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steel interchange

Tension-Only OCBFs, Revisited

Page 5-77 of the 2nd Edition of the AISC *Seismic Design Manual* has an example of a tension-only ordinary concentric braced frame (OCBF) and states that the braces must satisfy the requirements for moderately ductile members. However, the June 2013 Steel Interchange addressed a question regarding *b/t* ratios for a cable. Part of that answer states the width-thickness limits are for "compression" elements. This answer makes sense. If designing a tension-only system, it would seem that a smaller angle with less compression capacity, that may not meet the moderately ductile requirements, would perform better. The November 2015 Steel Interchange also indicates that Table D1.1 is only intended for compression elements, as the table's title suggests. Is the example in the *Seismic Design Manual* incorrect in stating that braces in a tension only system must meet the requirements for a moderately ductile element? (Both Steel Interchanges are available at www.modernsteel.com.)

No. The design example is correct. The 2010 *Specification* contains no exception, so all braces in a OCBF must "satisfy the requirements of Section D1.1 for moderately ductile members" as stated in the example. However, the 2016 *Seismic Provisions* will state that for tension-only OCBFs, "Braces shall satisfy the requirements of Section D1.1 for moderately ductile members. Exception: Braces in tension-only frames with slenderness ratios greater than 200 need not comply with this requirement."

In response to a number of questions about tension-only bracing, let's review the *Seismic Provisions*, the design example and the two Steel Interchange answers you cite as they relate to this topic.

At the Steel Solutions Center, rather than simply providing yes or no answers, we try to shed some light on the technical basis for the provisions we cite and the conclusions we draw. The answers provided in each Steel Interchange are related to specific conditions. However, the background we provide is often related to more general structural principles. When interpreting our responses, both the specific situation and the general concepts should be considered.

The statements made in the 2013 and 2015 Steel Interchanges are correct. They are summarized in the following lists:

Summary of the 2013 Steel Interchange (related to cable bracing):

➤ The width-thickness limits are for "compression" elements.

➤ The term "by design" can be taken to mean "it may be assumed" in the design. Therefore, it may be assumed that there are no compression elements in a tension-only system. In such cases the width-thickness limits would not apply.

➤ However, it should be recognized that the braces may see some compression.

➤ If they are so slender that they buckle elastically, their compression capacity would be very small if it was accounted for and the width-thickness limits would make little difference in the performance of the system.

In other words, if you assume there are no elements subjected to compression, and the actual conditions are consistent with this assumption, then neglecting the width-thickness limits is reasonable.

Summary of the 2015 Steel Interchange (tension-only bracing in OCBF):

➤ Tension-only bracing is permitted in OCBFs.

➤ Local buckling does not apply to a rod.

➤ The *Manual* provides guidance that should be considered by engineers.

It should be noted that simply assuming that the system is tension-only may not be sufficient to ensure proper behavior. Designing the braces as tension-only involves an assumption that the braces see only tension, but the engineer must also have some reasonable expectation that the braces will see only negligible compression. A cable will certainly have negligible strength and stiffness in compression. Other sections may not. Your assertion that a smaller angle with less compression capacity, that may not meet the moderately ductile requirements, would perform as well if not better than a stouter section has some merit. However, members that can resist whatever compression is applied to them without local buckling are deemed to be acceptable as well. Ultimately, for the 2016 *Seismic Provisions*, the committee concluded that meeting the moderately ductile width-to-thickness limits was not necessary for braces with high slenderness.

Both the assumed and actual or expected behavior must be considered. The approach of the committee has continued to evolve and reflects this.

The 2005, 2010 and 2016 editions of the *Seismic Provisions* all allow tension-only OCBFs. All three editions permit the use of slender angles, plate and cable bracing as tension-only braces. It is advisable to use braces that have little compression capacity in tension-only braced frames, but it should be recognized that issues can occur when the braces are extremely slender. The 2016 *Seismic Provisions* may allow the use of lighter and more slender-angle tension-only brace members than were permitted under previous editions.

Though there are some differences and subtleties, all editions of the *Seismic Provisions*, the *Manual* design example and both Steel Interchanges are all correct and based on the same basic underlying principles.

Larry S. Muir, PE

HSS Beams on Stiffened Seats

An HSS beam bears on a stiffened seat at its end. Can *Specification* Equations K1-9 and K1-11 be used to determine the strength of the HSS sidewalls/web?

For typical cases, the answer is no. Equations K1-9 and K1-11 assume that the chord (the HSS member) runs through the joint, as illustrated in the figure that accompanies these equations in the *Specification*. Therefore, these equations cannot be used at the unreinforced ends of HSS members.

You have a few options:

1. You could provide a cap plate at the end of the HSS and use the Chapter K equations. New language in the Commentary to the 2016 *Specification* will indicate that the where a cap plate is used the Chapter K equations can be conservatively applied at/near the ends of HSS members.
2. You could provide a cap plate and assume the load is delivered to the cap plate and then transferred to the HSS through the welds of the cap plate to the sidewalls. This would eliminate the need to check the limit states in Chapter K.
3. You could recognize that equations K1-9 and K1-11 are based on equations J10-3 and J10-8 and modified to account for the two HSS walls instead of the single web assumed in Chapter J. With this in mind, you could simply apply the Chapter J checks and adjust for the two walls. This approach is probably the most consistent with the typical design of seated connections supporting wide-flange beams

We have made several changes to the 2016 *Specification* and its commentary to try to clarify the relationship between the checks in Section J10 and those in Chapter K.

Larry S. Muir, PE

Skewed Single-Plate Shear Connections

The July 2012 SteelWise article "Designing Welds for Skewed Shear Tabs" (available at www.modernsteel.com) discourages the use of single-sided PJP groove or fillet welds but provides no reason for this. What is the basis for this advice?

There is no prohibition against the use of single-sided PJP groove or fillet welds for single-plate shear connections in either the *Specification* or the *Manual*. The Commentary to the *Specification* does, however, contain similar advice. The Commentary to Section J2.1b states: "The use of single-sided PJP groove welds in joints subject to rotation about the toe of the weld is discouraged" and the Commentary to Section J2.2b states: "The use of single-sided fillet welds in joints subject to rotation around the toe of the weld is discouraged." The concern is that the rotation about the toe of the weld will subject the weld to torsion about its longitudinal axis, which will tend to place a lot of demand on the root of the weld. Such loading is not addressed in the *Specification*.

A single-plate shear connection is subjected primarily to shear as the name suggests, but a moment can exist and is accounted for in the design of the weld. This moment may cause rotation around the toe of the weld, so it is suggested in the article that engineers should "avoid single-sided fillet and PJP groove welds for shear tabs." In many cases, it could be argued that a floor slab running over the top of a beam will likely prevent such rotation about the toe of the weld.

Even without the concern related to rotation about the toe of the weld, the use of single-sided fillet and PJP groove welds may present other challenges. A single-sided fillet weld will likely be made on the obtuse side, which increases the amount of weld metal. Trying to satisfy the $5/8\,t_p$ recommended weld size might require a fairly large fillet weld. Also, weld distortion might be an issue due not only to the amount of heat input from placing a large weld, but also to not having a weld on the opposite side to counteract the effects of weld shrinkage. Larger welds, of course, also require more weld passes (see Table 8-12 in the *Manual*) so a single sided-weld may not be the most economical solution.

Of course, there may be situations where, despite all of the considerations above, a single-sided fillet or PJP groove weld may still be the best or even the only available option. Care should be taken when using single-sided welds on skewed single-sided shear connections.

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