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steel interchange

Extended Gages

Part 10 of the AISC *Manual* contains a discussion entitled "Eccentric Effect of Extended Gages." The discussion seems to indicate that the engineer can assume the moment due to the eccentricity exists either at the bolt group or at the weld. I have two questions.

1. **If the moment is assumed at the bolt group, can the supporting member and the welds be designed to resist only the vertical shear?**
2. **If the moment is assumed at the weld, then the supporting girder must be designed to resist torsion. An open section, like a wide-flange girder, is torsionally quite weak, and designing it to resist torsions will result in a heavy, uneconomical section. Isn't it prudent to always design the connection, rather the support, to resist the moment?**

Though the second question specifically mentions a beam/girder as the support, the answers below consider conditions with both supporting beams and columns.

1. Yes, but ductility must also be considered. The design procedures for single-plate shear connections assume the model described in the *Manual* discussion: eccentricity on the bolts and shear on the weld. However, the procedure then ensures that the bolts and welds are stronger than the plate to accommodate end rotations. Section B3.6a of the *Specification* states: "A simple connection shall have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure." The design procedures for single-plate shear connections implicitly target an end rotation of about 0.03 radians and have been shown to accommodate such rotations through physical tests. An end rotation of 0.03 radians is relatively large and probably unrealistically high for a serviceable beam. You do not necessarily have to satisfy the design procedures shown in the *Manual*, but you must consider ductility and the effects of end rotation.
2. No, not always. The argument makes sense for connections to open-section beams that have framing to one side only, but may not make sense for other girder conditions (or connection to columns, where it is a moment and not a torsion).

When framing to a column flange, it is probably more economical to upsize the column to account for the eccentricity than it would be to account for the eccentricity in the connection. This is especially true for vertical brace connections. The additional cost of material in the column will usually be less expensive than the alternative of more and larger bolts and larger welds, since labor costs are typically higher than material costs. However, in my experience, it is rare for the moment to be resisted by the support, even when it is clear that doing so will result in a more economical design. There are a number of reasons for this:

- Columns other than those in moment frames have historically been designed as axial members, and engineers are sometimes reluctant to break with tradition.
- The design models are simpler if the column is assumed to resist only axial loads. However, this reason may be fading, as engineers routinely use computer analysis and design programs. Many of these programs can (and may, by default) include some eccentricity in the columns. In such cases, the moment is often accounted for twice.
- Owners, architects and general contractors tend to judge an engineer's performance based on indirect and inaccurate measures like weight per square foot. In effect, the individuals charged most directly with maintaining the economy of the project sometimes incentivize practices that increase the overall structural costs. Decades of efforts on the part of AISC and others have failed to change this.

For a girder support, an open section is much less efficient in torsion, so the weight penalty would be greater. Another consideration is that the connection would somehow have to transfer the moment into the girder. Since the web of the beam is also inefficient in weak-axis flexure, it might be beneficial to attach the connection to the flanges of the girder. This would involve additional labor and materials.

When connecting to a column web, the gage may be extended to get the bolted connection beyond the flanges to make erection more efficient. It might seem that designing the column for the resulting moment is inherently more efficient than adding bolts. But again, the moment must somehow get through the inherently inefficient web to the flanges of the column. Like at the girder, this will typically involve additional labor and materials.

A further consideration is that the additional labor and materials related to transferring the moment from the web of the support to the flanges will be performed in the shop, whereas additional bolts will be installed in the field.

So in my opinion:

- If you are framing to a column flange, in most cases it is probably more efficient to upsize the column than to resist the eccentricity at the bolts.

- ➤ If you are framing to a column web, in most cases I suspect it still may be more efficient to upsize the column than to resist the eccentricity at the bolts. However, a little more consideration is probably warranted and it is likely to be much more conditional.
- ➤ If you are framing to a girder, I suspect it is more efficient to resist the eccentricity at the bolts than to upsize the girder.
- ➤ Note that ductility needs to be considered regardless of the model assumed.

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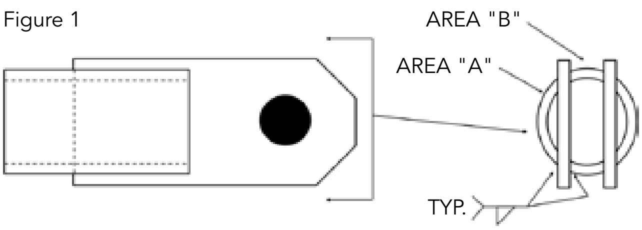
Double-Slotted HSS Connections

Two plates are slotted into and welded to a round HSS tension member, as shown in Figure 1. Since I am connecting more of the section than is assumed in Case 5 of Table D3.1 in the AISC *Specification*, I assume it would be conservative to check tensile rupture using an effective area based on that case. Is this correct?

Figure 1



Probably not.

This condition falls outside the scope of Table D3.1; attaching more of the section does not mean that more of the section is effective—i.e., Case 5 is not conservative. However, you are not merely seeking to satisfy an equation, or a set of equations, in a vacuum, but rather are trying to design a real structure to resist actual loads.

Case 5 is pretty simple: Each pair of welds delivers half of the load to half of the section. Your configuration is more complex and deserves more consideration. If you simply applied the checks one would typically apply to a connection similar to Case 5, you might end up using inconsistent models to design the various elements in the connection. This will be illustrated for the condition under consideration.

The following assumptions are made to simplify the discussion:

1. The length of the welds is assumed to be greater than 1.3 times the diameter of the HSS.
2. The full net area is effective. This is consistent with your assertion that Case 5 of Table D3.1 can be conservatively applied.
3. The required strength is assumed equal to the available net section strength of the HSS.
4. The net area is assumed to be 4.75 sq. in. and the tensile strength of the HSS is 58 ksi. This is done simply to so that we can work with numbers instead of variables or percentages.

From this, the available net section strength is (0.75)(58 ksi)(4.75 in.^{2}) = 207 kips. A typical approach would be to assume each of the eight fillet welds transfers one-eighth of the load, which is 25.9 kips.

Load is delivered by two welds to AREA "A". Therefore AREA "A" must resist one-quarter of the load, which is 51.8 kips. It can be seen that AREA "A" is roughly one-third of the net area, 4.75/3 = 1.58 in.^{2} The rupture strength of this section is (0.75)(58 ksi)(1.58 in.^{2}) = 68.7 kips, which is greater than the 51.8 kips assumed to be delivered by the welds.

However, an analysis of AREA "B" turns out differently. The load is also delivered by two welds to AREA "B". Therefore AREA "B" must also resist one-quarter of the load, 51.8 kips. It can be seen that AREA "B" is roughly one-sixth of the net area, 4.75/6 = 0.792 in.^{2}. The rupture strength of this section is (0.75)(58 ksi)(0.792 in.^{2}) = 34.5 kips, which is less than the 51.8 kips assumed to be delivered by the welds.

The assumption that the HSS is uniformly loaded is inconsistent with the assumption that welds are uniformly loaded. A uniform distribution among the welds optimizes the welds. A uniform distribution within the HSS optimizes the HSS. Both cannot be optimized simultaneously with the given geometry.

There are probably a number of ways to analyze this condition, but ultimately you must choose a model and then check each element based on the loads from the chosen model.

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