Erection Tolerances

AISC *C d S a a P ac c* Figure C-7.5 shows both a 1/1000 and a 1/500 tolerance on columns. Are both of these erection tolerances? Are these tolerances measured at the top of steel or floor level?

The 1/500 tolerance is an erection tolerance. The 1/1000 tolerance is a fabrication tolerance. From Section 6.4.2:

> "…For straight compression members, whether of a *u u h* or built-up, the variation in straightness shall be equal to or less than 1/1000 of the axial length between points that are to be laterally supported."

Figure C-7.5 in the Commentary to Section 7.13 illustrates mill, fabrication and erection tolerances that need to be considered for cladding systems. The following excerpts from the Commentary to Section 7.13 help to demonstrate the intent of this figure (and other related figures):

> "The alignment of lintels, spandrels, wall supports and similar members that are used to connect other building construction units to the *u u* frame should have an adjustment of sufficient magnitude to allow for the accumulation of mill tolerances and fabrication tolerances, as well as the erection tolerances. See Figure C–7.3."
>
> "The limitations that are described in this Section and illustrated in Figures C–7.4 and C–7.5 make it possible to maintain built-in-place or prefabricated facades in a true vertical plane up to the 20th story, if that provide for 3 in. [75 mm] of adjustment are used."

The 1/500 is an erection tolerance found in AISC

- "Erection tolerances shall be defined relative to member working points and working lines, which shall be defined as follows:
- (a) For members other than horizontal members, the member work point shall be the actual center of the member at each end of the shipping piece.

  …
- (c) The member working line shall be the straight line that connects the member working points."

The AISC *C* erection tolerances apply to the working points only. This is the top and bottom of a column shipping piece. If the column shipping piece spans multiple levels, the location of the column at levels between its ends is not subject to AISC *C* tolerances. Similarly, the location of a column at beam top of steel or at the top of slab is not necessarily subject to an AISC *C* erection tolerance. AISC *C* erection tolerances apply to the top and bottom of a building column and at any column splices in between.

Finally, it is worth noting that the 1/500 and 1/1000 tolerances only add directly in a specific worst case scenario. Consider three tiers of framing in which the bottom tier leans to the left at 1/500, the middle tier is plumb and the top tier leans 1/500 to the right. Figure C-7.5 illustrates this worst case where the envelope at mid-height of the second tier is the sum of the 1/500 lean and the 1/1000 curvature.

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Shear Lag

Does the concept of shear lag apply to connection elements? Since it is located in AISC 360 Chapter D Tension Members, one might assume it only applies to members.

Yes. As evidence, see Section J4.1 of the 2010 AISC , which specifically refers to Section D3 when it defines the effective net area as "effective net area as defined in Section D3, in.[2] (mm[2]); for bolted splice plates, $A = A$

connections, and the load
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Some engineers argue t

(FR) moment connection?

No. The 5⁄8 requirement is part of a recommended design procedure for shear tabs that are designed as simple shear connections and is not necessary for a shear tab used as part of an FR moment connection.

In a simple shear connection, you need to meet the requirements of AISC Section B3.6a, which states that a connection shall have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure. Since a shear tab is a stiff connection, this requirement is met by controlling the ratio of the bolt diameter to plate thickness and the 5⁄8 weld requirement; these combine to make the plate the controlling element in the connection. This allows for a ductile redistribution of moments and a weld that can develop the plate. By ensuring that the plate yields before the weld ruptures, the simple beam end rotation is accommodated through a combination of plate yielding and bolt plowing.

In an FR moment connection in =3 applications, there is little or no beam end rotation and the shear tab will not see any moment. It is reasonable to size the shear tab weld for shear only. A discussion that outlines this for FR moment connections can be found on pages 12-2 and 12-3 of the 14th Edition AISC *C u u*. In high-seismic applications with SMFs or IMFs, the foregoing discussion is irrelevant because the web connection matches a prequalified detail or a detail that is qualified by testing.

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Section M2.8 provides requirements related to the treatment of the base plates based on thickness.

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