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Fillet Weld Terminations

Our company standard is to extend fillet welds to the ends of connected parts unless noted otherwise on the construction documents. On a recent project, the inspector mentioned we should not be extending our welds to the ends of the part, but rather should terminate them one weld size before the edge. Is this correct for statically loaded fillet welds?

Not necessarily. Fillet weld terminations are addressed in AISC *Specification* Section J2.2b. Roughly two-thirds of the way through that section, you will find the statement: "Fillet weld terminations are permitted to be stopped short or extend to the ends or sides of parts or be boxed except as limited by the following." Four cases are then listed that have specific requirements. As long as one of these four cases does not apply to your joint, then the fillet welds can be stopped short or extended; either practice is acceptable.

If fillet welds are terminated, the inspector is correct regarding the appropriate distance to terminate a fillet weld from the edge of the part. Please see the "User Note" after the list of four cases in Section J2.2b. The user note recommends that "fillet weld terminations should be located approximately one weld size from the edge…"

Keith Landwehr

Bolt Installation

I was recently told by a steel erector that the steel used on a project had a "high friction coefficient," which made it excessively difficult to apply the turn-of-nut method for tightening bolts. The connections used 1-in.-diameter A325 bolts in standard holes to join two flat plates. Per RCSC *Specification* Section 6, washers were not required to be used under the bolt head or the nut. Is there a requirement for a maximum friction coefficient between the turned element and base metal when using the turn-of-nut installation method?

No. The friction coefficient between the turned element and the base metal is not specified for the turn-of-nut installation method in the RCSC *Specification*. The friction coefficient between the turned element and the base metal will vary based upon the surface condition and smoothness of each surface. It can depend on the materials and the exposure they have experienced, and also on whether the turned element galls the surface on which it is turned. Some people in the industry prefer to use a hardened washer under the turned element, even when it is not a specification requirement. Doing so makes for a more predictable surface under the turned element, and also eliminates the potential for galling.

Erin Criste

Built-Up Column Design

As part of a renovation project, I need to add cover plates to an existing wide-flange column in order for it to be able to carry additional load. I am having difficulty determining the effective slenderness ratio for this cross-section per AISC *Specification* Section E6.1. How are the variables α, a and r_{ib} determined for a cover-plated wide-flange column?

AISC *Specification* Section E6.1 does not apply to your built-up cross-section. The scoping statement of this section identifies that Section E6.1 applies to built-up members composed of two shapes. The intent is that they are members similar to double-angles or double-channels. Cover plates are not considered rolled shapes. The modified slenderness ratio in Section E6.1 is included as a convenience in lieu of specifically accounting for shear forces and deformations between the individual elements of the built-up member.

The prescriptive requirements of Section E6.2 do apply and you will likely need to do some calculations to determine the required shear flow between the wide-flange shape and the plates. It is likely that the prescriptive requirements will be sufficient for shear flow, but you will have to determine that for your particular case. One approach is to use an analysis similar to what is done in the following AISC *Engineering Journal* article: "Analytical Criteria for Stitch Strength of Built-Up Compression Members" by Aslani and Goel (3rd Quarter 1992). This article is available at **www.aisc.org/epubs**.

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

Flange Local Bending

In AISC 360-10 Section J10.1, Flange Local Bending, why is the width of the flange, b_f, not included in the equation for flange local bending capacity? One would think that a wider flange would have less bending capacity than a narrower flange of the same thickness.

The flange width is incorporated into the derivation of the equation for flange local bending capacity, but it drops out since it is on both the demand and the resistance side. On the demand side, the flange width is used to calculate the total load applied and its moment arm. On the resistance side, an approach similar to a yield line analysis is used to determine the amount of the flange, in the longitudinal direction, that participates in the resistance. This is dependent on the flange width.

The equation is based on the work of Graham (1960) listed in the references to the *Specification*.

Larry S. Muir, P.E.

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ANSI Roughness Criteria

AISC *Code of Standard Practice* Section 6.2.2 says surfaces noted as finished on the drawings are defined by a maximum ANSI roughness height of 500. Can you explain what the height of 500 is and how it is measured?

The 500 value refers to a finished surface roughness of 500 µin. (micro-inches). The user note to AISC *Code* Section 6.2.2 states that most cutting and milling processes meet this requirement. Guidance for measuring surface roughness is found in ANSI/ASME B46.1.

Erin Criste

HSS Connection

Using the equations in AISC 360-10 Table K3.1 for round HSS-to-HSS moment connections, my connection has more capacity than the branch member itself, as determined by AISC 360-10 Section F8. This seems odd. It would seem that the equations in Section K3 should have an upper bound of the member capacity given in Section F8. Why is this not the case?

AISC *Specification* Chapter K addresses connections between HSS in a manner consistent with how Chapter J addresses other connections. For example, one could put 100 rows of bolts in a W8×10 and calculate a bolt group strength that greatly exceeds the member strength, but the strength of the system will still be limited to that of the member. Chapter K only addresses the local effects of the connections, not the strength of the members themselves, which are addressed elsewhere in the *Specification*.

Larry S. Muir, P.E.

Extended Single-Plate Connection

The AISC *Manual* only shows stabilizer plates graphically at a beam-to-column web connection. Would the same concept apply to a beam-to-beam connection where we have to use an extended single plate? In this case we would end up with a full-depth shear plate using the beam flanges as the stabilizing element.

The need to check for adequate stabilization of the supported beam applies to any extended plate configuration, regardless of the supporting member. Stabilizing plates are only required when the extended single plate does not have the torsional strength to resist lateral displacement of the beam in the connection region. The following *Engineering Journal* article discusses how one determines if stabilizer plates are needed: "On the Need for Stiffeners for and the Effect of Lap Eccentricity on Extended Shear Tabs" by W.A. Thornton and P. Fortney (2nd Quarter 2011). This article is available at www.aisc.org/epubs.

The results of this paper have been incorporated into the 14th Edition AISC *Manual* discussion of, and design procedure for, extended single plates.

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

Seismic Compactness

According to AISC 341-05 Section 8.2b, members that are required to be seismically compact shall not have elements that exceed the limiting width-thickness ratios of Table I-8-1. Can a section that is not seismically compact be used if its available strength is determined using either of the following?

(a) An effective area and section properties calculated using reduced element widths that meet the maximum width-to-thickness ratio requirements of Table I-8-1.

(b) An effective yield stress determined from the width-to-thickness ratio meeting the requirements of Table I-8-1.

No. Your approach may work for members that behave elastically, but it is not appropriate for members that are expected to have stable cyclic performance in the inelastic range. The Commentary to AISC 341-10 states: "To provide for reliable inelastic deformations in those members of the SFRS that require moderate to high levels of inelasticity, the width-to-thickness ratios of compression elements should be less than or equal to those that are resistant to local buckling when stressed into the inelastic range." Using lower stresses in design would not accomplish the same effect and would not satisfy the intent of the AISC *Seismic Provisions*.

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

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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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