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steel interchange

Camber and Specific Instructions to the Contrary

The specification for a project requires camber to be measured in the field in the stressed condition and not in the fabricator's shop in the unstressed condition, as indicated in Section 6.4.4 of the AISC *Code of Standard Practice* (ANSI/AISC 303), available at www.aisc.org/standards. The specification then states that the fabricator will be responsible for any repairs required to bring nonconforming beams into compliance with the specified camber.

After the project was awarded, the fabricator issued a request for information (RFI) requesting the unstressed camber required so that the beam when installed would settle to the stressed camber noted in the contract documents. The RFI quoted the Commentary from Section 6.4.4 of the *Code* to explain why the camber measurement cannot be measured in the field in the stressed condition. In his response, the structural engineer of record stated that, per the contract, this determination must be made by the contractor.

I have several questions:

1. **Since the Commentary to Section 6.4.4 states that there is no way to inspect beam camber after the beam is received in the field (due to numerous factors), is it not the intent of Sections 3.1(e) and 3.1.5 that the magnitude of camber specified in the structural design documents be that which is measurable for the purposes of fabrication?**
2. **Does AISC permit the engineer to deviate from the *Code* in this manner?**
3. **Can the fabricator be held responsible for achieving a condition over which the fabricator may have little control?**
4. **Is there any practical method of determining the unstressed camber that must be provided to ensure that the stressed condition is within tolerance?**

We cannot arbitrate or address contractual issues. This issue should have been addressed during contract negotiations. If there are no contractual exceptions to the camber requirement, then the fabricator must satisfy the requirement. However, Section 7.13.13 of the *Code* requires the owner's representative for construction to verify plumbness, elevation and alignment prior to the placement of other trade materials. We are assuming that the notation "stressed condition" refers to the beam as erected prior to placement of finishes. Since you have agreed to measure the camber in the field, the camber should be measured before other materials are applied to the beam by other trades. In the event that the owner's survey identifies beam(s) not meeting the required camber, repair work may be the fabricator's responsibility.

Another wrinkle is that even if the fabricator did take exception to the camber requirement in the bid—and this was agreed to contractually—the contract would likely be between the fabricator and their client. The issue would then have to be addressed relative to the contract between the fabricator's client, the owner and the engineer of record. To answer your questions:

1. Yes, this is the intent. However, the engineer has chosen not to conform to the intent of the *Code*, and you have chosen to contractually accept this deviation. AISC recommends that specifiers adhere to the *Code* unless there is a very good reason not to, but we have no authority to govern the contracts parties choose to enter into. The February 2017 article "Specific Instructions to the Contrary" (available at **www.modernsteel.com**) provides further information.
2. Yes. Section 1.1 of the *Code* states: "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." The above-mentioned article provides a good discussion related to the proper use of specific instructions to the contrary.
3. Probably. This is likely a legal question, and we cannot provide legal advice. However, I believe it is common for contracts to allocate risk among the parties, even when the parties have limited control over the risks.

   In the case of camber, the Commentary to the *Code* lists several factors that are largely beyond any party's complete control. The structural engineer of record should in most cases have the most reliable information related to many of these items—and likely the best ability to account for them. However, there is no party that can fully control all of the potential effects, and some party must therefore assume the associated risk.
4. No. This would require information and coordination with the designer and contractor. As this is uncommon, the approach taken in the *Code* is what we recommend.

Larry S. Muir, PE, and, Patrick J. Fortney, SE

Reinforcing an Existing End-Plate Moment Connection

The ⅜-in. end plate of an existing connection (configured similar to an end-plate moment connection) is not adequate for an increase in design load, based on checks from commercially available connection design software. The connection has been defined as an end-plate moment connection in the software model. The connection transfers modest moments and shears but also significant

axial load. ASTM F3125 Grade A325-N bolts are provided. Can a square washer be used at each of the bolts to increase the thickness to meet the required thickness determined by the software? Are there better means of reinforcing this connection?

The answer to your first question is no. Adding a square washer at each bolt will not satisfy the assumptions likely made in the calculations. We cannot comment on what your software may be doing, but locally reinforcing the plate would not satisfy the models presented in either AISC Design Guide 4: *Extended End-Plate Moment Connections Seismic and Wind Applications* or Design Guide 16: *Flush and Extended Multiple-Row Moment End-Plate Connections* (both are free downloads for AISC members at **www.aisc.org/dg**), which probably form the basis of the checks used by your software.

Adding the washers may have some effect on the strength of the plate, but it will likely be small and difficult to quantify. Theoretically, one could use the plate washers to modify the yield lines used in Design Guides 4 and 16, which would result in an increase in strength if the strength of the connection is controlled by the plate yield lines. The guides also provide references to additional information on the models used. I am not aware of anyone that has taken this approach and cannot provide any definitive guidance on how to do so. You will have to rely on your own judgment.

Here are some other observations, in case you still wish to pursue this option:

1. Even if you used a reinforcing plate over the entire connection, you still may not be correctly interpreting the condition. The models in the design guides assume a solid plate. Your software probably makes the same assumption. Therefore the increased strength predicted is most likely based on the square of the total thickness. If you do not adequately connect the reinforcing plate to the original plate, then the strength increase would result from the sum of the squares of the two thicknesses, not the square of the sum of the thicknesses—a big difference.
2. The ability to form yield lines at the edges of the reinforcing plate will depend on several factors, including the distance the reinforcing is extended beyond the joint and/or that way in which is attached to the existing plate. This will further complicate the design.

Other approaches are possible and might provide a better solution.

If you have assumed thin plate behavior (with prying), as described in the design guides, then the apparent deficiency relative to the plate thickness might be addressed by changes to the bolts.

The first and most economical option to explore simply involves a change in the design assumptions—potentially no physical change to the condition at all. If you can confirm that the shear planes do not intersect the bolt threads in the existing condition, then you could take advantage of this fact to increase the bolt strength. It is typically assumed that only the bolts on the compression side of a moment end-plate connection resist shear. However, for your condition, where there is a modest moment and significant axial load, there may be no portion of the connection in compression. Therefore, the bolts will be subjected to combined tension and shear. Though you have stated that the shear loads are small, the increase from threads included (N) to threads excluded (X) might be enough to accommodate the increase in design load.

A second option that may avoid costly fieldwork involves reexamining the assumed distribution of force among the bolts. Design Guide 16 suggests a model for conditions where both axial loads and moments are applied. However, it only seems to address conditions where the moment is the dominate load, unlike your condition. Many models are possible and you might find one that will make your existing condition acceptable—again avoiding costly fieldwork and more uncertain structural models such as the square washer approach.

A second option would be to use a stronger bolt. Replacing the existing ASTM F3125 Grade A325 bolts with Grade A490 bolts would increase both the shear and tensile design strength of the bolts. Again, using the threads excluded shear strength, where appropriate, will provide an even greater strength increase. Note that your proposed solution of adding square washers would involve, at the very least, removing the nut at each bolt and installing the plate washer, so replacing the bolts may provide a much greater increase in strength with only slightly more cost.

Bottom line, you may want to exhaust fixes that can be accomplished with a pencil and a calculator before mobilizing crews and equipment in the field. Effectively addressing existing conditions often requires a deeper understanding of the design assumptions and the behavior of the systems involved. Off-the-shelf software and "canned" design procedures may provide a good starting point to evaluate the strength of connections, but such approaches may not lead to an optimal solution.

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