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.

Penetration into the Base Metal

I am concerned with a fabrication shop that is using E70C-6M. When inspecting heavy columns, I noticed the welds at the gusset plate have little or no penetration into the base metal. My wire book is saying the wire is good on carbon steel up to 70 ksi. Is the gas-shielded metal-cored wire, which I see as having little penetration, acceptable on structural steel because the drawing calls for the use of E70XX filler metal?

Duane Miller of Lincoln Electric provided his thoughts:

The "key" to welding is fusion, not necessarily penetration. The two often get confused.

Fusion is achieved when filler metal and base metal are melted together and fused together. Penetration refers to a specific amount of melting into the base metal. Typically, when penetration is achieved, fusion is assured. However, if penetration is minimal or nonexistent, then fusion is often uncertain (as appears to be the case in this situation).

To your question regarding acceptability, the answer is yes; the filler metal meets all the requirements of AWS D1.1:2006, Table 3.1. The AWS classification of E70C-6M meets the requirements shown on the drawings for E70XX. However, I hasten to add the caveat that, as with all welding, the welding parameters (amps, volts, travel speed, preheat, material cleanliness, etc.) must be such that proper fusion is achieved. With your report of "little or no penetration," you should not assume that fusion is being achieved, even though the electrode meets the AWS D1.1 requirements.

There is no inherent problem with the use of this particular classification of electrode, but as is the case with all electrodes, improper procedures and techniques can result in poor quality welds. If there is no fusion, there are major procedural problems that must be addressed. Short-circuiting transfer associated with GMAW welding, low currents, small diameter electrodes, and other factors can all lead to conditions wherein fusion is not achieved.

Rather than focusing on the electrode, the concern should be on the welding parameters and quality. Make sure the WPS meets the requirements of AWS D1.1 and that the electrode is operated within the manufacturer's recommended parameters. If there are remaining concerns about the suitability of the welding electrode and the welding procedure parameters being used, it would be appropriate to perform some mechanical testing on welded connections. Simple fillet weld break tests, which are normally used for tack welder qualification, can be used to identify fusion problems. See AWS D1.1 Figure 4.35 for an example of such tests. A good weld will fail through the throat. If fusion problems exist, failure will be along the face of the base metal.

Section Properties

I am trying to calculate the I_x values found in Section 1 of the 13th edition AISC manual. The numbers that I get are close, but don't quite match what is printed. Are you including the fillets in the calculation? What shape do you assume they are? Where is the thickness of the flange equal to t_f when shapes have an inner flange surface slope? Is it at the location half way between the face of the web and the end of the flange?

Yes, the fillets are included in the calculation of the section properties listed in the AISC manual tables. The fillet radii used are based on surveys we conduct of fillet practices used by producing mills, and the calculations are based upon a parabolic fillet in a right angle as shown on page 17-40 of the 13th edition AISC *Steel Construction Manual*. The t_f of a shape with a sloping flange depicts the average flange thickness taken between the tip of the flange and the transition point to the fillet.

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

PJP Weld Capacity

When AISC gave a seminar on the new code, they gave out laminated "short-cut" cards that identify the ASD tension capacity of a PJP groove weld as $0.32F_{EXX}A_w$. I am having trouble coming up with the origin of the 0.32 factor. For ASD I have always used an allowable shear stress of 0.3 times the nominal tensile strength of the weld metal. Can you explain the difference?

I think that your confusion stems from the distinction between the directions of applied load with respect to the axis of the weld. You will note that the laminated cards give a value of $0.30F_{EXX}A_w$ for shear on the weld, but $0.32F_{EXX}A_w$ for tension on the weld.

Available strength of welded joints is covered in Table J2.5 of the AISC specification (a free download at **www.aisc.org/2005spec**). Note that if you are looking at *tension* normal to the weld axis, the nominal strength is $0.60F_{EXX}$, and $\Omega = 1.88$. If you are looking at *shear*, the nominal strength is $0.60F_{EXX}$, and $\Omega = 2.00$. It will thus follow that the allowable ASD coefficient for the tension case would be 0.60/1.88 = 0.32, while that of the shear case will be 0.60/2.0 = 0.30.

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

Single-Angle Bending

Upon reviewing a condition I have encountered of an unequal-leg single angle bent about a geometric axis (X-X in this case) with no restraint against lateral-torsional buckling along its length, I do not see in AISC specification Section F10-2 any subsection that encompasses this condition. Is it advisable to assume that since this condition is not explicitly covered with a definition for M_e, that there is no case in which LTB will cause the failure of an unequal-leg single angle bent about a geometric axis? If not, which equation would be applicable to determine the LTB criteria for this condition?

No, lateral-torsional buckling does apply. Refer to the commentary of section F10 at the top of page 16.1-282 for your condition.

The biaxial bending can be treated using the provisions in Chapter H to combine the forces around the principal axes. Leg local buckling may also limit the flexural strength for noncompact angle legs as per Section F10.3

Amanuel Gebremeskel, P.E.

Slip-Critical Surface Classes

AISC 341 lists only Class A surface preparation for bolted connections that are part of the SLRS. This would appear to eliminate galvanizing as a corrosion control method, since, when roughened, it is considered a Class C surface. Is this correct?

No. There is no longer a Class C surface designation in the AISC specification. Class A surfaces include hot-dip galvanized with roughened surfaces. See Section J3.8 of the AISC specification (a free download at **www.aisc.org/2005spec**) for guidance.

Note that the RCSC specification has not been updated since the release of the 2005 AISC specification, and, therefore, the current 2004 release still contains the Class C reference.

For further discussion on the subject of combining the formerly separate Classes A and C into a single Class A, see the Commentary to Section J3.8 of the 2005 AISC specification.

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

Edge Distance for Anchor Rod Holes

What and where are the requirements for edge distance for the recommended maximum-size holes for anchor rods listed in table 14-2?

AISC does not directly address edge distance requirements for anchor rod holes in base plates. This is a matter of engineering judgment because only the designer can anticipate what types of loads the hole is likely to see. See FAQ 7.1.7 at **www.aisc.org/faq** for further discussion on the subject.

It is generally not recommended (see AISC Design Guide 1, *Column Base Plates*), but if shear is being transferred through anchor rods and the engineer can anticipate which anchors can reasonably be expected to bear against the oversized holes, the provisions of section J3.4, J3.10, and J4 may be helpful.

Amanuel Gebremeskel, P.E.

Historic Steel Beam Designation

We are renovating a job dated 1914. Beams are called out as 9×21. Is this an "I-shape," 9 in. deep at 21 lb. per ft? Is there a source for the old beam properties?

Yes, it is very likely an I-shape, and yes, there are sources that contain information on old beam shapes. The early 1900s era preceded the formation of AISC and the publication of the AISC manuals. However, AISC has Design Guide 15, which is a reference for historic shapes and specifications. This guide includes information on many shapes that predated the AISC manuals. The document is available at **www.aisc.org/epubs**. AISC also has a shapes database (v13.0 for contemporary shapes and 13.0H for historic shapes) available on the same web site.

It is likely that the 9×21 shape of your inquiry was an "American Standard Beam" produced by one of the mills of the time. Today these are called S-shapes and are listed as such in the design guide. There are multiple listings for shapes with the S9×21 designation because there was more than one producer of the shape and the properties may not have been identical. Despite the variations in cross-section, the S9×21 would be nominally 9 in. deep and weigh 21 lb per ft.

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

Single-Plate Connections with Axial Loads

Is it acceptable to design single-plate connections for shear and axial loads? If so, what are the design criteria for the axial load?

The tabulated values for shear connections in the AISC manual do not address beam connection with axial loads. However, yes, it is acceptable to design single-plate connections for shear and axial loads. The same limit states that are covered in Chapter J of the AISC specification (such as bolt shear, bearing strength at bolt holes, tension, shear, block shear, etc.) would be applicable.

The extended single-plate connection procedure provided on page 10-102 is intended as a starting point for cases that deviate from the limitations imposed on the more traditional conventional configuration. It is used in combination with consideration of axial force in several examples in the AISC *Seismic Design Manual*. As one example, see Example 5.2.

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

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