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Shear Lag

AISC 360 Table D3.1 defines $\bar{X}$ as the "connection eccentricity." Considering a single-angle connected on one leg only, is the $\bar{X}$ referred to in Table D3.1 always the same as the member property $\bar{X}$ for the angle?

As used in AISC *Specification* Section D3.1, $\bar{X}$ can be but is not always the same as the similar variable shown in the AISC *Steel Construction Manual* dimensions and properties tables. The dimension in Section D3.1 is always measured perpendicular to faying surface of the connected element. For example, consider a single angle with unequal legs:

- If connected to the short leg only, then the distance perpendicular to the connected leg is $\bar{Y}$ from the AISC *Manual* dimensions and properties table.
- If connected to the long leg only, then the distance perpendicular to the connected leg is $\bar{X}$ from the AISC *Manual* dimensions and properties table.

Heath Mitchell, S.E., P.E.

Partially Restrained Connections

AISC *Steel Construction Manual* Figure 11-3 illustrates a flange-plated partially restrained moment connection. It shows only the top plate weld being held back 1.5 times the flange plate width from the edge of the plate. Should the bottom plate weld also be held back in a similar manner?

Yes. The original intent of Figure 11-3 was to show that some fit-up clearance could be provided between the top flange and the flange plate without hindering the welding of the two elements. The welds on both the top and bottom plates are intended to be held back to allow yielding of the plates. This is shown in Figure 11-4 of the 14th Ed. AISC *Manual*.

It should be noted that this is different than the detail shown in Blodgett's *Design of Welded Structures*, which is referenced in this section of the AISC *Manual*. The detail shown by Blodgett allows for rotation in only one direction and therefore shows only the welding on the top plate held back. Blodgett references tests by Johnston and Deits, and his details reflect these tests.

Larry S. Muir, P.E.

WUF-W Prequalified SMF Connection

We are using a WUF-W special moment frame connection on a project. AISC 358 Section 8.6(2) requires that the weld of the single plate to column flange develop the expected shear strength ($0.6R_yF_y$) of the single plate. If I calculate the capacity of a CJP groove weld as $\phi(0.6F_{EXX})$ using matching weld metal, the weld is not strong enough. Is my only option to use double-sided fillet welds?

No. CJP groove welds made with matching weld metal are considered to fully develop the part. This is shown in AISC 360 Table J2.5, which states that the strength of the joint is controlled by the base metal for CJP groove welded joints—i.e., the $\phi(0.6F_{EXX})$ calculation does not apply to CJP groove welded joints made with matching weld metal.

There are other things you may want to consider when welding this single plate. You mentioned using double-sided fillet welds as an option. A fillet weld in the root gap for the beam web-to-column flange CJP groove weld may be disruptive to fit-up of the joint and plumbing of the frame. Also, since the beam web-to-column flange weld is made in the field and the single-plate welds are typically made in the shop, then the weld metal intermix requirements of AWS D1.8 Clause 6.3.4 need to be considered.

Heath Mitchell, S.E., P.E.

Welder Qualification

I am reviewing welder qualifications for a project. Certificates for fillet welds and CJP groove welds have been submitted. The contractor claims that these cover flare-bevel groove welds as well. Is this true? The welding certificates are from many years ago. How do I know that they have been properly maintained?

The AISC *Specification* defers to AWS D1.1 on the subject of welder qualification. I will offer some input based on my understanding of the AWS D1.1 *Code*.

Flare-bevel groove welds are considered to be PJP groove welds. Per AWS D1.1 Clause 4.25, a CJP groove weld qualification test also qualifies the welder for PJP welds. Clause 4.26 states the same for fillet welds. (Also, see Tables 4.10 and 4.11 for welding positions qualified and range of thickness qualified.)

As to continuity, AWS D1.1 Clause 4.2.3.2 states that a welder's qualification remains in effect indefinitely unless he or she is not engaged in the qualified process for a period exceeding six months. If you wish to verify continuity, you will need to request a "continuity log" of some type to verify compliance to this provision.

Keep in mind that welders are qualified by process and position. If a qualification was done with SMAW, then the welder is only qualified for the SMAW process in production (see AWS D1.1 Table 4.12). Welders tested in the flat position are qualified to weld in the flat position. Welders tested in the vertical position are qualified in vertical, flat and horizontal positions, and other combinations are shown in AWS D1.1 Table 4.10. Furthermore, if the welder tested on a ⅜-in. plate for CJP groove weld qualification, then he or she is limited to ¾ in. thickness, whereas tests on a 1-in. plate qualify the welder for unlimited thickness per AWS D1.1 Table 4.11.

Keith Landwehr

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NDT Methods and Frequencies

Is there a resource that can help an engineer determine the industry standard frequency and extent of RT, UT and MT to use for a project?

Historically, the decision has been left to the EOR to determine applicable NDT methods and frequencies with a few exceptions. Codes such as AWS D1.1 provide NDT procedures and acceptance criteria, but application of such has been left for the EOR to determine and then place in their specification or general notes.

With the introduction of the 2010 AISC *Specification*, AISC has incorporated a new Chapter N—Quality Assurance and Quality Control. The intent of this new chapter is to provide engineers with a standard, or model, QA/QC plan. Included are inspector qualifications and required inspections, as well as inspection frequencies. Section N5.5—Nondestructive Testing of Welded Joints contains requirements for frequency and extent of NDT for welded joints. Hopefully, the Chapter N QA/QC Plan will prevent EORs, such as yourself, from having to "start from scratch," so to speak, when determining inspection requirements for their projects.

Keith Landwehr

Upper Limit on C_b

In AISC 360-05 Equation F1-1, the calculation for C_b had an upper limit of 3.0. This limit was removed in AISC 360-10. Why was the upper limit removed?

The upper limit of 3.0 only applies to singly symmetric beams in reverse curvature. However, since the scope of application for AISC 360-05 Equation F1-1 included singly and doubly symmetric members loaded in single or double curvature, the upper limit of 3.0 was conservatively applied to the other cases.

In AISC 360-10, we removed the upper limit on Equation F1-1 and made a corresponding change to the scoping statement for the equation. AISC 360-10 Section F1(3) states that the equation for C_b (without the upper limit of 3.0) applies to "singly symmetric members in single curvature and all doubly symmetric members." The last sentence of the User Note in this section sends the reader to the Commentary for other situations such as singly symmetric members in reverse curvature. Commentary Equation C-F1-3 applies specifically to singly symmetric beams in reverse curvature and includes the upper limit of 3.0

A more detailed discussion of this topic can be found in the Commentary to AISC 360-10 Section F1.

Brad Davis, S.E., Ph.D.

Riding the Fillet

What is the maximum a connection element can encroach on the web-to-flange fillet of a rolled W-shape? Is this allowed even if it is a slip-critical bolted connection?

Figure 10-3 in the AISC *Manual* presents recommended distances for riding the fillet. This figure applies to snug-tightened, pretensioned and slip-critical joints. When the plate or angle rides the *k*-area by this amount, the assembled connection can meet the requirements for the snug-tightened condition. Once the snug-tightened condition is met, the bolts can be pretensioned as required for a slip-critical connection. The gap when riding the fillet is small enough that the bolt pretension will close it in the areas required to be in contact for slip resistance.

Larry S. Muir, P.E.

Tee Stem Local Buckling

AISC 360-05 Table B4.1 does not address flexure in stems of tees. What are the appropriate width-to-thickness limits for this condition?

The stem local buckling strength limit state was embedded into the lateral-torsional buckling equations in the 2005 AISC *Specification*. This is noted in the Commentary to Section F9 on page 277. Therefore, there was no need for a width-to-thickness limit to be defined for this case.

This has changed in the 2010 AISC *Specification*. The stem local buckling strength is no longer included in the lateral-torsional buckling equations. Tee stem width-to-thickness limits were added as Case 14 in Table B4.1 and a new Section F9.4 was added to specifically address the stem local buckling strength.

Brad Davis, S.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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