( ).

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 , 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 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:

AISC also publishes a brochure that discusses various coatings, which is free to download at: 1 00-

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:

3 0 2.

I will use Figure C-J2.11(b) in the Commentary to the AISC 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.”

-2 -34 *Design Examples* 14.0.

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 of the 1986 AISC National Steel Construction Conference, available at) 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 , also available at) 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.

The AISC does not prohibit welding in the -area. There have been some reported problems with welds made in the -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 : "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 -area, the resulting corner clip geometry is intended to avoid welding in the -area.

When welding in the -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 -area, visually inspect the web -area for cracks within 3 in. (75 mm) of the weld."

3 0-0 3.10( ) -1

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.

Seismic Design Manual

AISC 358-10 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.