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steel interchange

Eccentricity in Vertical Brace Connections Connected to Column Webs

I have several vertical brace connections connecting to column webs. Some of the columns have stiffener plates for moment connections. The erector has indicated that it will be difficult and costly to use double-angle connections for the beam-to-column and gusset-to-column connections, so we have elected to use extended single-plate connections so that the beam can stop short of the column flanges and be erected more easily. The extended plates for the beam and gusset to column connections are separate plates, not one continuous plate running through both the gusset and the beam.

We have designed these connections using the uniform force method, with e_c taken as half the column web thickness. V_c, H_c, V_b and H_b have been calculated per Equations 13-2 through 13-6 of the AISC *Manual of Steel Construction*. The extended single-plate at the gusset-to-column connection has been designed for V_c and H_c, and the extended single-plate at the beam-to-column connection has been designed for V_b and H_c. Both of the shear plates have been designed for the eccentricity from the face of the column web to the centroid of the bolt group. Must the extended single-plate connections be continuous between the beam-to-column and gusset-to-column interfaces in order to transfer the H_c? In order to avoid the need for a continuous connection, should double angles be used? Also, does AISC define a standard connection type and can the extended single-plate option be rejected as a non-standard connection?

If you consider the eccentricity of half of the column web, to balance the free-body diagrams for the extended single-plates you will have to transfer the H_c between the two plates somehow. Either you will have to connect them to each other or you will have to pass the H_c into and out of the column web. It is common to take e_c equal to zero and simply neglect the eccentricity of half the web. Since the calculation of H_c is determined only from statics and engineering judgment relative to the value of e_c, changing the connection type from an extended single-plate to a double angle will have no effect. If, based on your own engineering judgment, the eccentricity due to the thickness of the column web must be taken into account and the connection must therefore be continuous, then this applies to equally to extended single-plate and double angle connections.

Though several beam-end connection types are included in Part 10 of the *Manual*, the inclusion of these connections is not intended to restrict the use of other configurations. In this case, however, both alternatives—the double angles and the extended single-plate connections—are included in Part 10 of the *Manual*.

You might also want to look through AISC Design Guide 29: *Vertical Bracing Connections—Analysis and Design* (a free download for members from **www.aisc.org/designguides**) for further information. In particular, Appendix A includes a design example that uses an alternative location for ec to achieve a more optimal design.

Larry S. Muir, P.E.

Concentrated Loads on S-shapes

The paper "Flange Bending in Single Curvature" in the second quarter 2013 AISC *Engineering Journal* provides a means to determine the strength of a flange of a wide-flange beam subjected to concentrated loads. Is there a similar paper that addresses S-beams used as a monorail?

The paper you mention includes a reference to the CMAA *Specifications for Top Running and Under Running Single Girder Electric Traveling Cranes Utilizing Under Running Trolley Hoist* (Publication No. 74). That publication discusses flange bending due to wheel loads on members with tapered flanges (S-shapes). Equations 30 through 34 explicitly apply to tapered flanges; however, all of the equations developed in the *Engineering Journal* paper can be conservatively used with tapered flanges because the flange thickness values listed in Table 1-3 of the 14th Edition *Steel Construction Manual* are the average flange thickness. At the point of maximum moment (at the flange-to-web intersection) the flange is thicker than the average value.

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

Pre-tensioning ASTM A490 Bolts with ASTM A325 Pretension Values

ASTM A325 bolts were specified but A490 bolts were supplied. The connections are slip-critical. Can we pre-tension the bolts to the levels specified in Table J3.1 for the ASTM A325 bolts originally specified instead of the requirements for the ASTM A490 bolts that were supplied?

No. The pretension values provided in Table J3.1 are about 70% of the tensile strength of the bolts. These pre-tension values will cause yielding in the bolts. Once the bolt has yielded, the stress-elongation curve flattens and there is relatively little change in pre-tension as the elongation changes. In other words, there is greater confidence that the proper pretension has been provided when the bolt is pre-tensioned to this level. If you install the ASTM A490 bolts to the ASTM A325 levels, the bolts will likely still be on the steep vertical portion of the curve, and this will result in uncertainty about the actual clamping force achieved.

Larry S. Muir, P.E.

Length Tolerance on Members

Section 6.4.1 of the AISC *Code of Standard Practice* defines fabrication tolerances for the lengths of members. For a beam with double-angle end connections, does this tolerance apply to the length of the wide-flange section itself or only to the length of the fabricated piece including the end connections?

The tolerance shown in Section 6.4.1, related to your condition, applies to the final fabricated piece, including the connections. The length is measured from end connection to end connection.

Carlo Lini, P.E.

Gaps in End-Plate Moment Connections

We have several end-plate moment connections where small gaps exist—although the end-plates and the column flanges are in contact. We believe these gaps are due to warping of the end plates during welding. Will these gaps result in bending in the bolts due to the shear at the faying surface—and if so, how do we account for this bending?

The RCSC *Specification* requires that all bolted connections be installed at least snug-tight. A snug-tight joint is defined as: "a joint in which the bolts have been installed in accordance with Section 8.1. The snug tightened condition is the tightness that is attained with a few impacts of an impact wrench or the full effort of an ironworker using an ordinary spud wrench to bring the plies into firm contact." Firm contact is defined as: "the condition that exists on a faying surface when the plies are solidly seated against each other, but not necessarily in continuous contact." The definition is not precise, but it does indicate that the gaps are to be intermittent and likely small.

As you have implied, the gaps will result in bending in the bolts. But is the bending significant enough to be accounted for in the design? Section J5 of the AISC *Specification* addresses the use of fills. Fills, like gaps, will cause bending in the bolts. However, Section J5 allows bolts to be designed for their full strength with no reduction due to bending when the fill thickness is equal to or less than ¼ in. In an end-plate connection (or any snug-tight joint) the area of non-continuous contact in a joint that has achieved firm contact will not normally have gaps exceeding ¼ in. (if they do, something else is likely wrong). From all of the above, it is reasonable to deduce that bending need not be considered in the bolts in a properly erected end-plate moment connection.

In order to achieve a better fit, AISC Design Guide 16: *Flush and Extended Multiple-Row Moment End-Plate Connections* (a free download for members from **www.aisc.org/designguides**) suggests: "Stitch bolts are sometimes used between the tension and compression flange end-plate bolts, especially in deep connections. The purpose of these bolts is to reduce plate separation caused by welding distortions. Because stitch bolts are located near the center of gravity of the member, the contribution to connection strength is small and is neglected."

Larry S. Muir, P.E.

Use of Alternative C_b to Account for Load Height

Section 5.2 of the SSRC *Guide to Stability Design Criteria for Metal Structures*, specifically Equations 5-5 and 5-6, addresses the use of C_b values to account for application of load other than at the shear center of the beam. Although this approach is not addressed in the AISC *Specification*, it is mentioned in Commentary Section F1. I have been advised that since load location is not addressed in the AISC *Specification*, the use of a higher value of C_b to account for bottom flange loading is prohibited by the *Specification*. Is it acceptable to calculate beam flexural strength for bottom flange loading of wide-flange beams using a higher C_b value than those indicated in Chapter F?

Yes. The last two sentences of 2010 AISC *Specification* Section A1 allow "alternative methods of analysis" for limit states covered in the *Specification* and also "tests or analysis" for limit states not covered. Therefore, if the methods are acceptable to the authority having jurisdiction and, in the engineer's judgment, are based on sound engineering principles, then the intent of the AISC *Specification* has been satisfied.

The engineer is expected to account for the reduction in lateral-torsional buckling (LTB) strength when the load acts above the level of the shear center. The engineer can also account for the beneficial effect of loading below the shear center. Ultimately, you must use your own judgment to determine what is appropriate for your situation. In practice, I have personally used the SSRC equations to account for the effects of load height.

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

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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:

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