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Corrosion Resistance and Shear Connections

The April 2015 *Modern Steel* article "Considering Corrosion" by Steven A. Sebastian (www.modernsteel.com) includes several options for shear connections in Figure 4. Among these is a field-welded, single-plate shear connection. The recommended design procedure in the AISC *Manual* is: "The plate must be welded to the support on both sides of the plate and bolted to the supported member." Is a field-welded, single-plate shear connection acceptable? Is rotational ductility a concern?

There is often not a perfect engineering solution for a given situation. Often the pros and cons of a condition must be weighed to determine the best—or at least an acceptable—solution. For typical beam-end conditions, engineers should probably adhere to the design procedures in Part 10 of the *Manual*. However, special circumstances may sometimes dictate other approaches. The author presents just such a case, where corrosion resistance is a primary concern. The trade-off is acknowledged in the article when it is stated that "The option at the top [of Figure 4] is simple and flexible and has sufficient strength for most applications. However, it is the most susceptible to corrosion…"

If having a simple and flexible connection is the primary consideration, then a connection from Part 10 of the *Manual* should be chosen. As corrosion resistance becomes a bigger consideration, the engineer may have to move down the list of the connections perhaps making trade-offs in economy and behavior along the way.

There is nothing in the AISC *Specification* that would prohibit the use of an all-welded connection. The *Specification* requirements are provided in Sections B3.6a and J1.2. Both of these sections address the need for rotational ductility. The AISC *Manual* design procedures for shear tabs are intended to address the need to accommodate simple beam end rotations. A de facto standard for this rotation has become 0.03 radians, which is a very large demand. If the end rotation is significantly less than this, then it makes sense that the detailing recommendations in the *Manual* could be relaxed. Conditions where minimal end rotation are expected might include: lightly loaded beams, short beams, beams governed by deflection (not strength), struts primarily resisting axial loads and beams with concentrated loads applied close to the end.

The design procedure for conventional single-plate connections in Part 10 of the *Manual* assumes all of the end rotation is accommodated through plowing of the bolts, which obviously would not occur if the connection were welded. There are however other mechanisms that could be used to accommodate the end rotation. For instance, for the case of a connection to a beam web with no beam present on the other side, as is shown in Figure 4 of the article, the simple beam end rotation could be accommodated through weak-axis flexure of the web of the support.

Another mechanism that can be used to accommodate simple beam end rotations is flexure of the plate. This method is used primarily for extended tabs but there is no reason it could not be applied to a conventional single plate shear connection as well.

When applying strong-axis flexural yielding to an all-welded conventional tab, I tend to have concerns about the relatively small distance that might exist between the welds at the supported beam and the welds at the face of the support. For this reason I have typically tried to use only the vertical weld at the end of the plate or if horizontal top and bottom welds are used hold them back somewhat to provide a larger length over which the tab plate can yield. Holding back the weld would reduce the effectiveness of the detail relative to corrosion, so again the engineer must weigh the options carefully. The all-around weld may be okay in some instances where the rotational demand is low or other mechanisms are able to accommodate the rotation.

It should be noted that there are many tradeoffs inherent in the article. Opting for a field-welded detail over a field-bolted detail will likely incur additional costs relative to the erection, but as is stated, "The economic impact of shutdowns, repairs and maintenance may be of greater concern than the lowest initial capital cost." Another issue is the use of seal welds. Seal welds may conflict with Specification requirements and will certainly affect the flow of loads through the structure. Section 3.13.3 of AISC Design Guide 21: *Welded Connections—A Primer for Engineers* (a free download for members from **www.aisc.org/dg**) provides a good discussion on this topic.

Obviously, not all of these considerations can be fully addressed in a single article, so the article must be taken in context. Its primary focus is corrosion protection. As always, the engineer must consider a multitude of factors during the design process.

Larry S. Muir, P.E.

Composite Collectors

I am using equations from AISC *Specification* Section I3 to determine the spacing of shear studs to transfer collector forces in a concrete-filled steel deck diaphragm. ACI 318-11 Appendix D recommends using ϕ = 0.75 for cast-in headed studs with supplemental reinforcing, but I do not see anything specified in *Specification* Section I.3. Can the ACI ϕ-factor be used to design the shear stud in accordance with *Specification* Section I.3?

No. The provisions in ACI should not be intermixed with the AISC provisions for composite members except where the AISC provisions specifically reference ACI. The AISC provisions have evolved over the last couple decades based on research, which is not reflected in the ACI provisions.

AISC *Specification* Section I.3 does not specify a ϕ for headed studs. The studs are simply a component of the composite beam system, and the equations have been developed such that ϕ equals 0.9 for flexural bending of the composite beam.

For other composite members (not beams), the appropriate ϕ for the shear connectors is defined in Section I8.3.

I do not know whether you are designing the collector beam as a composite beam or a non-composite beam relative to the gravity load combination, and that can impact how you design the collector. The December 2008 article "Under Foot" (**www.modernsteel.com**) discusses collector beams in composite slabs that you may want to review, particularly if your collector beam is also a composite beam. Since this article was published, there has been some additional investigation into shear connector behavior, which pertains to non-composite beams used as collectors. When loads are applied to the floor system after the slab concrete has hardened, the floor beams will deflect. When a beam deflects, shear forces are induced at the interface between the steel section and the concrete section as the slab will try to "slip" along this plane. The shear connectors restrain this slip behavior and transfer force between the steel and concrete sections. If your beam is designed non-composite, you still need to consider these shear loads on the studs due to the slip which may reduce the strength of the shear connectors available for the lateral load condition.

For a simple span beam, the "slip" demand will be greatest at the beam ends. Therefore, it follows that studs located near the beam ends will be subject to higher shear forces due to the beam deflection than studs located at mid-span. If you are designing your beam as non-composite and only adding a few studs for horizontal load transfer, it may be best to locate the studs near mid-span where the slip demand is least.

Shear connectors on non-composite beams do not know they are not supposed to behave like shear connectors for composite beams. If you are going to distribute studs along the entire length of the beam, then you should ensure you have enough studs installed for the beam to act as a composite member or there is a possibility the studs at the beam ends subject to the greatest slip demand will be overloaded and fracture. This is why the above article recommends installing enough studs to develop a minimum of 25% of the members composite beam capacity.

Susan Burmeister, P.E.

Connection Design Forces

The 9th Edition of the AISC *Manual* required connections to be designed for one-half the total uniform design load (UDL) shown in the allowable uniform load tables, if loads were not provided in the design documents. Is this still a requirement?

No. The AISC *Specification* does not contain this requirement. The AISC *Code of Standard Practice*, which generally governs trade practices for the fabrication and erection of structural steel, addresses the reporting of loads for connection design.

Section 3.1.2 of the *Code* requires the engineer of record (EOR) to provide design loads when connections are to be selected or completed by the fabricator. Loads should not be assumed. If the contract documents do not provide sufficient information to determine the loads, then an RFI should be sent to the EOR requesting this information.

AISC has never recommended the use of one-half UDL. The use of actual reactions has always been the preference. Older editions of the *Manual* stated: "For economical connections, the beam reactions should be shown on the contract drawings." They went on to say, "If these reactions are not shown, connections must be selected to support one-half the total uniform load capacity… The effects of any concentrated loads must be taken into account." There were several problems with this language. First, the *Manual* is not adopted into law through the building code and therefore cannot introduce requirements; it can only provide guidance related to requirements in the *Specification* or the *Code*. Second, though the use of one-half UDL is generally going to be conservative, it is not fool-proof. Third, stating that a detailer/fabricator (the parties presumably being addressed) must account for concentrated loads does not really resolve an issue where no loads are provided. In essence the detailer/fabricator would have to make engineering decisions about when the concentrated loads existed and when they were significant. However, only an engineer can make engineering decisions.

The current language in the *Code* is a much better approach. The engineer is clearly responsible for providing the loads. The engineer can still choose to use one-half UDL criteria, but this is not the preferred or optimal method. Further discussion concerning the problems associated with the use of one-half UDL is provided on page 2-30 of the 14th Edition *Manual*.

Carlo Lini

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