

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](mailto:solutions@aisc.org).

### Drift Limits

**For a single-story steel moment frame building with CMU non-load-bearing walls, what requirements control the analysis requirements and allowable lateral displacement of the frame?**

The intent of the AISC 360 *Specification* (a free download from [www.aisc.org/2010spec](http://www.aisc.org/2010spec)) is to provide requirements related to the design and detailing of steel systems for the forces resulting from an analysis. It is not intended to dictate the analysis procedure itself. Analysis requirements are typically addressed in the building code, such as *IBC*, or in ASCE 7 in the absence of an applicable building code.

That said, there are a couple objectives to be considered in establishing analysis and lateral displacement criteria. First, there is structural stability, and this is addressed to some degree in the building codes as follows:

- Basic requirements in ASCE 7-10, Section 1.3
- Stability coefficient requirements in ASCE 7-10, Section 12.8.7
- Seismic drift limits as defined in ASCE 7-10, Section 12.12

Structural steel-related stability requirements are given in Chapter C of AISC 360-10, and some of the related methods and provisions are covered in Appendices 7 and 8.

There also are serviceability criteria. These are not addressed prescriptively in the codes and should be evaluated on a project-specific basis, taking into consideration the end user's needs and expectations, the architectural finishes (exterior and interior) and the detailing of how those finishes will attach to the structure and/or accommodate structure movements. ASCE 7-10 Appendix C addresses serviceability requirements, but in very general terms.

In my practice, I often refer to the 1993 AISC *Engineering Journal* article "Serviceability Limit States Under Wind Load," written by Larry Griffis. In this article, he discusses drift limits in depth and provides some guidelines for establishing limits based on different material finishes. Additionally, there is AISC Design Guide No. 3 *Serviceability Design Considerations for Steel Buildings*, 2nd Edition. These resources are available for free download by AISC members at [www.aisc.org/epubs](http://www.aisc.org/epubs).

Susan Burmeister, P.E.

### Grouting of Base Plates

**When should the base plates be grouted for a multistory structure?**

AISC Design Guide 10 (a free download at [www.aisc.org/dg](http://www.aisc.org/dg)) provides the following guidance on this subject: "Until the column bases are grouted, the weight of the framework and any loads upon it must be borne by the anchor rods and leveling nuts or shims. These elements have a finite strength. The timing of grouting of bases

must be coordinated between the erector and the general contractor."

It also states: "Leveling nuts bear the weight of the frame until grouting of the bases. Because the anchor rod, nut and washers have a finite design strength, grouting must be completed before this design strength would be exceeded by the accumulated weight of the frame. For example, the design strength of the leveling nuts may limit the height of frame to the first tier of framing prior to grouting. Also, it is likely that the column bases would have to be grouted prior to placing concrete on metal floor deck. Properly installed shim stacks can support significant vertical load. There are two types of shims: those placed on (washer) or around (horseshoe) the anchor rods. Shims placed on or around the anchor rods will have a lesser tendency to become dislodged. Independent shims must have a reasonable aspect ratio to prevent instability of the stack. In some instances shim stacks are tack welded to maintain the integrity of the stacks. When shim stacks are used, care must be taken to ensure that the stacks cannot topple, shift or become dislodged until grouting. Shims are sometimes supplemented with wedges along the base plate edges to provide additional support of the base plate."

AISC Design Guide 1 also provides some guidance. Section 2.9.1 states: "When designing anchor rods using setting nuts and washers, it is important to remember these rods are also loaded in compression and their strength should be checked for push out at the bottom of the footing. It is recommended that use of the setting nut and washer method be limited to columns that are relatively lightly loaded during erection."

Section 2.9.3 states: "Column erection on steel shim stacks is a traditional method for setting base plate elevations that has the advantage that all compression is transferred from the base plate to the foundation without involving anchor rods. Steel shim packs approximately 4 in. wide are set at the four edges of the base plate. The areas of the shim stacks are typically large enough to carry substantial dead load prior to grouting of the base plate."

Carlo Lini, P.E.

### PJP Groove Welds in Compression

**AISC Specification Table J2.5 provides three conditions related to partial-joint-penetration groove welds subjected to compression:**

- 1) Column-to-base plate and column splices designed per Section J1.4(1)
- 2) Connections of members designed to bear other than columns as described in Section J1.4(2)
- 3) Connections not finished-to-bear

**For case 2, the nominal strength of the weld is  $0.6 F_{EXX}$ . For case 3, the nominal strength is  $0.9 F_{EXX}$ . Why is the weld assumed to have less strength when the members are finished-to-bear than when the members are not finished-to-bear?**

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The Commentary to the AISC *Specification* provides the following information related to the first two cases:

“Column splices have historically been connected with relatively small PJP groove welds... Section M4.4 recognizes that, in the as-fitted product, the contact may not be consistent across the joint and therefore provides rules assuring some contact that limits the potential deformation of weld metal and the material surrounding it. These welds are intended to hold the columns in place, not to transfer the compressive loads. Additionally, the effects of very small deformation in column splices are accommodated by normal construction practices... Therefore the compressive stress in the weld metal does not need to be considered as the weld metal will deform and subsequently stop when the columns bear. Other PJP groove welded joints connect members that may be subject to unanticipated loads and may fit with a gap. Where these connections are finished to bear, fit-up may not be as good as that specified in Section M4.4, but some bearing is anticipated and the weld is designed to resist loads defined in Section J1.4(2) using the factors, strengths and effective areas in Table J2.5.”

Essentially what the Commentary is saying is that with a column, we expect pretty good (but not perfect) bearing. With members other than columns we expect pretty good (but maybe less perfect) bearing. We have a lot of certainty relative to what a column is, what its connection will look like and how it will behave. We have less certainty relative to what a member other than a column is, what its connection will look like and how it will behave, but we still design the weld for little load based on the fact that the members bear, so we knock down the strength of the weld to account for the uncertainty.

Now that we've compared Cases 1 and 2, let's compare Cases 2 and 3. For Case 2, we already discussed that we use 0.6 because we ask little of the weld in terms of the design load, but we have a good bit of uncertainty. For the members not designed to bear, we ask a lot of the weld, but we feel we have little in the way of uncertainty. For instance, for tension on a PJP groove weld, where we also apply the 0.6 factor, the Commentary states:

“The factor 0.6 on  $F_{EXX}$  for the tensile strength of PJP groove welds is an arbitrary reduction that has been used since the early 1960s to compensate for the notch effect of the unfused area of the joint, uncertain quality in the root of the weld due to the inability to perform nondestructive evaluation and the lack of a specific notch-toughness requirement for filler metal. It does not imply that the tensile failure mode is by shear stress on the effective throat, as in fillet welds.”

For PJP groove welds in compression, we're not really concerned with any of these factors, which explains why we permit a higher nominal stress for Case 3.

Larry S. Muir, P.E.

## Stiffened Plates in Flexure

**What section(s) in the AISC *Specification* can be used to determine effective width of stiffened plates used in built-up sections subjected to flexure?**

Because the AISC *Specification* is written with buildings and other structures similar to buildings in mind, there are no provisions for the effective width of plate in stiffened plate structures. The effective width used in design varies, depending on the type of structure you are designing (bin, stack, tank, ship, etc.). For general flat plate structures, API Bulletin 2V, *Design of Flat Plate Structures*, published by the American Petroleum Institute, can be used to determine the effective width.

A few of other sources may also be helpful:

- Page 6.6-7 of *Design of Welded Structures* by Blodgett uses an effective width of  $12t$  on each side of the stiffener, where  $t$  is the plate thickness. This is similar to the value in Section J10.8 of the 2010 AISC *Specification* (a free download available from [www.aisc.org/2010spec](http://www.aisc.org/2010spec)), which allows an effective width of web to be used in the design of stiffened beam and plate girder webs.
- Tables B4.1a and B4.1b of the AISC *Specification* can be used to determine the maximum effective width of compression elements.
- The steel stack code, ASME STS-1, allows an effective width of only  $8t$  on each side of the stiffener.

Bo Dowswell, P.E., Ph.D.

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Larry Muir is director of technical assistance and Carlo Lini is staff engineer-technical assistance at AISC. Susan Burmeister and Bo Dowswell are consultants to AISC.

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1 E Wacker Dr., Ste. 700, Chicago, IL 60601  
tel: 866.ASK.AISC • fax: 312.803.4709  
[solutions@aisc.org](mailto:solutions@aisc.org)



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