched out around bolts. The columns are part of a moment frame, so the bases were designed for the lateral forces. Is it possible to repair the plates by welding an angle or plate on top with drilled holes to receive the anchor rods? If a new plate is added on the top, the anchor rods may not have adequate projection. The rods are A307 material. Is this weldable, or would a coupler be required?

Many things are possible, including welding plates with standard holes over the enlarged holes in the base plate to the existing plate. However, the requirements for the base anchorage will largely depend on the type and magnitude of force to be resisted by the anchor rods. Major considerations are the method assumed to transfer the shear forces from the column to the foundation, and the magnitude of any tensile force in the rod. For guidance on this evaluation, refer to AISC's *Design Guide 1: Base Plate and Anchor Rod Design*, second edition (**www.aisc.org/epubs**). The recommendations therein can be applied to your modified column bases with proper engineering judgment applied based upon proper strength and stiffness of the actual modifications you need to make.

And yes, ASTM A307 is a mild carbon steel and is generally considered weldable. There are also coupler solutions that may be appropriate in some cases. Both of these options are discussed in the design guide.

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

I_y and J for Double-Angles

In attempting to calculate the lateral-torsion buckling of double-angles per the AISC manual, I cannot find properties such as I_y and J. Are these tabulated in the 13th edition manual?

These are not specifically tabulated for double angles in the 13th edition manual. However, they can be determined from the single-angle properties. J for a double angle is always twice the value for a single angle. J values for single angles are listed in Part 1 of the manual. When looking at the Y-Y geometric axis, the moment of inertia can be determined from the radius of gyration, r_y, and area, A, which are tabulated in Table 1-15. That is, $I_y = Ar_y^2$. You didn't ask about I_x, but in this case, the neutral axis goes through the center of gravity of both angles, and the moment of inertia about that axis is twice the value for a single angle.

Sergio Zoruba, Ph.D., P.E.

Slip-Critical Bolts used in Shear Tabs

I had the impression that we could not use slip-critical bolts with extended shear plates because the flexibility of shear plate connection is achieved by the plowing of the bolts against the main material. Based on the 13th edition AISC manual, it seems we can use slip-critical bolts with the extended configuration. What makes the connection flexible in this case?

It is rare that a single-plate connection would require the use of slip-critical bolts. But, if a slip-critical joint were used, the rotational demand would first cause slip and then proceed just as it would for a single-plate connection that had been designed as a bearing-type connection. This is why all high-strength bolted connections, including slip-critical and bearing-type connections, must be designed as if the bolt will eventually go into bearing—even if that does not occur at a service-load level. When the bolts do go into bearing, it is some combination of the plowing of the bolt and the yielding of the plate (and or beam web) that provides for the rotational flexibility of the connection. In the extended configuration of single-shear plates, the plate thickness is limited such that the plate flexural strength does not exceed the "flexural" strength of the bolt group in shear.

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

Filler Metal for High-Seismic Applications

Where can I find the filler metal requirements for high-seismic design?

Refer to the 2005 AISC *Seismic Provisions* (www.aisc.org/2005seismic). Section 7 addresses weld requirements, as does Appendix W. Note that Appendix W provisions are also consistent with provisions that subsequently have been released in AWS D1.8, a seismic addition to AWS D1.1.

Sergio Zoruba, Ph.D., P.E.

Knifed Connection

What is the definition of *knifed connection*? I have looked in several textbooks without any success.

The term *knifed connection* is usually used in reference to a double-angle connection that is shop attached to the supporting member (usually a column) and where the bottom flange of the beam is coped away or blocked out on both sides of the web. A small increase (1/16 in. or so) in spacing between the angles is provided to ease erection. The flangeless section of the beam web is then "knifed" down between the outstanding legs of the support angles during erection, and the bolts are inserted through the mating angles and beam web. The bolts easily pull the plies together, eliminating the small gap that was allowed in fabrication to ease erection.

This detail may be less common today, as one-sided connections such as single-plate connections are used with increasing frequency. Nonetheless, a knifed connection is sometimes considered when the attachments to the column are shop welded and other connection types are not feasible.

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

SCBF Brace Reinforcement

The 2005 AISC *Seismic Provisions* for SCBF states:

Where the effective net area of bracing members is less than the gross area, the required tensile strength of the brace based upon the limit state of fracture in the net section shall be greater than....* $R_yF_yA_g$ *(LRFD).

For an HSS brace, F_y = 46 ksi; F_u = 58 ksi; R_y = 1.4; R_t = 1.3

The required tensile strength of the brace based upon the limit state of fracture is $0.75R_tF_uA_e = 56.55A_e$ and the required tensile strength is $R_yF_yA_g = 64.4A_g$. Setting the required tensile strength of the brace greater than $R_yF_yA_g$ results in $A_e = 1.14A_g$, which is not possible. It seems that you cannot stiffen the member to increase A_e without also increasing A_g. How can the requirement of 13.2b(a) can be met?

One can increase A_e by adding reinforcement (plates, for example) at the ends of the HSS brace member. Since the reinforcement does not run along the entire length of the HSS brace (just at the ends), A_g for the brace is not increased. This means using the A_g from the non-reinforced length of the brace rather than from the reinforced ends.

Sergio Zoruba, Ph.D., P.E.

Heat-Straightening Columns

A two-story column on our project was erected approximately 1 7/8 in. out-of-plumb. The erector would like to heat one face of the HSS column and then cool it quickly in order to bend the column back to plumb. All other issues aside, what will this heating and cooling process do to the material properties of the column? It seems as though there will be some residual stresses in the column as well that may present a problem.

Controlled application of heat can be used to effectively to straighten or curve members. The mechanical properties of the structural steel material are generally unaffected by such heating applications as long as the procedures described in Section M2.1 of the AISC specification are followed. The AISC specification is available as a free download at www.aisc.org/2005spec.

The use of heat bending is as much of an art as it is a science, and significant experience is generally required to control the process and to achieve the desired results. Also, depending upon how much of the rest of the structure is present to restrain the column, there may be little actual movement induced by heating.

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