

If you've ever asked yourself "Why?" about something related to structural steel design or construction, *Modern Steel's* monthly Steel Interchange is for you! Send your questions or comments to [solutions@aisc.org](mailto:solutions@aisc.org).

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### Bolt Entering Direction

According to the specification for our project, the bolt head should be placed on the outside of the flange. Is this an AISC requirement?

No, it is not an AISC requirement. There are no requirements for orientation of the bolt in the AISC *Code of Standard Practice*, the AISC *Specification* or the RCSC *Specification*. Note, however, the scope statement in Section 1.1 in the *Code of Standard Practice*, which states:

This *Code* sets forth criteria for the trade practices involved in steel buildings, bridges and other structures, where other structures are defined as those structures designed, fabricated and erected in a manner similar to buildings, with building-like vertical and lateral load resisting elements. In the absence of specific instructions to the contrary in the contract documents, the trade practices that are defined in this *Code* shall govern the fabrication and erection of structural steel.

Thus, the specific instructions in the contract documents govern. Since the contract documents state that the bolt heads should be placed on the outside of the flange, this is a requirement per the contract documents for this particular project.

Carlo Lini

### Limiting the Number of Field Splices

The erector on our project is insisting that conditions indicated as field-welded splices in the contract documents should be shop welded for economy. Can we shop weld these splices?

Section 6.7.4 of the AISC *Code of Standard Practice* states:

Unless otherwise specified in the contract documents, and subject to the approved shop and erection drawings, the fabricator shall limit the number of field splices to that consistent with minimum project cost.

The key phrase here is: "Unless otherwise specified in the contract documents..." The contract documents call for field splices, and those requirements govern the project and must be adhered to unless a change to the contract is agreed to by the parties.

Toward that end, you could submit a request to the engineer of record (EOR) to modify the connection such that this would result in reduced field welding. Or, if you think the field weld symbol may have been a mistake, you could submit an RFI to clarify this.

Ultimately, however, you cannot change the condition shown in the contract documents without approval from the EOR.

Carlo Lini

### Partial Thread Engagement

Please explain the basis of this statement in Section 2.11.3 of the 2nd Edition of AISC Design Guide 1: "A conservative estimate of the resulting nut strength can be made based on the percentage of threads engaged, as long as at least half of the threads in the nut are engaged."

It's a statement based upon judgment, but I can add some science as well.

At full thread engagement, we know the threads will not strip before the bolt fractures through the net tensile area. Of course there is some "factor of safety" on this, so that somewhat less than what is defined as full engagement is required, but we define full engagement as existing when the nut is at least flush with the end of the bolt. If full engagement does not exist, then there is the possibility that the full tensile strength of the bolt cannot be developed.

When the nut is stronger than the bolt—the normal case—the strength of the threads relative to stripping is usually calculated as:

$$F_s A_{ts}$$

where

$F_s$  = shear strength of the bolt threads

$A_{ts}$  = area of the threads at the stripping failure plane

$A_{ts}$  can be calculated as:  $\pi n L_e D_{min} [1/(2n) + 0.577(D_{min} - E_{max})]$

where

$D_{min}$  = minimum major diameter of the bolt threads

$L_e$  = length of thread engagement

$E_{max}$  = maximum pitch diameter of the nut threads  $n$  is the threads per inch

As it turns out, fasteners are configured such that  $\pi n L_e D_{min} [1/(2n) + 0.577(D_{min} - E_{max})] > 1.0$ . So the strength relative to stripping always increases faster than the increase in thread engagement. So if we simply assume that the strength relative to stripping always increases at the same rate as the increase in thread engagement, we will have a conservative estimate of the strength.

Some caution must be exercised when applying these equations, since there are other factors—such as friction, the relative nut-to-bolt strength, etc.—that can affect the stripping strength of the bolt. Therefore, it is advisable to err on the side of caution, as has been done in the guidance presented in the design guide.

It should also be noted that though exceptions relative to full thread engagement are sometimes made for anchor rods, it is generally not considered appropriate to make such exceptions for bolted connections.

Larry S. Muir, P.E.

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## Eccentricity on Coped Beams

I am designing connections for a project in which the EOR has delegated the design of connections using Option 3 in the AISC *Code of Standard Practice*. When checking coped beams, the definition of "e" on page 9-6 of the AISC *Steel Construction Manual* allows the point of inflection to be assumed some distance from the face of the supporting member. I have opted to assume that the point of inflection is located at the bolt line in the beam end rather than at the face of the support. Is this acceptable?

The information you noted in the AISC *Manual* explicitly allows the use of an eccentricity less than the distance from the face of the support to the end of the cope. However, in my experience the inflection point is almost universally assumed to be at the face of the supporting member. If you were to assume instead that the inflection point is at the bolt line, then you would have to ensure that the other elements in the connection could resist the moments consistent with your chosen model. The change would also potentially add moment to the supporting members, and these members would have to be designed for the additional moment.

It is generally safe to assume the inflection point is located at the face of the supporting member because this assumption is consistent with common design practices. Some structural analysis and design programs assume an inflection point away from the face of the support, but this assumption would have to be confirmed or the adequacy of the members otherwise determined before the assumed inflection point could be moved away from the face of the support.

A further consideration is that rotational ductility per *Specification* Sections B3.6a and J1.2 must be satisfied. Moving the assumed inflection point could produce additional demands on the connection that would increase the size of some of the elements and make satisfying the rotational ductility requirements difficult. The move you are contemplating, from the face of the support to the bolt line, should not cause any problems in this regard, but other locations, such as assuming an inflection point at the end of the cope, could present issues.

The main points here are:

1. You cannot unilaterally change the assumption about the location of the inflection point. The EOR must be consulted.
2. Consistent assumptions must be used throughout the design and all design considerations must be properly addressed.

Larry S. Muir, P.E.

## Splices in Bent Edge Plate

At splices in bent edge plate (pour-stop) the inspector is insisting that the vertical legs of pour-stop be welded with butt welds. Are we required to weld the splices?

There is no specific AISC provision that requires this, but what is shown in the contract documents? Are there any details or notes on the structural drawings or specifications that call for the vertical legs of the pour-stops to be butt welded?

On projects in which I am the EOR, I do not typically provide any requirements or details for how to splice the bent plates and I have seen the vertical legs welded, the vertical legs butted together snugly with no weld and the vertical legs adjacent with a gap that gets covered with duct tape. Personally, I have no preference and will accept any of these approaches for a typical bent plate pour-stop splice.

In general, independent third-party inspectors should be inspecting the structure for conformance with the contract documents and not interpreting or modifying the contract documents. Under this assumption, if there are no details on how to treat the bent plate splice in the structural drawings or specifications, the inspector should not be commenting, and I would recommend the contractor submit an RFI to the EOR asking them to confirm your proposed detail is acceptable. On some projects, the inspector will be acting more as an agent of the EOR—but again, if there's no detail in the contract documents, you should still be able to seek written confirmation of the EOR's intent.

Susan Burmeister, P.E.

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