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steel interchange

Unless specifically stated, all AISC publications mentioned in the questions and/or answers reference the current edition and can be found at ***www.aisc.org/specifications***.

Cover of Design Guide 29

The picture on the cover of AISC's Design Guide 29: *Vertical Bracing Connections—Analysis and Design* conflicts with the advice given in Chapter 3 of the guide. Chapter 3 cautions against the use of plates alone at the brace-to-gusset connection and states: "Small wide-flange braces with this orientation are typically connected to the gussets by WTs or double angles back-to-back on the near and far side of the gusset. Alternatively, single angles on each side of the brace could be employed. If the brace is subjected to compression as well as tension, plates should not be used in place of the WTs or angles." It also states: "Plates can be used to attach the web, and 'claw' angles can be used to attach the flanges. The outstanding angle legs provide for stability."

I have encountered engineers who design brace-to-gusset connections employing plates assuming an effective length factor, *K*, of 0.5 and an unbraced length from the last row of bolts (closest to the work point) to the beam or column flange. This seems like a potentially dangerous practice.

Why are splice plates shown in the cover photo of Design Guide 29?

The short answer is that splice plates are not shown in the cover photo of Design Guide 29.

The splice is actually made using channels on both sides of the gusset and the plate knifed into the HSS. The flanges of the channels provide more out-of-plane strength and stiffness than a plate, though not as much as the WTs or double angles recommended in the guide. I_y for a 1-in.×18-in. plate is 1.5 in.^{4}. I_y for back-to-back MC18×42.7 is 47.6 in.^{4}. This is a significant increase in strength and stiffness. Even with the channels, there is still a small gap between the channels and the HSS. The ends of the gap can likely be considered clamped (fixed) and the gap is quite short. This, combined with relatively compact gusset plates, might make the overall stability of the condition less of a concern than it would be for a chain of lapped plates. However, engineering judgment must be applied to every condition encountered. It is also possible that a stiffener exists at the back side of the plate knifed into the HSS.

Though we do not mention the use of plates alone (other than to generally discourage their use) in Design Guide 29, AISC Design Guide 24: *Hollow Structural Section Connections* does address the design of similar conditions and recommends: K = 1.2, an assumed length equal to the entire length between the end of the brace and the face of the supports, use of geometric properties of the thinner element and consideration of eccentricity where it exists. These seem to be pretty good recommendations and are considerably more conservative than what you report seeing in practice.

As the saying goes, "You shouldn't judge a book by its cover." Engineers also should not look at a condition and judge it based solely on the way it looks. We sometimes get sketches and photos of connections and members with questions like "Is this crazy or what?" "Does this look wrong?" or "Is this okay?" I cannot simply look at any condition and decide whether it is okay or not. Each condition must be judged against its intended function, not some arbitrary measure of what looks "okay." I will admit that I tend not to like brace-to-gusset connections that employ only plates. To me, it looks like someone pushing on a chain. However, with proper consideration and judgment, these conditions can be safely designed, and I have used them. The fact that some figure or photograph in the *Steel Construction Manual* or an AISC Design Guide does or does not look like a condition in the real world should not be the deciding factor in its suitability.

Larry S. Muir, PE

Second-order Effects and Column Design

Are second-order effects to be considered in design for all columns or just columns that are part of a frame?

Second-order effects are increases in the moments and forces on columns that are part of the lateral frame due to lateral deformations caused by the first-order loads. The leaning columns (those that are not part of the lateral frame) add to those increases in the lateral frame because they go along for the ride.

Section C1 of the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360) provides general stability requirements and states: "Stability shall be provided for the structure as a whole and for each of its elements. The effects of all of the following on the stability of the structure and its elements shall be considered:… (b) second-order effects (including P-Δ and P-δ effects);…" The Commentary provides further guidance. It states: "Columns in gravity framing systems

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can be designed as pin-ended columns with K = 1.0, However, the destabilizing effects (P-Δ effects) of the gravity loads on all such columns, and the load transfer from these columns to the lateral force-resisting system, must be accounted for in the design of the lateral force-resisting system."

The second-order effects associated with all columns must be considered in design. However, the structure will "redistribute the story P-Δ effects to the lateral force-resisting elements in that story in proportion to their stiffnesses." The Commentary goes on to state: "In a building that contains columns that contribute little or nothing to the sway stiffness of the story, such columns are referred to as leaning or gravity-only columns. These columns can be designed using K = 1.0, but the lateral force-resisting elements in the story must be designed to support the destabilizing P-Δ effects developed from the loads on these leaning columns. The redistribution of P-Δ effects among columns must be considered in the determination of K and F_e for all the columns in the story for the design of moment frames. The proper K-factor for calculation of P_c in moment frames, accounting for these effects, is denoted in the following by the symbol K2." Note that the *Manual Design Examples* (a free download at **www.aisc.org**) illustrates the design of leaning columns.

Jonathan Tavarez

I have specified ASTM A992 steel for a structural steel frame. The bidders have asked if the connection plates and angles will be A572 Grade 50 steel. Is A572 Grade 50 an acceptable substitution for A992? Are there cost impacts to my requiring A992 for everything?

You should discuss the cost impact of various decisions with the fabricators.
ASTM A992 specifically addresses "rolled structural shapes." ASTM A572 addresses, "shapes, plates, sheet piling and bars." ASTM A6, which is referenced from both A992 and A572, defines plates and shapes. Based on the ASTM specification, A992 plate does not exist. Table 2-5 of the *Manual* indicates that ASTM A36 and ASTM A572 Grade 50 are both preferred materials for plate.

The *Manual* states: "The designation of preferred material specifications is based on consultations with fabricators to identify materials that are commonly used in steel construction and reflects such factors as ready availability, ease of ordering and delivery and pricing. AISC recommends the use of preferred materials in structural steel designs, but the final decision is up to the designer based on project conditions. Other applicable material specifications are as shown in grey shading. The availability of grades other than the preferred material specification should be confirmed prior to their specification."

Angles present a different situation. Angles are shapes and therefore can be made to satisfy ASTM A992. Table 2-4 of the *Manual* lists A36 as the preferred material specification for angles. 50-ksi material is becoming more common in U.S. fabrication. The article "Are You Properly Specifying Materials?" (**www.modernsteel.com**) states: "The preferred material specification for these shapes is in transition. ASTM A36 (F_y = 36 ksi, F_u = 58 ksi) is now only slightly more common than 50-ksi grades like ASTM A529 Grade 50, ASTM A572 Grade 50, or ASTM A992; each of these 50-ksi grades has F_y = 50 ksi and F_u = 65 ksi for these shapes."

There are several grades that are applicable for 50-ksi angles and plates. The common ones are covered in the article mentioned above, and your fabricator can let you know which one is suitable for your project.

Jonathan Tavarez