If you've ever asked yourself "Why?" about something related to structural steel design or construction, *Modern Steel*'s monthly Steel Interchange is for you! Send your questions or comments to **solutions@aisc.org**.

steel interchange

New Columns over Existing Columns

On a recent project where a new addition is being erected above an existing structure, a survey has determined deviations in the center line of the existing columns from the locations shown in the original design drawings. All of the new steel has been fabricated based on the dimensions shown in the original design drawings. What is the tolerance on the location of the centerline of the new columns relative to the existing columns at the column splice?

Tolerances are provided in the AISC *Code of Standard Practice for Steel Buildings and Bridges* (ANSI/AISC 303) available at **www.aisc.org/standards**. However, the *Code* does not provide tolerances that relate to existing work. It does provide erection tolerances that are sometimes related to the design assumptions inherent in the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360) also available at **www.aisc.org/standards**, and the *Code* tolerances could be used as a guide for evaluating your condition. However, it should be noted that the erection tolerances in the *Code* would still apply to the erection of the new steel. The potential exists that the difference between the assumed and actual geometry may increase erection tolerances beyond what is acceptable in the *Code*.

The detailing of the new structural steel should have been based on an accurate survey of the existing steel, and some allowances should have been made in both the overall design and the details to accommodate errors in the survey. Generally, the survey should be completed by the owner's designated representative for construction (often the general contractor) and provided to the fabricator in a manner consistent with the schedule and the fabricator's bid. Section 1.8.3 of the *Code* states: "Surveying or field dimensioning of an existing structure is not within the scope of work that is provided by either the fabricator or the erector. Such surveying or field dimensioning, which is necessary for the completion of the approval documents and fabrication, shall be performed and furnished to the fabricator in a timely manner so as not to interfere with or delay the work of the fabricator or the erector."

Larry S. Muir, PE

WT Availability

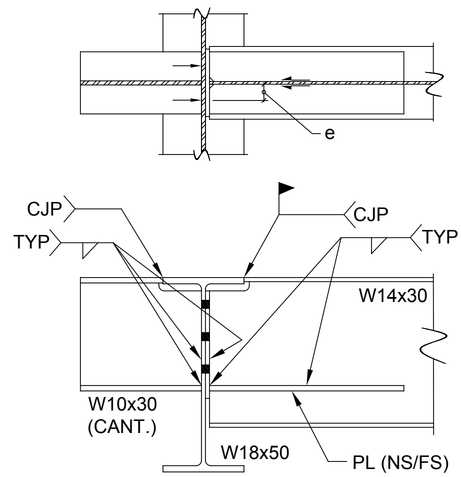
There are WTs listed in the AISC *Steel Construction Manual* (available at www.aisc.org/publications) but there is no availability information related to WTs on the AISC website. How should I determine the availability of WTs?

WTs are typically split from wide-flange shapes. You should refer to the availability of the section from which the WT will be cut to determine availability. For example, check availability of a W16×40 when considering the availability of a WT8×20.

Carlo Lini, PE

Consideration of Small Eccentricities

In calculations, I have seen some engineers neglect the eccentricity (shown as "e" below) in the design of the stiffener welds to the beam web, and other engineers consider this eccentricity. Must this small eccentricity be considered?



Yes, the eccentricity must be considered. However, this consideration may conclude that the effect will be negligible and therefore need not be explicitly addressed in the calculations.

The AISC *Specification* does not address small eccentricities. Ultimately, each engineer must decide whether or not the effects of an eccentricity are negligible based on their own judgment.

Personally, I would start with a balanced free-body diagram before beginning any design, including eccentricity. In this specific case, I would expect the moment to be taken in the long weld. At the same time, I would neglect the eccentricity in the calculations because the weld at the web is long relative to the eccentricity, and its resistance relative to the moment created is based on the square of the weld length. If I needed further justification, I would note that the strength of the weld relative to transverse loading caused by the eccentricity is 50% greater than that relative to the shear.

That said, a better solution may be to replace the W10×30 outrigger with at W14×30 outrigger.

Carlo Lini, PE

Preferred Plate Material

I have been told by several fabricators that their preferred material for plates is ASTM A572 Grade 50 rather than ASTM A36 as currently indicated in Table 2-5 of the *Manual*. Has a change occurred since 2010?

Yes. The February 2015 article "Are You Properly Specifying Material?" (available at **www.modernsteel.com**) states: "The preferred material specification for structural plates is in transition. Use of ASTM A36 (F_y = 36 ksi for plate thickness equal to or less than 8 in., F_y = 32 ksi otherwise; F_u = 58 ksi) is as common as use of ASTM A572 Grade 50 (F_y = 50 ksi, F_u = 65 ksi for plate thickness equal to or less than 4 in.)."

The 15th edition of the *Manual*, available this summer, will show both A36 and A572 Grade 50 as the preferred plate materials up to 4-in. thickness. Material should be specified based on specific requirements for the project and/or local fabricator preference.

Leigh Arber, PE

Bolted Wide-Flange Connections in Special Concentrically Braced Frames

Are bolted gusset-to-beam connections and wide-flange members similar to those shown in Figures 3-4 and 3-5 of Design Guide 29: *Vertical Bracing Connections—Analysis and Design* (available at www.aisc.org/dg) permitted in special concentrically braced frames (SCBF)? All the published examples that I have seen involve welded HSS braces. Is there a reason why bolted wide-flange details are almost nonexistent in literature?

Both bolted gusset-to-beam connections and wide-flange members are permitted for SCBFs. Connections similar to those in Design Guide 29 have been used in SCBFs in practice. As an aside, details similar to Figure 3-6 of the Design Guide can also be used in SCBFs to force the brace to buckle in-plane, thereby eliminating the need to consider out-of-plane inelastic buckling. When this is done, care must be exercised relative to the assumptions made about the end-restraint in each direction to ensure that the brace will buckle in the in-plane direction as intended.

In my experience, hollow structural sections (HSS) are by far the most commonly used brace sections in SCBFs. This is likely because the strong- and weak-axis buckling strengths of the HSS are more equal than for a wide-flange section, thereby reducing the difference between the design strength and expected strength of the brace. However, there are conditions such as very high loads where HSS simply cannot be used. In such cases wide-flange members with bolted brace-to-gusset connections are common.

Some engineers incorrectly conclude from the prevalence of HSS braces in SCBFs that HSS are inherently better suited than wide-flange sections to seismic applications. As stated above, there are economic advantages to the use of HSS braces in SCBFs. However, wide-flange sections also have their advantages. NEHRP Seismic Design Technical Brief No. 8: *Seismic Design of Steel Special Concentrically Braced Frame Systems—A Guide for Practicing Engineers* states: "Wide-flanges and other open sections do not localize the strain as quickly and as severely as rectangular tubes. Hence, wide-flange braces typically provide approximately 25% larger inelastic story drift than rectangular HSS braces prior to brace fracture if all other factors are equal."

AISC currently makes available over 7,000 pages of design guidance including manuals, design examples and design guides. This does not include the hundreds of pages included in the AISC codes and specifications. Even with the vast amount of information that has been produced and distributed, we cannot address every conceivable condition. The absence of a particular configuration is not meant to convey a prohibition. Bolted, brace-to-gusset connections can be and are used in SCBFs. Wide-flange sections can be and are used as braces in SCBFs. Moreover, just to close the loop, bolted HSS brace-to-gusset connections can be and are used in SCBFs.

Another common and similar misconception is related to the dearth of examples of SCBF connections to column webs, which are also not prohibited. SCBF connections to column webs are addressed in the May 2016 Steel Interchange.

Larry S. Muir, PE

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