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!

Seismic Provisions & Bolted Joints

I am required to use the 1997 AISC *Seismic Provisions* for a braced frame detail with typical loads and configuration, except that it has a significant axial transfer force from the beam to the column. The only way to get this transfer force from the beam into the column (sometimes to the flange and sometimes to stiffeners in the column web) is to weld the flange directly to the column. The weld is a CJP groove weld.

Is it still allowable to use pretensioned bearing bolts in the gusset to column connection (again, it might be to flange or web)? The gusset to column connection is either two angles or an end plate with shear plates along the column web.

My opinion is that pretensioned X-bolts are acceptable in this situation, since the weld is not sharing load in the same axis with the bolts and since the CJP groove weld in the beam flange to column, if properly detailed with proper weld access holes, will have sufficient ductility/flexibility to accommodate the small potential slip under full load. What about low-seismic situations?

Question sent to AISC's Steel Solutions Center

You asked a good question. The answer is in two parts:

1. Assuming the detail is based on a seismic load resisting system (i.e. frame) found in the 1997 AISC *Seismic Provisions,* such as a SCBF or OCBF, then Section 7.2 applies to bolted joints. That is, all bolts shall be pretensioned high-strength bolts. All faying surfaces shall be prepared as required for Class A or better slip-critical joints. However, the design shear strength of the bolted joints is permitted to be calculated as that for bearing joints (i.e. prepare as a slip-critical joint with design strength based on the larger bearing joint values rather than the smaller slip-critical values.)

   Please note that in the *Seismic Provisions,* Section 7.2 does not allow bolted joints to share load in combination with welds on the same faying surface, which appears to be satisfied in your case. Also, refer to Section 7.2 for handling the design of the bolted joints with combined tension and shear (due to the brace loading.)
2. If your detail is based on low-seismic (i.e. the *Seismic Provisions* do not apply) or a gravity frame within a building that utilizes the AISC *Seismic Provisions,* then pretensioned joints are required only in the situations described in Section J1.11 of the 1999 *LRFD Specification* (a free download from www.aisc.org/lrfdspec) and Section 4.2 of the 2000 RCSC *Bolt Specification* (a free download from www.boltcouncil.org.)

Sergio Zoruba, Ph.D.
American Institute of Steel Construction

Grout Packs & Base Plates

Is there a maximum thickness recommended for a grout pack (assume a non-shrink grout) under a column base plate? Assume there are leveling nuts under the plate. Assume a multi-story structure. Would the grout thickness be predicated on the thickness of the base plate and the open space beneath it? Assume the method or placement (packed, poured, injected) will adequately fill the space between the concrete pier, footer, etc. and the underside of the base plate.

Question sent to AISC's Steel Solutions Center

Grout packs for large base plates are often used with thicknesses in the 1″ to 2″ range. Often relief holes are provided in the interior of the base plate to relieve the entrapped air as the grout is pumped from the edge. Yes, there will be a maximum thickness limitation but this is a function of the grout capacity. Most grouts perform well in pure compression, but when the height of the grout pack becomes excessive bulging forces occur around the perimeter that can lead to failure of the grout. You should consult the grout manufacturer to determine recommended maximum thicknesses for the application of their product.

Kurt Gustafson, S.E., P.E.
American Institute of Steel Construction

Notches in Wide-Flange Beams

If you take away most of one side of a top flange of a steel beam, say an 8″ flange and on one side you notch out 3″, what are some acceptable methods of reinforcement? There will be a pipe going through the notch, and I cannot do anything from the top. Is welding a plate of sufficient area to the underneath of the opposite flange ok? I see how this will replace the section modulus and compressive block that I am losing, but I am concerned with how the internal stresses of the beam "jump" from one side to the other.

Question posted on SEAINT (www.seaint.org)

A lot depends upon the details of the slab (if there is a slab) and the location of the material removed. The interconnection to the slab may be enough to handle any out-of-plane effects that develop due to the asymmetry. On a short segment, it is most important to extend the reinforcement beyond the zone of the cut to allow for the redistribution that has to occur from the full cross-section to the reduced and back to the full. Two ways I can think of to do this:

1. Consider the situation analogous to shear lag in a tension member that is connected at some but not all of the parts of the cross-section and use a $1-x/L$ calculation to determine the length to extend the reinforcement beyond the zone of the cut.
2. Pretend there are splices to be designed at each edge of the cut zone to connect it into the full member. Determine the force to be transferred and determine the length of flange/weld needed to do so. That will then tell you how long to extend the reinforcement.

Without running any numbers, I think these will work out to similar values.

Charlie Carter, S.E., P.E.
American Institute of Steel Construction

OSHA & Double Connections

We are reviewing our detailing standards and, the question of what "exactly" is required by OSHA concerning double connections was asked. I am sure you have addressed this question many times and hopefully have an (end all) answer. Could you please help us to clear up the confusion we have concerning this issue?

Question sent to AISC's Steel Solutions Center

The OSHA Regulations are printed in the Federal Register available on the OSHA web site at www.osha.gov.

You are looking for *1926 Subpart R – Steel Erection.* In there you will find a definition of "Double connection means an attachment method where the connection point is intended for two pieces of steel which share common bolts on either side of a central piece." The application to beams and columns is covered in article 1926.756.

Kurt Gustafson, S.E., P.E.
American Institute of Steel Construction

Composite HSS Column - Minimum Wall Thickness

Table 4-13 (Composite HSS Compression Members) in the 3rd edition *LRFD Manual* seems to include HSS that will not satisfy the minimum thickness requirement found in Section I2.1 of the 1999 *LRFD Specification.* For example, for an HSS 16×16×3/8, the minimum t would be 0.368". This is greater than the listed t_{design} = 0.349", but is less than the nominal t of 0.375". Should we be using the nominal thickness rather than the t_{design} for the minimum t criteria (which would contradict what is currently used for W-shapes, for example) or should this HSS be "disqualified"?

Question sent to AISC's Steel Solutions Center

The minimum thickness expression in Section I2.1 was adopted from ACI 318 to address local buckling concerns. As such, the width b from Section I2.1 should be based on Section B5.1 as b_e (effective width of a stiffened compression element.) That is, b_e is defined as the clear distance of the rectangular HSS wall, which is typically $b - 3t_{design}$.

Hence the Section I2.1 thickness expression would result in a minimum required thickness of only 0.344 inch. Since t_{design} = 0.349 inch, this requirement is satisfied.

Sergio Zoruba, Ph.D.
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Preheat & Stud Welding

I have two questions:

1. **When welding headed studs to beams or plates using the standard welding gun, are there any requirements to preheat the base material in cases where the base material is say 2- or 3-inches thick or more? I know that when the stud is fillet welded, AWS specifies that the base material must be preheated but there is no mention of preheat when the stud gun is used.**
2. **What problems can be expected if proper preheat is not provided and conventional fillet welds are used? Cracks in base metal? Cracks in weld? Other?**

Question sent to AISC's Steel Solutions Center

As you indicated, AWS D1.1 does not set preheat requirements for stud application by Automatic Welding Machine. Instead they refer to past practices and recommendations of the stud and equipment manufacturer, or both. AWS considers stud welding as being unique among the approved welding processes in their Code. This is largely due to the special testing requirements for stud applications. AWS has a publication AWS C5.4 Recommended Practices for Stud Welding. We do not have a copy available in our office; therefore, do not know if there is any information included relative to preheat requirements. You may want to contact them at www.aws.org.

Typically, preheat is used to help control the cooling rate of weld metal and adjacent base metal. When cooling is sufficiently slow, it helps to reduce hardening and cracking. Where cracking could occur if proper pre-heat is not employed is very difficult to predict. It could possibly occur in the weld metal, base metal or both.

Kurt Gustafson, S.E., P.E.
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Steel Interchange is a forum for *Modern Steel Construction* readers 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.

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If you have a question or problem that your fellow readers might help you to 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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