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steel interchange

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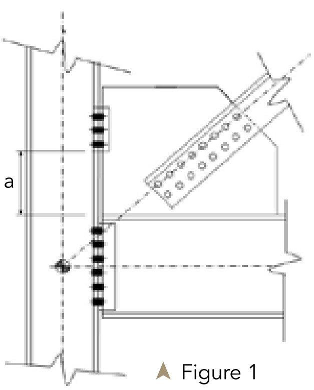


Figure 1

Gusset-to-Column Connection and Line-of-Action Buckling

We are reviewing a connection similar to what is shown in Figure 1. We have concerns about the stability of the gusset. Is there a limit on the length of the free edge of the gusset at the gusset-to-column portion of a vertical brace connection (labeled *a* in Figure 1)?

There is no explicit limit on the free length of the gusset shown in Figure 1; this arrangement is not prohibited. We have seen a significant number of similar details recently. Presumably, the gusset-to-column connection is set high on the gusset in an attempt to reduce the couple, H_c, that resists the moment caused by the vertical component of the brace force, which is delivered eccentric to the column centerline. While this practice will tend to reduce the couple, it also may raise other concerns. The checks used to determine the available buckling strength of the gusset must consider the restraint provided at the connection.

AISC Design Guide 29: *Vertical Bracing Connections—Analysis and Design* and the Commentary to the *Specification for Structural Steel Buildings* (ANSI/AISC 360) both recommend procedures developed by Bo Dowswell (see "Effective Length Factors for Gusset Plate Buckling" in the second quarter 2006 issue of AISC's *Engineering Journal*) and illustrated in the Design Guide. Historically, the effective length factor, K, has often been assumed to be 0.50. Both of these models assume significant restraint at the gusset-to-column and gusset-to-beam interfaces. Table C-A-7.1 of the Commentary to the *Specification* indicates that 0.5 corresponds to the theoretical effective length factor, K, when both ends of the column are fixed against both rotation and translation.

When the gusset-to-column connection is short relative to the height of the gusset and/or located away from the corner of the gusset, the typical assumptions related to effective length should be reconsidered. I am not aware of a published guideline, but in practice I tended to limit the free length to about 6 in. I am not aware of a document that provides guidance related to an appropriate effective length factor for conditions with large unsupported edges near the column. Dowswell recommends an effective length factor equal to 0.7 for the single-brace condition, and this case might be viewed as a lower-bound estimate.

To avoid controversy and for my own peace of mind, I would either add bolts or increase the bolt spacing at the gusset-to-column connection such that more of the gusset is supported and therefore more consistent with the typical buckling checks. Ultimately, you must use your own judgment.

Larry S. Muir, PE

Alternative Design Procedures

In Part 10 of the AISC *Steel Construction Manual*, the discussion of the extended configuration of the single-plate shear connection states: "The design procedure for extended single-plate shear connections permits the column to be designed for an axial force without eccentricity. In some cases, economy may be gained by considering alternative design procedures that allow the transfer of some moment into the column…"

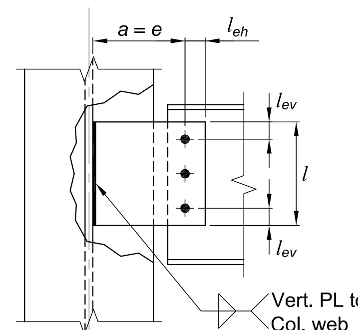
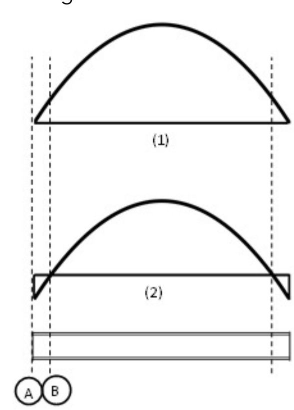


Figure 2

Based on this statement, we have assumed a pin located half the width of the column flange plus 3 in. from the column's centerline indicated by the red target superimposed on the figure from the *Manual* (shown here as Figure 2). We have designed both the beam and the column consistent with this assumption.

The beams have been designed assuming a span between the assumed pins (span = $L_{\text{cen-to-cen}} - 2(b_f/2 + 3$ in.). The columns have been designed as beam-columns accounting for the eccentricity (shown as *a* and *e* in Figure 2).

Figure 3



We have also delegated connection design—Option 3 in Section 3.1.1 of the *Code of Standard Practice for Steel Buildings and Bridges* (ANSI/AISC 303)—specifying that extended single-plate shear connections were to be used at all beam-to-column web conditions. The fabricator has designed the extended single-plate shear connections using the procedure in Part 10 of the *Manual*. We are concerned that the beams, columns and connections are designed using incompatible models. Is our concern valid? What could be done to avoid this problem in the future?

I believe your concern may be valid. The *Manual* does not present of free-body diagram or directly describe the model assumed. However, it does indirectly describe the assumed distribution of moment in the connection. It states: "Determine the bolt group required… with eccentricity, *e*, where *e* is defined as the distance from the support to the centroid of the bolt group." It also states: "The design procedure for extended single-plate shear connections permits the column to be designed for an axial force without eccentricity." In addition, near the beginning of Part 10, the assumed (or typical) model is also described: "…the ends of members with simple shear connections are normally assumed to be free to rotate under load. While simple shear connections do actually possess some rotational restraint (see curve A in Figure 10-1), this small amount can be neglected and the connection idealized as completely flexible. The simple shear connections shown in this *Manual* are suitable to accommodate the end rotations required per AISC *Specification* Section J1.2." In my experience, this is the model commonly assumed in the design of beam-to-column simple shear connections.

The issue here seems to be that the beams were not designed as simply supported beams, which would result in the moment diagram in Figure 3 (1). Instead, they were designed as beams, which though not exactly typical fixed-end beams, were subject to some flexural restraint at their ends resulting in the moment diagram in Figure3 (2).

When connection design is delegated, the owner's designated representative for design—generally the engineer of record (EOR)—must provide any restrictions on the types of connections that are permitted and data concerning the loads sufficient to allow the selection, completion or design of the connection details. Though the physical end of the supplied wide-flange section falls somewhere between points A and B in Figure 3, the end of the beam (the horizontal member supporting the floor) in the model extends to point A, where a moment exists in the bottom diagram (2). This moment, along with the shear, should have been reported in the contract documents.

Though simply reporting the moment may not have been sufficient to convey the design intent, it would have likely prompted questions from the engineer designing the connections. A better approach would have been to provide more detailed information about your intent—perhaps including a free-body diagram of the connection plate in addition to the end reactions—since you were employing an alternative design procedure. In my experience, this sort of detailed design guidance is often provided when delegating the design of the steel elements of embed connections to concrete.

Though there was a miscommunication here, it is good to see that the problem has been caught. The *Code* requires the fabricator to "submit in a timely manner representative samples of the required substantiating connection information" to the engineer. It requires the engineer to "confirm in writing in a timely manner that these representative samples are consistent with the requirements in the contract documents" or to "advise what modifications are required to bring the representative samples into compliance with the requirements in the contract documents." This is required to catch such issues so that they can be resolved between the EOR and the delegated connection design engineer.

I assume that you have adopted the model based on the idea that delivering a moment to the column will result in a more economical structure or will help you satisfy project-specific requirements. There are arguments that can be made to support this position. By reducing the demand on the beam, it may be possible to provide lighter or shallower beams. The model will also tend to reduce the number of bolts that must be field installed. However, in order to deliver the moment to the column, you will likely also need to provide stiffeners at the top and bottom of the single plate (rendering it no longer a single-plate connection). The benefits described above may be offset to some extent by the cost associated with the stiffeners. Providing the stiffeners increases the fabrication costs but may reduce the erection costs.

Larry S. Muir, PE

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