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steel interchange

Eccentricity on Columns

Are there any formal recommendations concerning the inclusion of eccentric moment in a steel column due to the physical distance between the beam end and column centerline?

The decision to account for the eccentricity or neglect it is one that you have to make based on your engineering judgment. Ioannides ("Minimum Eccentricity for Simple Columns," *ASCE Structures Congress Proceedings*, Volume 1, 1995) suggests that even for a column that is loaded on one side only, the restraint a connection provides to the column will help mitigate the eccentric effects in normal framing configurations.

There are some common parallels in design where we neglect eccentricity. The AISC *Steel Construction Manual* states that for standard or short-slotted holes, eccentricity on the beam side of double angle connections may be neglected for gages (distance from the face of the support to the centerline of a single vertical bolt row, shown as dimension a in Figure 10-4[a] in the *Manual*) not exceeding 3 in. While you are permitted to neglect this eccentricity, there will still be a resulting moment that will exist somewhere in the system. Some of the moment will go to the column and some to the beam, based on the stiffness of the elements. The reasons we neglect the eccentricity are largely historical: The basics of bolted joint design evolved before analysis capabilities had progressed sufficiently to account for it. However, there are some technical justifications. First, any assumption about where the moment will exist will be wrong, since the system will distribute the moment throughout the system based on stiffness. The usual eccentricity is relatively small, and its effects become arguably negligible when distributed within the system—even if the effects might be significant when assumed to be concentrated at an individual element. There also are other influences like the fact that the bolt strengths provided in the AISC *Specification* (available for free at **www.aisc.org/2010spec**) have been reduced to account for uneven loadings that occur in primarily end-loaded connections. These reductions also help to account for some eccentricity without explicit consideration of the moment by the designer.

A final thought: Check the settings in your software to see if your columns are already being designed for eccentricity automatically. Some software programs account for an assumed amount of connection eccentricity as a default when sizing the columns.

Carlo Lini

Minimum Weld Sizes

We have a project where a 1-in.-thick angle is welded to a 1-in.-thick plate. A ¼-in. weld has sufficient strength, but a 5⁄16-in. fillet weld was specified to meet AISC *Specification* Table J2.4's minimum requirements. A ¼-in. weld was completed in the field. Is it possible to come back and augment the existing weld, or does the weld need to be removed?

Adding additional weld would not address the issue, which is related to having a high enough heat input to prevent cracking. However, you may not need to repair the weld either. Duane Miller addressed this issue at the 2013 NASCC: The Steel Conference in his presentation "Welding Questions Answered" (view session N78b at at **www.aisc.org/2013nascconline**). Fast-forward to the 19:00 minute mark, and you'll see a progression of three options: one based upon low-hydrogen process solutions, one based upon evaluation of heat input, carbon equivalent and cooling rate and one based upon removal and replacement. I believe the information from this presentation should help you address this issue.

Carlo Lini

Slenderness Limits on Columns

Does Equation E3-3 in the AISC *Specification* apply even when *KL/r* is greater than 200? Do the provisions of Section E5 for single angles apply when *KL/r* is greater than 200?

According to Equation E3-3, the nominal critical stress is $F_{cr} = 0.877F_e$. F_e is the theoretically derived elastic buckling stress according to Equation E3-4. It was originally derived by Euler in a slightly different form. The coefficient 0.877 is an empirical reduction factor that is based on a statistical analysis of the geometric imperfections. The User Note in Section E2 recommends that "the effective slenderness ratio *KL/r* preferably should not exceed 200." This is a recommendation, not a requirement. Equation E3-3 is valid for $KL/r > 200$. The Commentary discusses this further.

Because the equations in Sections E5(a) and E5(b) were developed empirically, the stated limits of $KL/r \leq 200$ must be met. If the effective *KL/r* is greater than 200, concentrically loaded angles can be designed according to Section E3, E4, or E7, as appropriate. If the angle is loaded eccentrically by connecting one leg to a gusset plate, the eccentricity can be addressed using the equations in Chapter H. AISC *Manual* Table 4-12 was developed using Section E3, E4, E7 and Chapters F and H.

Bo Dowswell, P.E., Ph.D.

Minimum Loads for Splices

We designed a beam to support a floor. The contractor has asked to put a bolted splice in the beam to simplify erection. We provided a shear and moment from the design loads at the splice. Since the loads at this location are small, the splice they have designed seems too light. Is there a minimum splice required, such as to design the moment splice to 75% beam capacity, regardless of the actual loads?

No, there are no minimum criteria in AISC standards for beam splices. Generally, the AISC *Specification* provides requirements relative to design and detailing based on the forces determined by the engineer. As such, the *Specification* typically does not provide minimums. Section J6 simply states that "splices in cross sections of plate girders and beams shall develop the strength required by the forces at the point of the splice."

That said, engineering judgment must be exercised. The splice must provide both sufficient strength and stiffness.

Susan Burmeister, P.E.

Delegated Connection Design

When performing connection design that has been delegated by the engineer of record (EOR), we sometimes receive contract documents that do not appear to comply with the building code. We seem to have two choices: We could strictly follow the requirements as shown in the contract documents, or we could redesign the connections to meet the building code. Doing the latter may be stepping on the EOR's toes and would be detrimental to our client, the fabricator, as the details would likely be more expensive than those shown in the contract documents. Can you provide any advice?

There are really two issues: your responsibility as a licensed engineer and your responsibility to your client as it relates to your and their contractual obligations.

As a connection design engineer, you must satisfy the intent of the engineer of record as it is conveyed in the contract documents. The EOR ultimately has responsibility over the project. Compliance and interpretation of the building code—including the building code compliance of the connection design criteria specified in the contract documents—is within the EOR's scope. Unless I have strong reasons to believe the EOR is doing something unsafe, I ultimately would leave these decisions to them.

This is not to say that I would remain silent. If I saw something that appears to be unsafe or conflicts with my understanding of the design intent, I would question it. This is consistent with Section 3.3 of the AISC *Code of Standard Practice*, which requires that the fabricator promptly notify the owner's designated representative for construction (usually the general contractor) of discrepancies. Note the fabricator and delegated connection engineer need not review the documents for discrepancies, but must notify the owner's representative of discrepancies that have been recognized. As described in Section 3.3, it is the owner's designated representative for design (the EOR) who resolves the discrepancy.

The *Code* also provides explicit requirements related to delegated connection design for option 3 of Section 3.1.2. It should be noted that the connection design engineer is required to provide substantiating connection information, which the EOR then reviews for conformance with the contract documents. These requirements define a process intended to ensure both engineers are on the same page. This reflects the relationship described above, where the connection design engineer strives to satisfy the EOR's intent. The contract documents communicate the EOR's intent to the contractors. If the EOR's intent changes or the original contract documents prove insufficient to properly convey the intent, then the contract documents must be revised. The *Code* addresses this situation as well.

Section 9.3 addresses revisions to the contract documents and indicates that contract price and schedule shall be adjusted in accordance with Sections 9.4 and 9.5 when the contract is revised.

As the connection design engineer, you cannot unilaterally change the contract by introducing requirements. If you feel discrepancies exist, you must notify the owner's designated representatives. If your arguments are persuasive, then the owner's designated representatives will revise the contract to address your concerns. This revision will prompt the parties, including your client, the fabricator, to assess the impact of the change. The fabricator should not absorb the costs associated with requirements that were not clearly shown in the contract bid package.

Larry S. Muir, P.E.

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