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**.

steel interchange

Cold Weather Welding

Is there a threshold temperature beyond which I should indicate field welding of deck to structural steel, or structural welding should not be attempted?

I once asked a related question of Omer Blodgett at Lincoln Electric. He told me it is a matter of heat input, and there are procedures in AWS D1.1 that address this. His advice to me was that when the weather gets cold, it is the tack welding that requires the most scrutiny because the heating and quenching rates are so much faster in this case.

The technical answer is also available. AWS D1.1 Table 3.2 has minimum temperatures for welding. The table often lists none, but there is a footnote that sets a floor at 32 °F. If conditions are below that, it is required to raise the steel to 70 °F via preheating.

Deck welding is covered in AWS D1.3, but it does not require heating. The requirements in that document were based on a study showing that low temperatures do not cause a problem with arc spot welds.

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

Washer Plates for Column Base Plates

The projection of some of the anchor bolts on our project is shorter than expected. We have ½-in. plate washers to cover the holes as shown in Table 14-2 of the AISC *Steel Construction Manual*. The table does not refer to steel grade for the plate washers. We are wondering if we can use thinner plate washers if we use A572-50 steel plate instead of A36 steel.

Table 14-2 does not indicate the steel grade, but the discussion on "Washer Requirements" on page 14-10 indicates that washers are most typically furnished from A36 material, which is the basis for the table.

The length and width dimensions in Table 14-2 are the minimum recommended dimensions to cover the correlating base plate hole sizes. The minimum plate thickness indicated is selected based on two considerations: (a) the strength required to prevent the anchor rod from pulling through when the base plate is subject to uplift and (b) the stiffness required to prevent "large" displacement when the base plate is subject to uplift. In general, washer thicknesses roughly one-third of the anchor rod diameter meet the stiffness requirement.

Increasing the strength of the washer material to 50 ksi would give you more strength, but making it thinner would reduce your stiffness. That said, if you evaluate the washer strength and stiffness based on your actual anticipated loads, you may be able to change material and reduce the thickness.

Susan Burmeister, P.E.

Plate Girder Stiffeners

Where did Equation G4-3 of the 1989 AISC ASD *Specification*, used to compute weld requirements for stiffeners, go?

Equation G4-3 in the 1989 AISC ASD *Specification* calculated the required shear strength of the welds attaching the transverse stiffeners to the web of a plate girder and read:

$$f_{vs} = h\sqrt{\left(\frac{F_y}{340}\right)^3}$$

The 1989 AISC ASD *Specification* was the last time the equation made an appearance. The Commentary to the 1989 AISC ASD *Specification* states: "The amount of shear to be transferred between web and stiffeners is not affected by the eccentricity of loading and generally is so small that it can be taken care of by the minimum sized fillet weld. The specified Equation (G4-3) affords a conservative estimate of required shear transfer under any condition of stress permitted by Equation (G3-1)." The 1st Edition AISC LRFD *Specification* contained no provisions governing the connection of the stiffeners, perhaps based upon the Commentary statement that minimum-sized fillet welds would suffice.

Lest anyone ever try to connect the stiffeners to the web with chewing gum, prescriptive requirements are included at the end of Section G2.2 of the 2010 AISC *Specification*. These requirements replace the previous conservative calculations.

Larry S. Muir, P.E.

Domestic or Foreign Steel?

Are U.S.-dimensioned W-shapes produced in the U.S. or by foreign sources?

The vast majority of W-shapes used in the U.S. construction market are produced in the U.S. These shapes meet the dimensions in ASTM A6/A6M and usually meet the strength and other requirements of ASTM A992. In recent quarters, domestic mills such as Nucor-Yamato, Gerdau and Steel Dynamics have exported up to 14% of their production. The notable exception is Arcelor-Mittal, which provides ASTM A913 W-shapes to the U.S. market from its Luxembourg facility.

Martin Anderson

HSS Wall Thickness

ASTM sets minimum wall thickness at 90% of nominal, while AISC bases section properties on a design wall thickness of 93% of nominal. Therefore, a member can meet the ASTM specification even if its actual wall thickness is less than the design wall thickness presented in AISC's spec. Shouldn't the tolerance and the reduction be the same?

The design wall thickness (0.93 times the nominal wall thickness) is an estimate of the actual wall thickness provided by manufacturers in the U.S. for electric-resistance-welded HSS. When the design wall thickness is used with the safety factors and resistance factors in the AISC *Specification* (a free download at **www.aisc.org**), HSS will have approximately the same reliability as a rolled shape. This is because the allowable mass variation is similar to that of other shapes when the design wall thickness is the basis for design. Here is why:

Based on ASTM A500 for an HSS, the minimum area as a percentage of the design area is 0.90 / 0.93 = 96.8%

Based on ASTM A6 for a wide-flange shape, the minimum area as a percentage of the design area is 100% – 2.5% = 97.5%

ASTM A1085-13 also merits mention here. It's a new HSS standard that includes a mass tolerance, and we expect it will allow the design rules in AISC 360-16 to use the nominal wall thickness in design for ASTM A1085 HSS. For more on A1085, see **www.aisc.org/a1085**.

Bo Dowswell, P.E., Ph.D.

Lateral-Torsional Buckling of Rectangular Bars and Rounds

The limit shown for applying Equation F11-1 in the AISC *Specification* is dependent on the thickness. For solid round bar, should I assume the thickness to be the diameter of the round?

Section F11 in the AISC *Specification* requires you to check yielding and lateral-torsional buckling.

The *Specification* states that yielding applies for rounds, rectangular bars bent about their minor axis and rectangular bars bent about their major axis that comply with the following compactness limit:

$$\frac{L_b d}{t^2} \leq \frac{0.08E}{F_y}$$

Lateral-torsional buckling will not govern the strength of rounds and rectangular bars bent about their minor axis. Lateral-torsional buckling also does not apply to closed sections.

Carlo Lini, P.E.

Fixity of Truss Web Members

The diagonal of a truss is a double-angle long-leg back-to-back welded to the stem of the WT chords. The connections are not detailed to resist moment. However, the typical detail will provide some moment resistance. Is it possible to determine a reduced effective length factor, K value, of a truss diagonal for buckling in the direction of the plane of a truss?

Your proposed approach is possible. However, I can think of a few potential issues.

First, there is the obvious question of moment continuity between the web and chord. I think you should check the weld to ensure it can resist a sizeable percentage of the double-angle flexural strength. Unfortunately, I don't know how to define "sizeable percentage," so you must use your own engineering judgment. See the fourth full paragraph on Page 16.1-514. You might choose to make an adjustment for connection flexibility.

Second, the alignment charts were developed using the multitude of assumptions described in the surrounding text. Study these assumptions carefully to make sure your condition is consistent with the assumptions, or at least close enough that the approach can still be used. It is likely that adjustment must be made as described in the second paragraph on page 16.1-514 (girders with significant axial load). This adjustment must, at the very least, be performed due to compression chord axial load.

Third, you'll have to be consistent in analysis and design to avoid violating Section B1, which states "The design of members and connections shall be consistent with the intended behavior of the framing system and the assumptions made in the structural analysis." Your structural model can't use pinned web-to-chord connections to determine the force effects in the webs and chords if your strength calculations are based upon continuous connections. Whether the discrepancy between your analysis and strength calculations is significant depends on the details of your truss and analysis.

Finally, flexural-torsional buckling per Section E4 applies also. The flexural-torsional buckling strength will not be helped by reducing K about the x-axis.

Ultimately, there is a lot of work to make what may be a comparatively small gain. And you may be complicating the connections (which cost more) to save member weight (which costs less).

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

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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.

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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