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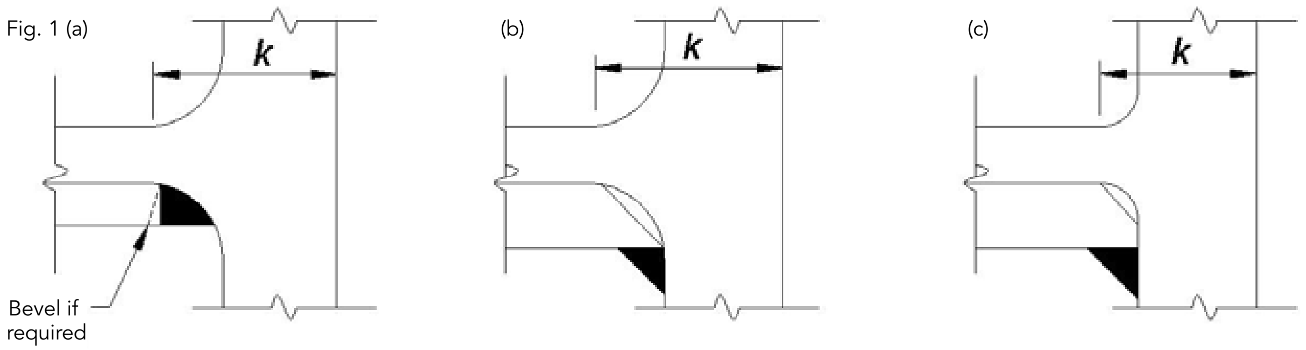

Figure 1. Figure 4-13 of Design Guide 13 (modified).
(a) CJP groove weld detail.
(b) Fillet welded detail with plate bevel equal to plate thickness.
(c) Fillet welded detail with plate bevel less than plate thickness.

Column Web Doubler Plates

Our office has always used Figure 4-13 in AISC Design Guide 13: *Wide-Flange Column Stiffening at Moment Connections* (see Figure 1) to specify welds between web doubler plates and column flanges. However, some of our fabricator clients have expressed a preference for the detail shown in Figure 2. The argument is that only the web needs to be reinforced and that once the fillet weld transfers the load to the k-region radius, this thicker part of the web should be enough to carry the load to the flanges.

We prefer to base our designs on details that have a history of use and that are shown in AISC documents. We'd like to know whether there has been an update to Design Guide 13 (available at www.aisc.org/dg) that includes the details shown in Figure 1. If not, could you please provide an explanation on whether the proposed detail is an acceptable alternative?

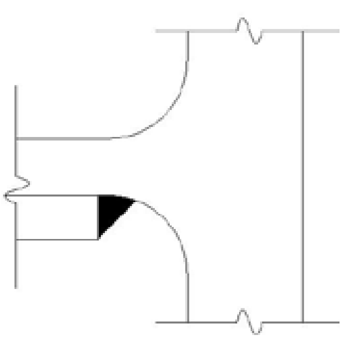

Figure 2. Proposed detail.

Personally, I would avoid using this type of detail. Lack of access may make welding difficult if not impossible. The column flange may prevent adequate access to the root of the fillet weld. As indicated in Figure 1(a), the plate may need to be beveled to provide welding access, which adds further complexity. Fitting and welding the doubler plate will also be complicated. In order to ensure that all of the considerations described are satisfied, the doubler plate will have to be placed with a good deal of precision and/or the size of the elements will have to be increased to account for tolerances in fitting.

The proposed detail is not a common approach in my experience, and while it may be possible to justify using this detail, the load transfer relies on the increased thickness resulting from the fillet between the web and flange of the column, which would be very difficult to quantify. The design and fabrication may be more complex and the benefits, in terms of economy, may be less than might be perceived at first glance.

The best way to handle member reinforcement is to size the member such that reinforcing isn't necessary. Sizing the members to eliminate reinforcing simplifies the contractual, fabrication and design processes and generally results in lower overall cost for the project. Often, fabrication costs associated with reinforcing members far exceed any savings associated with the member weight. Least weight does *not* mean least cost. Eliminating reinforcing also eliminates the contractual complications that can arise when member reinforcing requirements are not clear in the contract documents. It also obviously eliminates controversies related to design models like the one you're asking about.

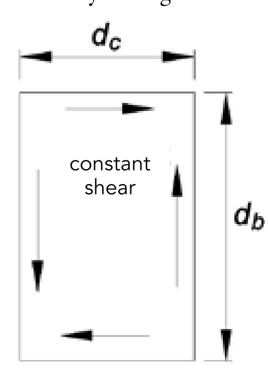

Figure 3. Idealized panel zone.

It may be possible to justify the detail shown in Figure 2, but I am not sure it is a practical detail. The dimension of the fillet between the flange and the web of wide-flange members is not well defined. ASTM A6 states: "Radii of fillets and toes of shape profiles vary with individual manufacturers…" Without knowing the dimension of the fillet, it would be challenging to evaluate the strength at any given point. It would also be difficult to ensure that the condition is detailed and fabricated consistent with the design intent.

One periodic mistake is not recognizing that the shear stress is constant around the panel zone (see Figure 3). Engineers commonly recognize that the flange force delivered by the moment connection causes horizontal shear across the column web. This shear stress per inch of column web thickness is roughly equal to $(M/d_b)/d_c$ or $M/(d_b\, d_c)$. However, the vertical couple needed to maintain the panel zone's equilibrium is sometimes overlooked. This shear stress per inch of column web thickness is roughly equal to $(M/d_c)/d_b$ or $M/(d_b\, d_c)$, which is equal to the shear in the orthogonal direction, indicating a constant shear at the perimeter of the panel zone.

Since the shear stress is constant, the simplest way to size the doubler plate is to maintain a constant thickness around the perimeter. This approach is reflected above in Figure 1(a). Though this detail can require more welding than other details, it is preferred by some fabricators due to its simplicity in terms of design, detailing and fabrication. Variations in the weld size are easily accommodated, exact placement of the doubler plate is not critical and prepping the doubler plate edge isn't required for thinner doubler plates.

Another common alternative is to maintain a constant minimum doubler plate thickness using a detail similar to Figure 1(c). The doubler plate must be prepped to avoid the fillet, but the minimum required doubler plate thickness is maintained throughout the panel zone. This results in a doubler plate that is thicker than necessary except at the bevels, but that is relatively easy to design and detail. Because of mill tolerances on such dimensions as depth and flange tilt, the doubler plate may have to be fabricated smaller than the dimensions of the panel zone, thus leaving a gap around the perimeter. This gap will have to be accounted for when welding and may cause an increase in the weld size, potentially compromising some of the perceived reduction in weld size shown in Figure 1(a).

A third approach recognized in Design Guide 13 is shown in Figure 1(b). Though the shear stress remains constant, the doubler plate tapers to a point. In this case, the combined available strength of the doubler plate and the fillet weld is kept greater than the required strength. The web shear strength is commonly assumed constant. There is no allowance made for increased thickness and strength near the web-to-flange fillet. This solution is closer to optimal, but it is more complex relative to design, detailing and fabrication than the other options. In our experience, it also tends to be misunderstood and misapplied with some frequency. And as with Figure 1(c), a gap will have to be accounted for when welding and may cause an increase in the weld size, potentially compromising some of the perceived reduction in weld size.

Other approaches are possible. Having reviewed the more common details and outlining some of the considerations involved, we can return to the proposed detail. Theoretically, a model similar to that used to justify the detail shown on Figure 1 (c) could also be applied to the proposed detail in Figure 1. In this case, however, the total available strength of the fillet weld and the web-to-flange fillet must exceed the required strength. This is where things get tricky and the model may become impractical.

Regarding the dimensions of the web-to-flange fillet, the AISC *Steel Construction Manual* (**www.aisc.org/manual**) states: "Because of the variation in fillet sizes used in shape production, the decimal value, k_{des}, is conservatively presented based on the smallest fillet used in production, and the fractional value, k_{det}, is conservatively presented based on the largest fillet used in production." Based on this, it can be assumed that the full doubler plate thickness must be provided at least to the k_{des} dimension. Beyond this, the combination of the fillet weld and the web-to-flange fillet must provide the required strength. Though the *Manual* provides us with a couple of dimensions for the web-to-flange fillet, we do not know the shape of the fillet; it does not necessarily form a quarter circle. What we do know is that that the fillet weld is tapering (losing thickness) faster than the beam-to-flange fillet is gaining thickness. This means that the size of the fillet weld, and therefore the doubler plate, will have to be increased to satisfy the required strength.

Unlike the details shown in Figure 1, there is no direct transfer of load from the beam into the column's web-to-flange fillet or the fillet weld. This may also further complicate the design.

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