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# steel interchange

## Erection Tolerances

**AISC *Code of Standard Practice* 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 *standard structural shape* 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 *structural steel* 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 *connections* that provide for 3 in. [75 mm] of adjustment are used."

The 1/500 is an erection tolerance found in AISC *Code* Section 7.13.1.1 as follows:

> "For an individual column shipping piece, the angular variation of the working line from a plumb line shall be equal to or less than 1/500 of the distance between working points"

AISC *Code* Section 7.13 defines the working points and working line of a column as:

> ➤ "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 *Code* 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 *Code* 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 *Code* erection tolerance. AISC *Code* 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.

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

## 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 *Specification*, 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, $A_e = A_n \leq 0.85A_g$." So, Section J4.1 does require that shear lag be considered in connection elements.

More generally speaking, shear lag is defined in the AISC *Specification* as "Nonuniform tensile stress distribution in a member or connecting element in the vicinity of a connection." From this it is clear that shear lag can apply to either members or connecting elements, but even on a more basic level nonuniform tensile stress distributions can clearly exist in either members or connecting elements.

The concept of shear lag is predicated upon conditions that usually exist in a tension member; there is a uniform stress distribution along the length of the member between connections, and the load is removed at the connection over a length that affects the efficiency over which the load can be removed. That efficiency is what the shear lag effect describes.

Some engineers argue that typical connecting elements are too short to use the same rules we use for members and that the effective area of connecting elements should be limited to the area of only the portion connected (the area of the connected leg only of a claw angle, for example). However, this is not usually the approach taken by the Manual Committee in the AISC Design Examples. Usually, the same shear lag criteria are used for connection elements, though there are some cases where the area of the connected part is used for simplicity or because the shear lag calculation need not be taken less than the connected area. Other special approaches also are used, such as the Whitmore section in gusset plates. This is, after all, just another way to account for nonuniform stress distributions (shear lag) in a connecting element.

*Larry S. Muir, P.E.*

## Shear Stud Tolerances

**Are there any tolerances on the placement of shear studs in a composite beam? Can studs that meet the minimum flange thickness criteria be placed off-center of the web?**

The minimum thickness requirement to which you refer is stated in AISC *Specification* Section I8.1; it limits the diameter of the stud to 2.5 times the thickness of the flange unless it is welded directly over the web. In practice, this limit is applied the other way; if the number of studs to be placed in a group requires placement off the web, the designer should select only beams with flange thickness equal to or greater than the stud diameter divided by 2.5. This requirement is easy to comply with for W-shapes since the minimum flange thickness is 0.3 in. for a ¾-in.-diameter shear stud.

Clause 7.4.5 of AWS D1.1 indicates a 1-in. tolerance on the longitudinal and lateral spacing of studs and prohibits the stud from being placed less than the diameter of the stud plus ⅛ in. from the edge of the flange (recommending not less than 1½ in.).

As long as these requirements and tolerances are met, the stud can be placed off-center.

*Larry S. Muir, P.E.*

## Single-Plate Design

**Does the ⅝$t_p$ weld size requirement apply to a single-plate shear connection that is part of a fully restrained (FR) moment connection?**

No. The ⅝$t_p$ 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 *Specification* 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 ⅝$t_p$ 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 $R$=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 *Steel Construction Manual*. 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.

*Carlo Lini, P.E.*

## Contact Bearing

**AISC *Specification* Section M2.8,** Finish of Column Bases**, states that milling is not necessary "provided a satisfactory contact bearing is obtained." What constitutes a satisfactory contact bearing?**

AISC *Specification* Section M4.4 provides the tolerance. It states: "Lack of contact bearing not exceeding a gap of 1⁄16 in. (2 mm), regardless of the type of splice used (partial-joint-penetration groove welded or bolted), is permitted. If the gap exceeds 1⁄16 in. (2 mm) but is equal to or less than ¼ in. (6 mm), and if an engineering investigation shows that sufficient contact area does not exist, the gap shall be packed out with non-tapered steel shims. Shims need not be other than mild steel, regardless of the grade of the main material."

Therefore, in your condition if "sufficient contact area does not exist," then the gap can be packed out with non-tapered steel shims under the offending flange. Note that there is no requirement to provide a shim, so if you can transfer the forces considering the gap, then it is okay to leave as is. Satisfactory contact bearing is determined based on the area in contact and its bearing capacity compared with the load required to be transferred between the column and the base plate.

A square saw cut is generally sufficient to provide bearing at column ends, so milling of the column is typically not done. Section M2.8 provides requirements related to the treatment of the base plates based on thickness.

*Larry S. Muir, P.E.*

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