

## AISC Live Webinar Series: *Connection Design for Moment Frames and Braced Frames*

### Selected questions with answers from the speakers

March 27, 2020

#### **Session 1: Moment Connections, Part 1**

1. **Question:** What does it mean "When load is away from end of the member" in Slide 51?

**Brad Davis:** In Section J10.5 of the Specification, according to the next-to-last paragraph, Equation J10-8 is reduced by 50% when the opposing loads are applied within  $d/2$  of the end of the member. Slide 51 is indicating the other situation -- the loads are applied  $d/2$  or farther from the end of the member.

2. **Q:** Are slip critical bolts required for the beam web shear connection in moment connections?

**BD:** No. See Example II.B-1 of Volume I of the *Manual Companion* for an example with bearing connections rather than slip-critical. <https://www.aisc.org/publications/steel-construction-manual-resources/#37583>.

3. **Q:** Can a doubler plate be installed only on one side of a web region, or do they always have to be on both sides?

**BD:** Yes, one doubler plate can be used rather than two. See DG13 Figure 4-11 and surrounding text for examples.

*AISC tip: Note that it can be economical to design heavier columns to avoid doubler plates altogether. Carol Drucker discusses this in a 2018 webinar at the following link.*

*<https://www.aisc.org/education/continuingeducation/education-archives/stiffeners-doublers-and-web-plates-oh-my/>*

4. **Q:** In Slide 51, do the  $C_v$ s have to be directly across? For example, what if they were offset by a few inches? Is there a limit to when Web Compression Buckling would still apply?

**BD:** That's a matter of engineering judgment. If they were offset by only a few inches, then I would check this limit state. To my knowledge, there is no guideline for what counts as close enough. It seems to me that a distance of about half the column depth makes sense for this.

#### **Session 2: Moment Connections, Part 2**

5. **Q:** Where does the  $\gamma$  factor come from (e.g., Slide 59)?

**BD:** It comes from the research that was performed mostly at Virginia Tech. It was found that an adjustment of this magnitude is required to make sure the connection qualifies as an FR moment connection.

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6. **Q:** Do we ever use minimum area for tension design on bolts? AISC has a minimum net tensile area for bolts and I'm tempted to use that area when doing tension on bolts. (Table 7-17)

**BD:** No, the reduction factor for going from the gross area to the net tensile area is already included in  $F_{nt}$  (given in Table J3.2). See Design Guide 17, Page 24, left-hand column of text.

*AISC tip: See also AISC 360-16 Commentary Section J3.6 (page 16.1-436).*

7. **Q:** On slide 50, how is  $Q_{max}$  calculated for a tee that does not meet the minimum no-prying thickness  $t_{np}$ ?

**BD:** For regular tee connections, it is typically not necessary to compute the prying force. When it is needed, it can be computed using the Manual Equation 9-24.

8. **Q:** For end plate moment connections, are we required to resist all the shear with the compression bolts?

**BD:** No, that is not a requirement. It's just the recommended method in DG16. Note that design guides are not adopted by the Building Code -- the Specification is. The allocation of forces in a connection such as this is a matter of engineering judgment and structural analysis. One could consider all bolts to resist the shear and in that case, the tension-side bolts would be subjected to combined tension and shear.

9. **Q:** On Slide 36 where does the value  $T_b$  come from?

**BD:**  $T_b$  is the minimum pretension from the Specification Table J3.1.

### **Session 3: Introduction to Seismic Connections**

10. **Q:** Can you allow the use of erection bolts in SCBF brace connections?

**Matthew Eatherton:** The last paragraph of the commentary to Section D2.2 of AISC 341-16 states that the Seismic Provisions do not prohibit the use of erection bolts on a field-welded connection... but the welds would need to be designed to resist the entire anticipated force in that element. Two considerations: 1) the erection bolts should not be positioned in a way that they affect the effective net area, or interfere with / interrupt reinforcement for the net section, and 2) I think erection bolts would be exempt from D2.2(d) and don't need to be fully pretensioned (and probably shouldn't be fully pretensioned if passing through a hollow section like HSS).

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11. **Q:** I work in the petrochemical (industrial) sector. For a structure in a high seismic zone, to follow a SCWB (strong column weak beam) philosophy, can I expect columns (generally shallower than the beam) to be significantly heavier (by weight) than the beams? For instance, in a typical pipe rack, I generally expect the columns (shallower section) to be about the same weight as the beams. So, in a high seismic zone, I should estimate and plan on my columns being much heavier.

**ME:** At an interior connection with beams on both sides and column extending through the connection, yes the column will end up being heavier than the beams. I have heard from a practicing engineer who designs SMF on a regular basis that he uses  $\Sigma Z_{xc} / \Sigma Z_{xb} > 1.5$  as a good starting point in a design and then does the individual checks including SCWB.

12. **Q:** Is the 6" doubler plate extension shown on Slide 58 a requirement of the Seismic Provisions?

**ME:** AISC 341-16 Section E3.6e.3 (starts on page 9.1-46) states that if there are no continuity plates, the doubler plate should extend 6 in. past the outside of the beam flange. If the connection has continuity plates, then the doubler plate can either extend the same 6 in., or the doubler plate can terminate at the inside of the continuity plate.

13. **Q:** Slide 58: Why does the backing have to be removed on the bottom of the connection and can remain on the top of the connection?

**ME:** The backing bar at the bottom of the connection is on the outside of the flange is thus at the extreme fiber, meaning that the stresses and strains are the worst at the location of the backing bar. At the top flange, the backing bar is on the inside of the flange and is thus subject to smaller stresses and strains. Furthermore, it has been shown that concrete slabs that act compositely with the beam act to shift the neutral axis up, thus creating larger stress and strain demands at the bottom flange compared to the top flange.

*AISC tip: The article at the following link also has interesting discussion on this topic.*

<https://www.aisc.org/globalassets/modern-steel/archives/2017/02/takeamomenttoconsiderthismomentconnection.pdf>

### **Session 4: Bracing Connections**

14. **Q:** Slide 33: Can this connection carry moment as well, since has pretty large connecting end plate with lots of bolts?

**BD:** Yes, it can certainly transmit some moment from the brace and beam to the column. The level of this moment will typically be low because the vast majority of the braced frame stiffness

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is due to axial stiffness. I think the following is an interesting exercise. Create a 2-3 story braced frame in a frame analysis program. Add hinges to make the frame behave as a classical truss, and run the analysis. Now, take away all hinges and run it again. The axial loads typically decrease a little and some small moments will occur, but they are typically not significant. Of course, there could be exceptions to this.

15. **Q:** On Slide 68, what is  $k_{des}$  (1.38) and where do you get it?

**BD:** It is the fillet dimension from Table 1-1 of the Manual. There are two: a decimal one used for design and a fractional one used for detailing. The design value is smaller, which is almost always more severe for design, as was the case here, so it was selected.

*AISC tip: There is also discussion on the difference between  $k_{des}$  and  $k_{det}$  on page 1-3 of the AISC Steel Construction Manual, 15<sup>th</sup> Edition.*

16. **Q:** Slide 75: Why is  $K=0.5$ ? It appears to be unconservative. One side fixed and other free results in  $K=2$  or min. 1.5.

**BD:**  $K = 0.5$  follows the examples in DG29, such as on Page 51. See also Page 97 of the Gross 1990 Engineering Journal paper. (<https://www.aisc.org/Experimental-Study-of-Gusseted-Connections>)

*AISC tip: See also the alternative discussion in the following 2006 paper by Dowsell. <https://www.aisc.org/Effective-Length-Factors-for-Gusset-Plate-Buckling>*

17. **Q:** Where does the 25% increase come from (Slide 60)?

**BD:** The 2004 Engineering Journal paper by Hewitt and Thornton, "Rationale Behind and Proper Application of the Ductility Factor..." (<https://www.aisc.org/Rationale-Behind-and-Proper-Application-of-the-Ductility-Factor-for-Bracing-Connections-PDF>)

*AISC tip: See also the discussion on page 13-11 of the AISC Steel Construction Manual, 15<sup>th</sup> Edition.*

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