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## Plates as Beams

**I'm used to the old ASD approach. How do I design a plate in strong-axis bending using the 2005 AISC *Specification*?**

A plate designed as a main member in strong-axis bending would be designed according to Section F11 of the 2005 AISC *Specification*. Typically strong-axis bending will not govern the strength of plates used for connections, except for the extended configuration of the single-plate shear connection. Bending in this case is checked as described on page 10-103 of the 13th Edition AISC *Steel Construction Manual*.

*Larry S. Muir, P.E.*

## Weld Metal Choice in Seismic Applications

**Are my choices of electrode strength level more limited in high-seismic applications? Specifically, where could I choose to use E60 electrode for welding ASTM A36 material?**

For the purposes of this inquiry there are three types of welds in a high-seismic application (when AISC 341 applies): Demand Critical, which is required where AISC 358 says it is required; AWS D1.8 welds in the seismic force resisting system other than those that are Demand Critical; and AWS D1.1 welds for connections that are not in the seismic force resisting system (SFRS).

The welds outside the SFRS meet D1.1 requirements that define E60 as matching weld metal and permit its use in prequalified welding procedure specifications (WPS) where the connected base metal is A36 and less than ¾ in. thick (see AWS D1.1 Table 3.1 Category I). In the SFRS, AWS D1.8 requires the use of E70 or E80. Additionally, the filler metals used in these welds need to be Charpy V-notch (CVN) tested and exhibit 20 ft-lb at 0 °F or better (see AWS D1.8 Section 6.3). Demand Critical welds add a further requirement that the filler metal must be tested to show a level of toughness at a range of heat inputs by passing heat input envelope testing.

*Thomas J. Schlafly*

## Minimum Percentage for Composite Design

**The Commentary to the AISC *Specification* recommends that small levels of partial composite design (low percentages) should not be used. I like that it is left to my engineering judgment, but what guidance can you give me?**

AISC has used 25% as a minimum amount of composite action in tables and information in the AISC *Manual* because low levels of composite action may require significant deformations to achieve the strength. Figure C-I3.5 of the Commentary to the 2005 AISC *Specification* (which is available as a free download at **www.aisc.org/2005spec**) illustrates this graphically. Moreover some tests have shown that below composite action ratios of 25% some physical separation between the steel and concrete may be exhibited at maximum loading.

*Amanuel Gebremeskel, P.E.*

## Fire protection for HSS

**Can I fill an HSS or steel pipe with concrete to serve as fire protection?**

Yes. Filling with concrete works better if it is reinforced (the rebar maintains the heat sink capability as the concrete inside the section cracks when exposed to heat). A calculation procedure to determine the fire rating is discussed in AISC *Steel Design Guide 19*, which is available as a free download for AISC members at **www.aisc.org/epubs**.

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

## Sawing Inside Corners

**Can I use a band saw to cut an inside corner square or do I have to form a radius at the intersection of the sides of the cut?**

It is not acceptable to cut an inside corner square, and I also should point out that an "overcut" at this corner creates an even worse condition. Section M2.2 of the *Specification* states:

"Reentrant corners, except reentrant corners of *beam copes* and weld access holes, shall meet the requirements of AWS D1.1, Section 5.16. If another specified contour is required it must be shown on the contract documents."

*Larry S. Muir, P.E.*

## Tee Stem in Compression Due to Bending

**I'm comparing the 2005 AISC *Specification* (Chapter F, Section F9 as well as F6 and F11) and an "ancient" article that was published in the 1965 AISC *Engineering Journal* titled "One Engineer's Opinion," by William A. Milek. There are some differences between these references; does the 2005 information agree with the Milek paper?**

The Commentary to Section F9 (see page 277) discusses the derivation of the lateral-torsional buckling equation used in the 2005 AISC *Specification*. There is a reference in this information to the 5th Edition of the *Guide to Stability Design Criteria for Metal Structures*, while the Milek paper references the 1st Edition of this same document (note that the 6th Edition is now current, available at **www.stabilitycouncil.org**). Both references recognize that the solution of this limit state is somewhat complex, and each provides a simplified equation for WTs in bending.

The 2005 AISC *Specification* benefits from additional, more recent research, which allows a further simplification beyond what the Milek paper recommended. The effect of the stem in compression on the lateral-torsional buckling strength is accounted for with the B factor. It is negative for stems in compression, thus reducing the available flexural strength.

*Heath Mitchell, P.E.*

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## Balancing Welds for Snow Load?

**AISC *Specification* Section J1.8 requires weld balancing for unsymmetric members subjected to cyclic loading. Should the designer consider snow load as a cyclic load?**

No. Snow load is not considered to be a cyclic load. Cyclic loading is something that produces more than 20,000 cycles in the life of a structure, like a crane or vibratory equipment.

*Amanuel Gebremeskel, P.E.*

## Strength Increase for PJP Groove Welds?

**I know that you can take up to a 50% increase on the strength of fillet welds loaded other than longitudinally. I'm wondering whether this is also applicable to PJP welds. Does the reasoning behind the increase for the fillet weld also apply for PJP groove welds?**

The AISC *Specification* does not apply the 50% increase to PJP groove welds. The use of the 50% increase is based on testing of fillet welds that indicated this design approach is safe. No such testing has been conducted on PJP groove welds.

Although it has been discussed, there is no agreement about what the results of these PJP tests would likely be, if they were conducted. Some believe a similar strength increase would be found to apply. Others, noting that the fracture plane was nearer the leg than the 45° assumed throat in the transversely loaded fillets, argue that the strength increase will not be present in PJP groove welds since the geometry of the weld is different.

*Larry S. Muir, P.E.*

## Cracking in Composite Slabs Over Girders

**We have a building with large cracks over the girders. I know these are not structurally of concern, but why did they occur and what can I do to prevent them?**

It is common to have cracks over the beams and girders that frame into columns in unshored slabs. These often are thought to be due to negative moment, but also can occur due to shrinkage. The slab is thinnest at those locations and it will crack somewhere.

Often, these cracks do not matter. However, if the floor treatment is sensitive to the presence of these cracks, reinforcement can be provided over the support to prevent one large crack (and instead cause several smaller cracks to occur over a distributed width).

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

## Paint on Faying Surfaces

**Was there a recent change in the AISC *Specification* that now allows paint on the faying surfaces of slip-critical joints?**

There has been no recent change, but perhaps the change you are referring to is an old one. It is (and has been for some time now) permissible to use paint on faying surfaces of joints as stated in Section 3.2 of the RCSC *Specification* (a free download at **www.boltcouncil.org**). That section allows paint unconditionally in bearing-type joints and allows paint that has been qualified to achieve the necessary slip resistance for slip-critical joints. AISC requirements follow what is in the RCSC *Specification*.

*Amanuel Gebremeskel, P.E.*

## Finding an AISC Member Fabricator or Erector

**I'm trying to find an AISC member fabricator. Does AISC provide such a list?**

Yes. There is a tab at the top of the AISC website that says "Find a Company/Person" or you can use the URL **www.aisc.org/members**.

This is a directory search of AISC member fabricators and erectors. There is a drop-down menu on this page where you can select a fabricator or an erector. Also, you can search by city and/or state for a person or company.

*Erin Criste*

## Axial Force and Rotational Ductility in Shear Connections

**I am designing a beam end connection for combined axial force and shear force where the axial force is large compare to shear force. A double angle connection is not workable, but I can use a shear end-plate connection detail with a thickness of ¾ in. How can I do this when for flexibility the maximum thickness is limited to ⅝ in.?**

You have a couple of options.

(1) The rotational ductility requirements are based on shear connections that have proven capable of accommodating the simple beam end rotation, which usually is assumed to be 0.03 radians. If the beam is subjected to a large axial load relative to the shear load, it may not develop as large an end rotation and you might decide that the rotational ductility checks (maximum thickness in this case) can be relaxed.

(2) You also could apply the rotational ductility checks given in Part 9 of the 13th Edition AISC *Steel Construction Manual* to determine the acceptability of the detail. These ductility checks are based on the worst case assuming an ASTM A325 bolt is used. If ASTM A490 bolts are used the minimum bolt diameter could be less.

It is common to relax the ductility procedures in the *Manual* when beams are subjected to significant axial loads, because these beams will often have reduced end rotations.

*Larry S. Muir, P.E.*

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Charlie Carter is vice president and chief structural engineer, Tom Schlafly is director of research, Amanuel Gebremeskel is a senior engineer and Erin Criste is steel solutions center advisor at AISC. Larry Muir, Heath Mitchell and Brad Davis are consultants to AISC.

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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:

Steel SolutionsCenter
One East Wacker Dr., Suite 700
Chicago, IL 60601
tel: 866.ASK.AISC • fax: 312.803.4709
**solutions@aisc.org**