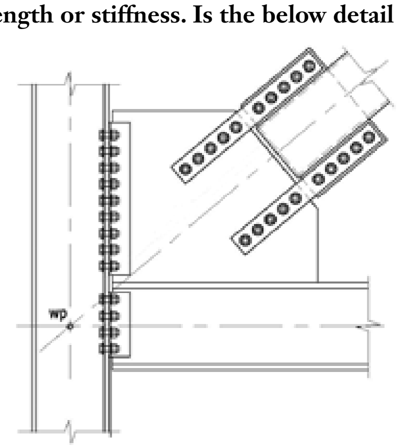
If you've ever asked yourself "Why?" about something related to structural steel design or construction, *Modern Steel*'s monthly Steel Interchange is for you! Send your questions or comments to **solutions@aisc.org**.

steel interchange

Gusset Buckling

A detail has been proposed for a brace-to-gusset connection. Instead of using four claw angles, like in Figure 3-4 of AISC Design Guide 29, it uses four "claw plates." We are concerned that the claw plates provide little out-of-plane strength or stiffness. Is the below detail acceptable? If so, can the buckling length of the gusset as shown in Appendix C of Design Guide 29, or is the buckling length a greater dimension due to the lack of an outstanding leg in the connecting element?



There is nothing in the AISC *Specification* that would prohibit the detail described. However, the authors of the Design Guide discourage the use of such details. Section 3.1 states: "If the brace is subjected to compression as well as tension, plates should not be used in place of the WTs or angles." Section 3.2 states: "Plates can be used to attach the web, and 'claw' angles can be used to attach the flanges. The outstanding angle legs provide for stability."

The gusset plate will likely buckle in a sway mode and can be modelled as a column along the work-line of the brace from the connection of the gusset at the beam and column to the end of the brace. This is similar to Figure 5-5 in AISC Design Guide 24, only without the eccentricity.

Since this condition is not addressed in any of the AISC documents, you will have to exercise your own judgment. It seems reasonable to use an effective length factor, K, of 1.0 for the non-compact corner gussets shown in Appendix C of Design Guide 29. For other gusset plate shapes, an K = 1.2 may be more appropriate.

Another design consideration with this type of connection is how to define the radius of gyration for the equivalent column. Some research is available that recommends averaging the radii of gyration at each end of the equivalent column.

The simplest solution may be to provide claw angles in lieu of the lap plates, which would provide brace continuity, and the gusset buckling strength could be calculated using the traditional Whitmore buckling method shown in AISC Design Guide 29.

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

Moment Connections to Unstiffened Column Webs

A beam requires a moment connection to a column web without a back-up beam. I have used an end-plate moment connection similar to the connections shown in AISC Design Guide 4. Since the Design Guide does not address the strength of column webs, I have determined the strength of the column based on the model presented in the *Engineering Journal* article "Yield Line Analysis of a Web Connection in Direct Tension" (Kapp, 1974). A colleague believes the strength of the column web should be determined by modelling the web as a simply supported beam. Who is correct?

I think the two of you may be arguing the wrong point.

The ultimate strength will be more closely approximated by the yield line approach described by Kapp in the article you mentioned. Because the yield line approach neglects catenary (membrane) action, it is still likely to give you a conservative estimate relative to strength. The beam model will be even more conservative.

For rotational stiffness, however, I think you may be missing the forest for the tress. The detail you describe may not be a good choice if the intent is to provide a fully restrained moment connection. The web of the column will likely have a good bit of flexibility. It is unlikely, unless the column is very stout or the moment is very low, that the connection will have "have sufficient strength and stiffness to maintain the angle between the connected members at the strength limit states" as required in Section B3.6b for fully restrained moment connections. Even if the intent is not to provide a fully-restrained moment connection, I am not sure how you would determine "the force-deformation response characteristics of the connection" or insure that the component elements of the connection "have sufficient strength, stiffness and deformation capacity at the strength limit states" as required in Section B3.6b for partially restrained moment connections.

It should also be noted that Section C2.1.(1) states: "The analysis shall consider flexural, shear and axial member deformations, and all other component and connection deformations that contribute to displacements of the structure."

While I am sure that the vast majority of structures are designed with no explicit checks relative to Section B3.6 or C2.1.(1), the requirements must still be considered. An engineer must exercise judgment based on knowledge and experience. In a typical end-plate moment connection framing to a column flange, the compression force is transferred through bearing, which provides a good deal of stiffness. On the tension side, the check is based on a yield-line approach, similar to Kapp, but such connections have been tested.

I think your proposed connection probably deserves a little more attention than a more typical connection. Before determining the strength, you might want to first establish whether or not this can even be considered a moment connection and whether it is fully-restrained. Obviously no connection is truly fixed. Fortunately the Commentary provides guidance and states: "If $K_S L/EI \geq 20$, it is acceptable to consider the connection to be fully restrained (in other words, able to maintain the angles between members)." Though B3.6b places requirements on the connection relative to "strength limit states," it should be noted that the Commentary criterion is based on the behavior under service loads.

It may not be necessary to determine precisely the moment-rotation behavior of the connection in order to classify the connection. You might opt to begin with simple, conservative models, such as the beam model for which deflection (and therefore, connection rotation) can be readily determined. If the behavior from these models satisfies $K_S L/EI \geq 20$, then you might deem the connection to be fully restrained. If the stiffness falls well short, then you might decide this configuration is a dead end and opt for something more traditional.

Structural steel is a nice material to design in. It conforms well to many of our basic design assumptions. It is inherently ductile, relatively homogeneous and isotropic, and there is generally a relationship between strength and stiffness. When something looks right, it generally is. When something looks wrong, it may not be wrong but we need to pay attention. When our designs conform to what has always been done, we can relax a little and get away with some degree of plugging and chugging. However, when we encounter or propose something unusual it deserves a closer look. As Albert Einstein said, "Everything should be made as simple as possible, but not simpler."

Larry S. Muir, P.E.

Levelling a Composite Floor

A composite floor was constructed without the specified cambers in the beams. There are excessive deflections in the floor, and we need to find a way to bring it back to level. We are proposing to remove a center strip of concrete about 3-ft wide along the mid-span of the beams, jack the beams into position from below and then place dowels and cast high-strength, low-shrinkage concrete in the created gap. Does this sound like a reasonable approach?

This does not sound like a reasonable approach. I don't think it will work, and I'd be surprised if it is more cost-effective than the more traditional approach of installing a self-leveling compound (and reinforcing the beams for the additional weight, if required). The process you describe involves a number of trades. Since I imagine there will be debate over who bears the burden of the cost for this repair, it is probably best to go with a simpler approach that involves the fewer parties in the solution.

Some further things for you to consider:

1. I do not believe the jacking of the beam will give you the result you are after. When a beam is cambered in the shop, a permanent, inelastic deformation is induced into the beam. In contrast, since the upward deflection created by the jacking force is an elastic deformation, as soon as the jacking load is removed, the associated upward deflection would disappear and the beam would rebound back to its original shape. The downward deflections from the gravity loads would then be additive to the rebounded beam deflections. It's basic superposition. Since we are talking about relatively flat angles of curvature in the beam, I do not think the new concrete strip would "lock-in" the upward displacement like you might see in a structure with enough curvature to develop arch action.
2. Jacking the beam in the field will induce stresses due to rotations at the connections, which have presumably been tightened down. This may or may not be an issue, but should be considered.

Susan Burmeister, P.E.

The complete collection of Steel Interchange questions and answers is available online. Find questions and answers related to just about any topic by using our full-text search capability. Visit Steel Interchange online at **www.modernsteel.com**.

Larry Muir is director of technical assistance at AISC. Bo Dowswell and Susan Burmeister are consultants to AISC.

Steel Interchange is a forum to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Opinions and suggestions are welcome on any subject covered in this magazine.

If you have a question or problem that your fellow readers might help you solve, please forward it to us. At the same time, feel free to respond to any of the questions that you have read here. Contact Steel Interchange via AISC's Steel Solutions Center:

1 E Wacker Dr., Ste. 700, Chicago, IL 60601
tel: 866.ASK.AISC • fax: 312.803.4709
solutions@aisc.org