

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.

How to Specify AESS

I have designed a monumental stair that is designated as architecturally exposed structural steel (AESS). How should I specify AESS so the finished product meets with our expectations? How do I determine the cost increase for the specification of AESS?

There are distinct differences in the fabrication and cost of structural steel designated in the contract documents as architecturally exposed structural steel (AESS). These differences include differing tolerances, handling procedures and erection procedures for AESS when compared to structural steel not designated as AESS. The AISC *Code of Standard Practice*, Section 10 stipulates the requirements for AESS members. These include requirements such as tighter tolerances for straightness and smaller uniform gaps at copes, to name a few.

Discussing your expectations with the fabricator is the best way to match expectations and budget. To start that process, AISC has several references on AESS that will help all to understand what to specify and what to expect. There is an AESS reference discussing the AESS *Specification* that was developed jointly by the Structural Engineers Association of Colorado and the Rocky Mountain Steel Construction Association. It can be found at the following link: www.aisc.org/uploadedFiles/Steel_Solutions_Center/Conceptual/My_Project/Files/ArchitectsGuide.pdf.

AISC also publishes a brochure that discusses various coatings, which is free to download at: www.aisc.org/store/p-1500-architecturally-exposed-structural-steel.aspx.

This brochure includes a cost matrix to determine a conceptual estimate for your AESS project.

The Canadian Institute of Steel Construction recently published a brochure covering AESS. It is also free to download and can be found at: www.architecture.uwaterloo.ca/faculty_projects/terri/steel/AESS-FINAL.pdf.

Erin Criste

Loads on Welds

Can you explain what is meant by the load type and direction relative to the weld axis in AISC 360 Table J2.5?

I will use Figure C-J2.11(b) in the Commentary to the AISC *Specification* for reference. Tension and compression normal to the weld axis would be a load that is transferred between the plates through the weld in the direction of the line 3-3. Shear would be a load that is transferred between the plates through the weld in the direction of the line 2-2 or into or out of the page. Tension or compression parallel to the weld would be a compression or tension force distributed through the section (both plates) that does not cause shear in the welds. Since this type of loading requires no transfer of the force through the weld, Table J2.5 states: "Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts."

Larry S. Muir, P.E.

Gusset Plate Design

This question relates to the average stress calculation in Example IIC-2 on page IIC-34 of the AISC *Design Examples* v14.0. Why is the bending stress included in this calculation? For example, if there were no tension stress and no shear stress, then the average stress calculation would equal the peak stress by this formula. This does not make sense. Should not the average stress be computed by considering all stresses over the full length of the weld? If this is done, the bending stresses would sum to zero and would not contribute to the average stress.

You are correct that the average flexural stress is zero over the full length of the joint. However, the flexural stress does have an effect on the weld size and we are trying to capture that effect, so we look at the flexural stress over each half of the joint length.

Richard (see the *Proceedings* of the 1986 AISC National Steel Construction Conference, available at www.aisc.org/epubs) found that the ratio of the maximum force per unit length of gusset edge joint was 1.4 times the average force per unit length. This ratio was changed to 1.25 in a paper by Hewitt and Thornton (see 1st Qtr. 2004 AISC *Engineering Journal*, also available at www.aisc.org/epubs) based on a probability analysis.

Since we don't know the true or actual force distribution at the joint, and because the fillet welds are loaded with some transverse component of force, the calculation given in the example you cite is an attempt to follow the original work of Richard by calculating a peak and an average force per unit length of the joint. Then, to ensure some ductility in the fillet welds, they are made to accommodate either the peak force or 1.25 times the average, whichever is larger.

Bill Thornton, P.E., Ph.D.

Field Bolt Quantities

Is there an AISC requirement for a fabricator to supply extra field bolts and if so, what percentage of extra bolts are we required to furnish? Does this apply to only connection bolts or also to anchor bolts and other concrete anchors such as expansion and epoxy anchors?

This topic is addressed in AISC *Code of Standard Practice* Section 7.8.3 (a), which states that when the erection is not performed by the fabricator, the fabricator shall furnish "Bolts, nuts and washers of the required grade, type and size and in sufficient quantity for all *structural steel-to-structural steel* field connections that are to be permanently bolted, including an extra 2% of each bolt size (diameter and length)." Since the anchor rods and other concrete anchors are not part of a structural steel-to-structural steel connection, they are not included in this requirement.

Keith Landwehr

steel interchange

K-Area Welding

Does AISC 360 prohibit welding in the *k*-area of a wide-flange shape?

The AISC *Specification* does not prohibit welding in the *k*-area. There have been some reported problems with welds made in the *k*-area, so it is generally avoided, when possible. Nonetheless, there are times where welding in this area is unavoidable. For more information on this topic you can refer to the following article, which can be downloaded at www.modernsteel.com/archives: "AISC Advisory Statement on Mechanical Properties Near the Fillet of Wide Flange Shapes and Interim Recommendations, January 10, 1997" (02/97).

AISC 358 Section 3.6 (and its associated Commentary) describes requirements for continuity plate corner clips. Although this is not a direct prohibition of welding in the *k*-area, the resulting corner clip geometry is intended to avoid welding in the *k*-area.

When welding in the *k*-area is performed, it should be noted that AISC 360-10 Chapter N Table N5.4-3 requires visual inspection: "When welding of doubler plates, continuity plates or stiffeners has been performed in the *k*-area, visually inspect the web *k*-area for cracks within 3 in. (75 mm) of the weld."

Larry S. Muir, P.E.

Bearing Strength at Bolt Holes in HSS

AISC 360-05 section J3.10(c) refers the user to Section J7 and Equation J7-1 for bearing strength at bolt holes in an "unstiffened box member or HSS." Does a round HSS with a cap plate qualify as a stiffened member, such that Section J7 does not apply?

No. The bolt bearing strength equations in J3.10(a) and (b) were developed based on testing of plies that were confined due to the presence of a bolt head on one side and a nut on the other. This is not true of through-bolted HSS connections. The appropriate limit state for this condition is that of pin bearing rather than bolt bearing.

A stiffened HSS is one that has internal elements that provide confinement to the joint such that it will behave in bearing as a bolted joint, rather than as a pin joint. A cap plate will not accomplish this. An example of a stiffening element that will accomplish this is a tubular insert that spans the interior of the HSS between bolt holes and has an inside diameter approximately equal to the hole diameter. Such a detail likely would be more expensive to fabricate, and so it may be more desirable to just design with the pin bearing equation. I am not aware of any testing that would define how to design the internal stiffening elements. This is left to the judgment of the engineer.

Heath Mitchell, S.E., P.E.

RBS Welding

The RBS connection design example in the AISC *Seismic Design Manual* shows a single-plate web connection with erection bolts in a special moment frame (SMF). There is no shop welding information for the single plate. There is a CJP groove weld called out for the beam web-to-column flange joint. On one of our projects, the structural drawings called out the CJP groove weld, but the contractor also used a fillet weld between the single plate and the column flange for erection purposes. Since it is for erection purposes only, is it acceptable to fillet weld the single plate to a column?

AISC 358-10 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* Section 5.6 discusses requirements for single-plate web connections used in RBS moment connections. For SMF connections it states: "The single-plate shear connection shall be permitted to be used as backing for the CJP groove weld [between the beam web and column flange]."

Keeping in mind that the shear tab and its attaching welds must be sufficient to accommodate construction loads, the shear tab may be welded with a fillet, a PJP groove weld, or a CJP groove weld. Double-sided fillet welds are less desirable, as this puts a fillet weld in the root of the beam web-to-column flange CJP groove weld, which will be welded in the field. My experience has been that a shop welded PJP groove weld, placed on the opposite side of the shear tab (that is, on the non-CJP side), is common if the one-sided fillet is not sufficient to accommodate the construction loads.

Keith Landwehr

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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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