STEEL CONSTRUCTABILITY CONSIDERATIONS

Expectations vs Reality

SEAC/RMSCA Steel Liaison Committee

September 2019

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Executive Summary

The objective of this white paper is to raise awareness of constructability issues by presenting some of the more common constructability challenges and solutions that committee members address in their day-to-day practices. While the constructability issues presented are derived primarily from structural steel projects, the lessons learned may be applicable to projects using other structural systems.

While older construction projects tended to use a limited number of structural systems, many modern era construction projects utilize a combination of structural systems. These days, it is not uncommon for a structural steel project to also include one or more of the following structural systems: cast-in-place concrete, precast concrete, reinforced masonry, light-gauge steel framing, conventional wood framing, heavy timber framing, structural glass, etc. Each system is represented by an industry in its own right, with its own standards, construction customs, and tolerance expectations. Those standards, customs, and tolerances are often incompatible where different systems interface and is a primary source of modern era constructability challenges. This white paper presents some of the more common examples of clashes between different systems. Other more mutually exclusive constructability challenges presented by this white paper are related to erection stability, construction sequence, welding clearance, member availability, and cumulative tolerance impacts.

For each example of a constructability challenge that is presented, the white paper offers suggestions intended to help structural steel projects navigate these types of constructability issues. These suggestions are derived from the experience of committee members, and although they may or may not be directly helpful/applicable to the reader's projects, their main purpose is to encourage the reader to try to anticipate common constructability challenges and to be proactive in resolving them before they become a significant problem on their own projects.

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1.0 Introduction

The members of this steel committee collectively have over two hundred years of experience designing and constructing steel structures, primarily in the Rocky Mountain region. From a strictly empirical perspective, it has generally been the experience of the committee members that the challenge of constructability for building structures of all types, including structural steel, has increased over the past few decades. Anecdotally, the committee identifies the seemingly ever-increasing quantity of RFIs as one indication of increasing constructability issues, as well as several potentially contributing factors, such as increased building complexity, compressed design and construction schedules, reduced engineering fees (when taken as a percentage of construction cost), increased reliance on computer modeling and other software methods, increased reliance on delegated design, expanding and ever-changing codes, a declining labor pool, etc.

2.0 Steel Constructability Considerations

2.1 Mill and Fabrication Tolerances

2.1.1 Mill Tolerances

Variations in the cross sectional geometry of hot rolled structural shapes are an unavoidable reality in steel design and construction. These variations occur at the mill, during and after the hot rolling process, and can be caused by thermal distortions, differential cooling distortions, and roll wear. This is understandable when considering the difficult task of forming masses of hot, liquid steel into relatively precise solid structural shapes. Acceptable mill dimensional tolerances have been established in ASTM A6/A6M-17a (ASTM, 2017) and are summarized in Tables 1-22 through 1-26 in the AISC Steel Construction Manual (AISC, 2017).

2.1.2 Fabrication Tolerances

Variations in member length, member straightness, and accuracy of curved, cambered and built up members represent variations that can be controlled in the fabrication shop. Like mill variations, fabrication variations are an unavoidable reality and are related to each Fabricator's specific equipment, processes, and personnel. AISC has established permissible fabrication tolerances for such variations, and defined them in Section 6 of the AISC Code of Standard Practice for Steel Bridges and Buildings (COSP, 2016).

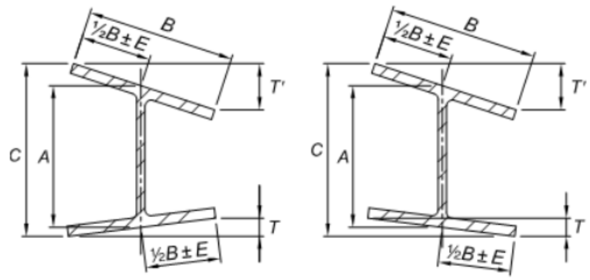
2.2 Beam Depth/Out-of-Square

Cross section variances that result in a beam being deeper or shallower than theoretical are often referred to as beam depth tolerances. Similarly, cross sectional variances that result in the corners of members being further away from or closer to their theoretical depth (such as may result from wide flange members having their flanges tilted) are referred to as out-of-square tolerances. [See Table 1 for a graphical depiction of these variances it is important to note that these variances cannot always be altered by the fabricator or erector. They are known unknowns and should be considered by the EOR, Detailer, Fabricator, and Erector to simplify/ease fit-up where members are joined. Fitting of connection material by the Fabricator is one opportunity to address these issues prior to erection but often has minor effects upon connection design. Examples include slotted versus standard holes, filler plates, and reduced or increased edge distances.

Depth and out-of-square tolerances (condensed to just "depth" for brevity hereon) are specified in ASTM A6/A6M (ASTM, 2017) and reproduced in the AISC Steel Construction Manual Table 1-22 (AISC, 2017). This discussion focuses on connection fit-up, so other tolerances including width, sweep, camber, cross-sectional area, etc. are only mentioned in passing.

1. Depth tolerance is +/- $^{1}/_{8}$ inch measured at center
2. Out of square tolerance varies based on depth; +/- ¼ inch or +/- $^{5}/_{16}$ inch
3. Max depth tolerance measured at any section is ¼ inch

Table 1-22
ASTM A6 Tolerances for W-Shapes and HP-Shapes



Permissible Cross-Sectional Variations

Nominal Depth, in.	A, Depth at Web Centerline, in. Over	A, Depth at Web Centerline, in. Under	B, Flange Width, in. Over	B, Flange Width, in. Under	T + T′, Flanges Out of Square, Max. in.	E[a], Web Off Center, in.	C, Max. Depth at any Cross Section over Theoretical Depth, in.
To 12, incl.	1/8	1/8	1/4	3/16	1/4	3/16	1/4
Over 12	1/8	1/8	1/4	3/16	5/16	3/16	1/4

Permissible Variations in Length

Nominal Depth[b]	Variations from Specified Length for Lengths Given, in.			
	30 ft and Under		Over 30 ft	
	Over	Under	Over	Under
Beams 24 in. and under	3/8	3/8	3/8 plus 1/16 for each additional 5 ft or fraction thereof	3/8
Beams over 24 in., All columns	1/2	1/2	1/2 plus 1/16 for each additional 5 ft or fraction thereof	1/2

Mill Straightness Tolerances[c]

Sizes	Length	Permissible Variation in Straightness, in. Camber	Permissible Variation in Straightness, in. Sweep
Flange width equal to or greater than 6 in.	All	$1/8 \text{ in.} \times \frac{(\text{total length, ft})}{10}$	
Flange width less than 6 in.	All	$1/8 \text{ in.} \times \frac{(\text{total length, ft})}{10}$	$1/8 \text{ in.} \times \frac{(\text{total length, ft})}{5}$
Certain sections with a flange width approx. equal to depth & specified on order as columns[d]	45 ft and under	$1/8 \text{ in.} \times \frac{(\text{total length, ft})}{10}$ with 3/8 in. max.	
	Over 45 ft	$3/8 \text{ in.} + \left[1/8 \text{ in.} \times \frac{(\text{total length, ft} - 45)}{10}\right]$	

Other Permissible Rolling Variations

Area and Weight	−2.5 to +3.0% from the theoretical cross-sectional area or the specified nominal weight[e]
Ends Out of Square	1/64 in., per in. of depth, or of flange width if it is greater than the depth

Table 1: Mill Cross-Section Tolerances for W Shapes per ASTM A6/A6M-17 (ANSI/AISC 360-16)

2.3 Controlling Side / Location:
Steel members are typically detailed based on a controlling side or location.

1. For horizontal roof and floor members, this is typically the top of the member. Bolt holes (and consequently connection material) are detailed from top of steel down (locating the top bolt 3" below top of steel is common). This ensures correct fit-up of connections since beam depth tolerance is avoided entirely.

2. Centerline of columns, vertical braces and other non-horizontal members. This tolerance is factored into connection detailing along with beam overrun/underrun, connection detailing and erection methodology. It will be covered in the "tight connections" portion of this paper.

2.4 Common Details Where Member Tolerances Affect Fit-Up:

1. Flange plate moment connections (bolted or welded)
2. Flange plate column/beam/chord splices, as shown in Figure 1

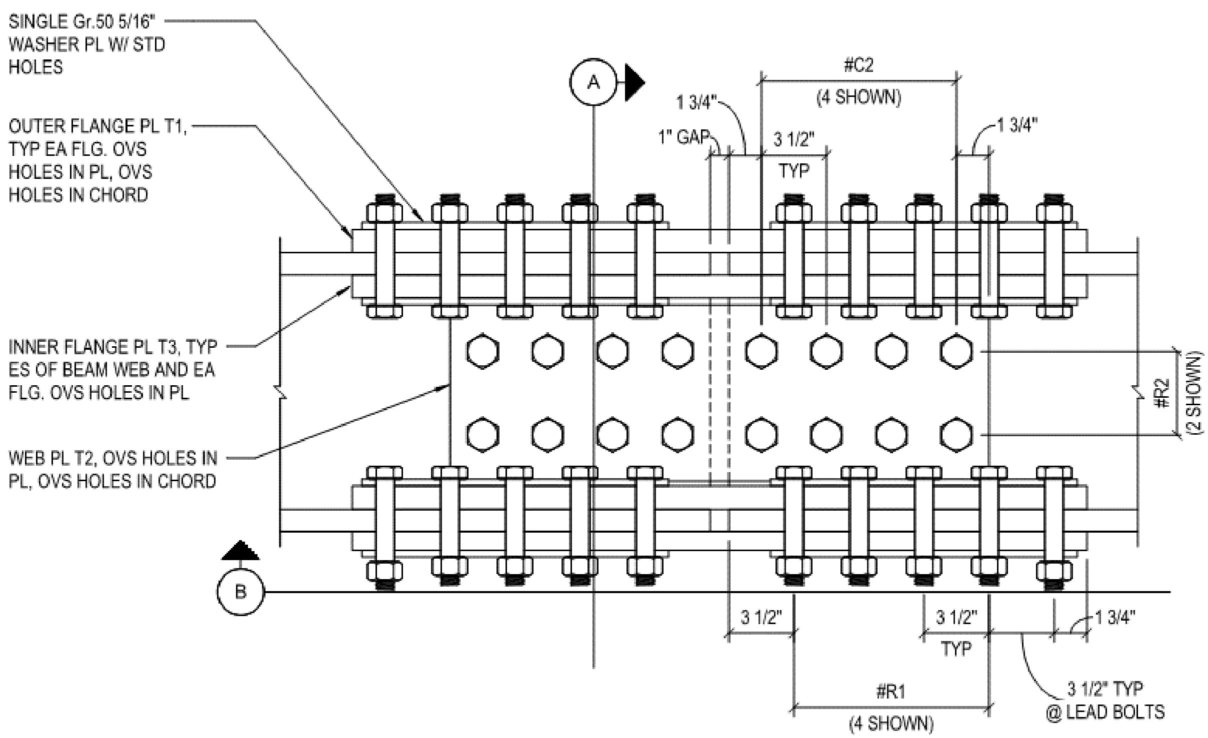


Figure 1: Common Wide Flange Splice Connection

3. Column splices
4. Beams running continuous through columns
5. Moment connections across beams of similar depth, as shown in Figure 2

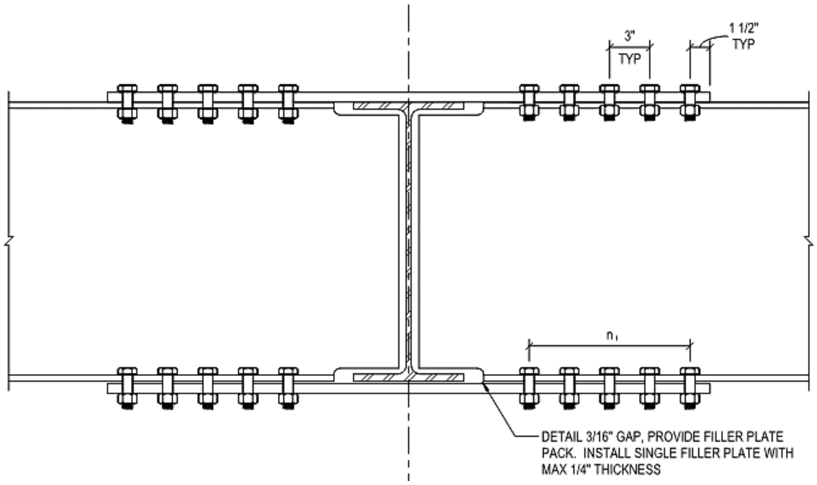


Figure 2: Flange Plate Moment Connection across Girder

6. Brace frame gussets where the plate is shop-welded to the bottom of the beam and brace is bolted

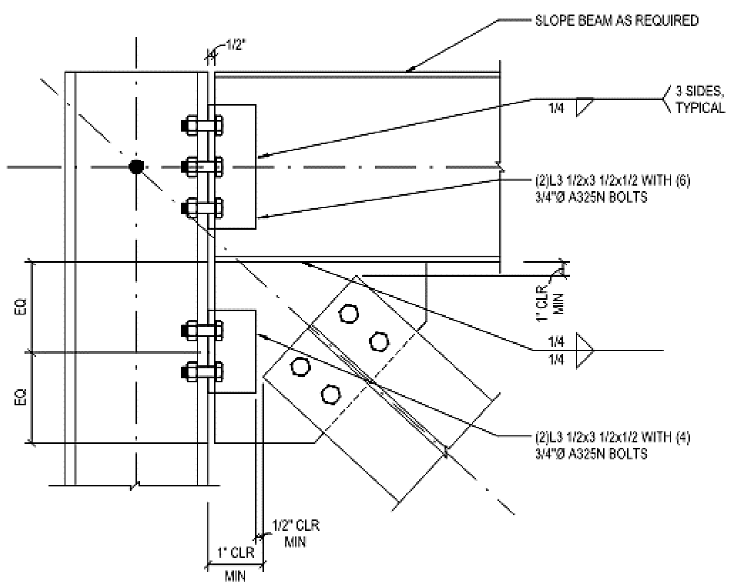


Figure 3: Vertical Brace Connection: Gusset Shop-Welded to Beam

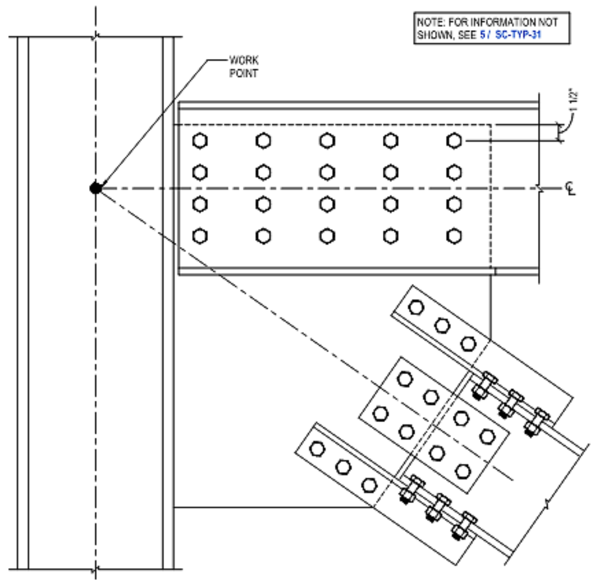


Figure 4: Vertical Brace Connection: Gusset Shop-Welded to Beam

7. Welded connections can be challenging too if the brace slot isn't long enough to allow for beam depth/tilt when field installed

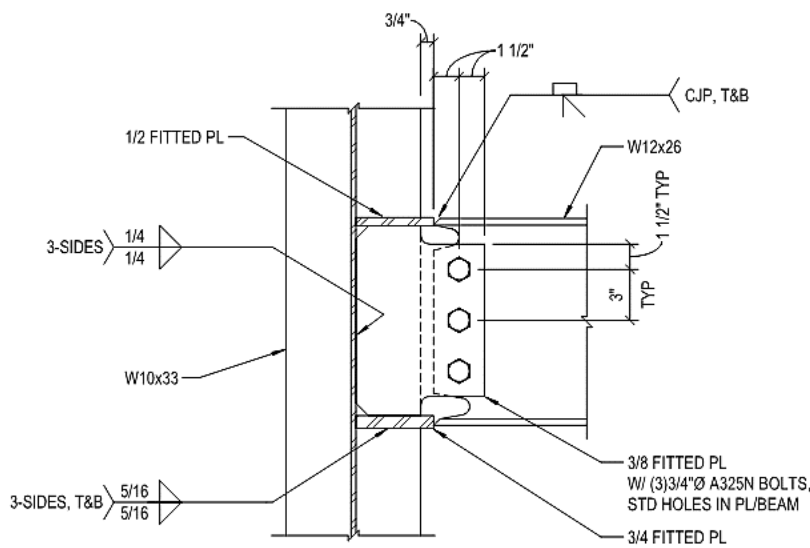


Figure 5: Moment Connection to Column Web

8. Connections where one beam is defined based on top of steel and another on bottom of steel (i.e. one beam is bearing on a wall or column, and the other supported by a girder

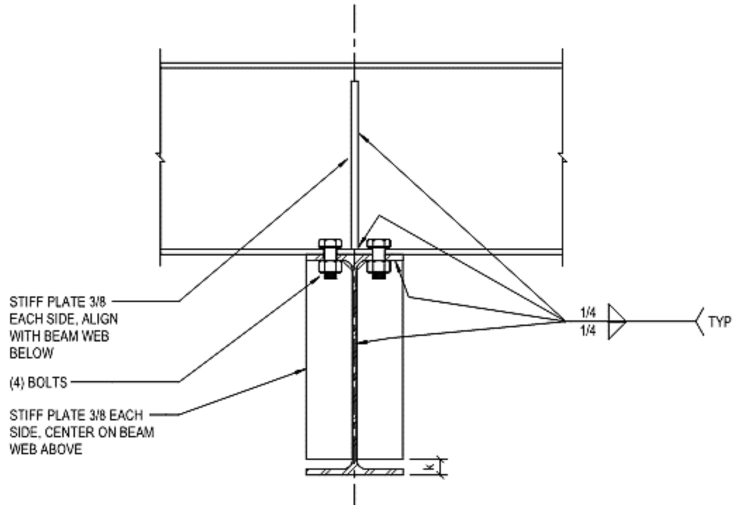


Figure 6: Beam over Girder

2.5 Common Methods to Resolve Depth Tolerance:

1. The EOR dictates framing sizes and configurations on all projects, regardless of whether connection design is delegated. The choice to frame beams through columns, bear over columns/walls as well as frame moment connections across girders of like-depth is the EOR's alone (although it can be remedied by a savvy fabricator and willing EOR partner).
2. Connection selection by the EOR, fabricator, erector or connection engineer is an early method to mitigate fit-up issues. Use of extended end plate moment connections (with filler plates) and bracing connections that avoid top-down detailing can avoid many problems. It is important to note that some of these decisions affect member design. A common rule of thumb is to limit member utilization to 85% (shape factor, flexural rupture).
3. Use of slotted holes in the direction perpendicular to load for connections such as flange WT moment connections allows for the resolution of minor tolerances.
4. Filler plates are a very common method of resolving depth tolerance. Newer codes offer less-restrictive options for transferring load. The loss of permissible bolt shear is relatively slight for bearing bolts and there is no loss when using slip-critical bolts. Refer to Section J5 of AISC 360-16 (AISC, 2016b) for further information and limitations.
    a. Reduction in bolt shear capacity due to filler plates
    b. No loss in capacity for slip critical bolts
    c. Develop filler plates
5. Over-sized holes with slip-critical bolts allow for an additional 1/16 inch in any direction relative to standard holes. Note that loss of bolt shear capacity is significant due to the need to use slip critical bolts.
6. Field welding is a common method of resolving tolerance by forcing fit-up to the as-built condition.
    a. Relative cost increase of field welding over shop welding is approximately 1 ½ times and the increase may approach double the cost based on the project location

b. Options for welding procedures are often limited due to equipment availability and environment
c. Weld access may be limited
d. Weld position is often dictated
e. Setup to weld often takes additional time (the weld station must travel to material rather than material travel to weld station as would be done in a fabrication shop).
f. Inspection

7. Flexibility may be built into adjacent members or within the connection itself to allow for minor adjustability during erection. A common example of this is to increase the gap between a moment-connection beam and the column and push the first row of bolts out away from the end of the beam. This gives the flange plates more length to flex. Downsides of this approach include increased connection material and unbraced length on compression elements. This approach is also limited in effectiveness to relatively thin plates. Note that for single plate connections, eccentricity on the bolt group may be neglected as rotation of the joint is limited by the moment connection.
    a. Flange plate example with leading ¾” diameter A325 bolts
        i. Minimum bolt pretension is 28 kips per AISC 360-16 Table J3.1 (AISC, 2016b)
        ii. With a = 2” and ½”x6” plate, deflection = 0.025”
        iii. With a = 4” and ½”x6” plate, deflection = 0.173”
8. The fabricator may take additional care during fit-up of known problematic connections. For example, the fabricator might measure beam depth and adjust gusset plate fitting accordingly or fit-up the bottom flange plate of a moment connection to compensate for the beam’s tolerance. This is especially useful if typical beam sizes are used throughout a project.
9. Another tool the fabricator may have is to take additional care during procurement. By sourcing material from same mill heat in connections such as chord splices, the chance of fit-up issues will be reduced. This may only apply to larger projects. 350T is a common minimum size to place a mill order.
10. Detail connection from top down, as shown in Figure 2

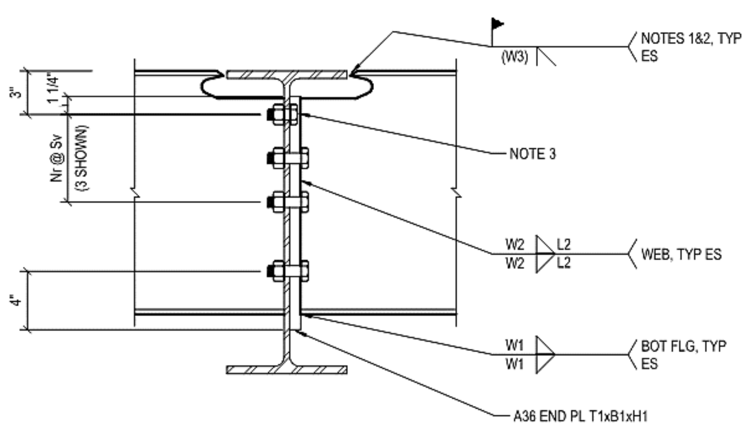


Figure 7: Connection Dimensioned From Top of Steel

2.6 Instances when Mill Tolerances Team up with Fabrication Tolerances – and Not in a Good Way

There are occasions when mill tolerances and fabrication tolerances can combine and accumulate to create field alignment issues if design and detailing do not allow for field adjustment.

Consider the following case study that includes 30ft bays of framing, with wide flange columns are oriented such that the beams are framing into the column flanges.

Figure 8: Ten Bay Frame

Per AISC COSP Section 6.4.1 (AISC, 2016a), beams with 30 feet spans or less can vary by ${}^{1}/_{16}$ inches in length. Beams over 30 feet lengths can vary by ${}^{1}/_{8}$ inches. So 10 beams, all under 30 feet, can each be long by ${}^{1}/_{16}$ inches. Per AISC 360-16 Table 1-22 (AISC, 2016b) the mill tolerance of wide flange columns can have depths measured at web centerline that vary by ${}^{1}/_{8}$ inches. So all 11 columns (in reality 10 since only half of each outside column needs to be accounted for) could be deep by ${}^{1}/_{8}$ inches.

Beams: 10 x ${}^{1}/_{16}$ inches = ${}^{5}/_{8}$ inches long overall
Columns: (9 x ${}^{1}/_{8}$ inches) + (2 x ${}^{1}/16$ inches) = 1-¼ inches deep overall
${}^{5}/_{8}$ inches + 1-¼ inches = 1-${}^{7}/_{8}$ inches long overall

Therefore, if the erector starts at one end with a plumb column, holds it plumb and starts erecting, the last column will be 1-${}^{7}/_{8}$ inches out of plumb (at the beam elevation)

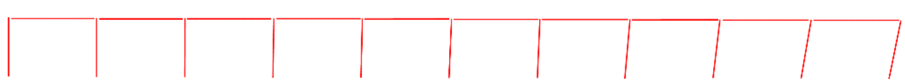


Figure 9: Ten Bay Frame – Potential Consequences of Combined Mill and Fabrication Tolerances

Now push the example to an extreme. Say the column spacing is 32 feet on center, meaning the beam lengths are over 30 feet.

Beams: 10 x ${}^{1}/_{8}$ inches = 1-¼ inches long overall
Columns (9 x ${}^{1}/_{8}$ inches) + (2 x ${}^{1}/_{16}$ inches) = 1-¼ inches deep overall
1-¼ inches + 1-¼ inches = 2-½ inches long overall

This results in the last column being out of plumb by 2-½ inches at the beam elevation.

The Engineer may consider allowing the Fabricator to detail the structure using either standard or short slotted holes as desired. The use of horizontal slots on repeated bays of framing can increase the Erector's effort to plumb the building, so often they will be used only every few bays.

Both of these examples assume that the beams are long AND the columns are deep. Similar results could occur if the beams were short and the columns were shallow. While it is not likely that every beam will be long by the maximum tolerance and ever column be deep by the maximum tolerance, but these examples illustrate how issues can arise. Even if deviant from theoretical by half of the tolerances, then the last column would still be out of plumb by $^{5}/_{8}$ inches for the 30 feet bay example and 1-¼ inches for the 32 foot bay example. It is worth noting that often material is purchased from a single mill heat and that all columns would have the same depth/tilt tolerance. As schedule allows, a Fabricator may measure the column material when it is delivered and adjust beam lengths and connection fit-up accordingly.

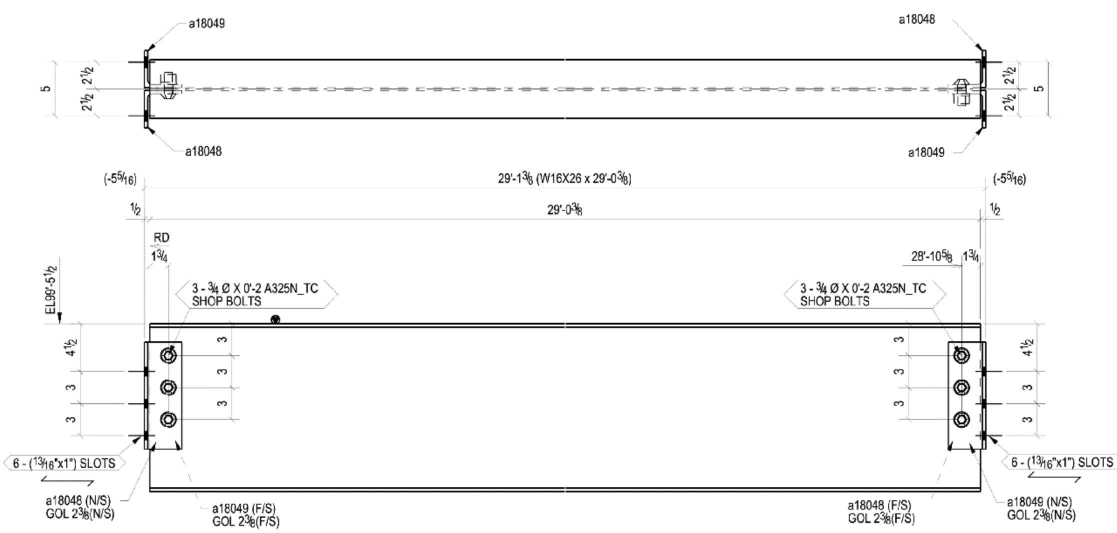


Figure 10: Double Angle Connections – Not Friendly to Tolerances

The same applies for end plate connections.

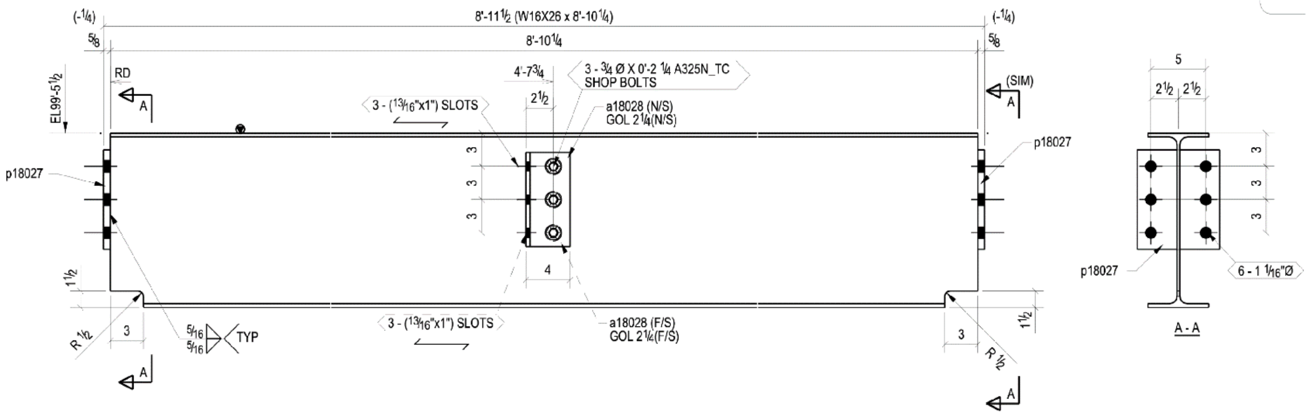


Figure 11: End Plate Connections – Not Friendly to Tolerances

2.7 Common Methods to Resolve Accumulating Mill & Fabrication Tolerances

Often the most cost effective solution is to allow for the beam to column flange connections to be either have periodic slotted holes in the connection material that bolts to the beam web. This allows for field adjustment as the pieces are erected.

If double angle or end plate connections must be used, then extra care is needed. The beams in some bays should be detailed short to allow the field to take up overruns in beam length and column depth. Conversely, filler plates can make up if beams are short or columns are narrow. The field will need to monitor the columns as they go and shim accordingly. Ideally the shims would be installed at the same time as the beams and not until after the connection is fit up.

2.8 HSS Constructability Considerations

The origin of what is now known as Hollow Structural Sections (HSS) was not in the structural arena. Carbon steel round sections that were initially used for mechanical applications to convey steam and gas later became common compression elements in industrial structures. The first rectangular HSS is thought to have been produced in England in 1952. Many years later in the United States, fabricators found they could create a rectangular or square section by cold forming a round section using their existing machinery. Unfortunately, the metallurgy of this practice was not adequately researched and often led to weldability issues during manufacturing and in the field. These concerns were the catalyst to the development of ASTM A500 (ASTM, 2018a) which was first published in 1964. The approval of ASTM A500 increased usage of HSS in the structural and industrial building markets. Steel pipe and HSS were first introduced into the AISC *Specification* in 1969. A separate Specification for the Design of Hollow Structural Sections was first published in 1997 and then incorporated into the main Specification, AISC 360, in 2005.

There are two manufacturing processes by which HSS are made in the United States. The first, and most common, is called Continuous Forming. This process starts with a flat strip of steel cut to size from a larger coil (Figure 12.A). This slit coil is then pushed through a series of rollers to form a round section (Figure 12.B). The weld is then made by heating the two edges (Figure 12.C) and pressing them together to form a closed section (Figure 12.D). The weld bead is then removed (Figure 12.E). If the final section is to be rectangular or square, it passes through a series of dies to cold form it into the final shape and size (Figure 12.F).

The second manufacturing process, which is used by only one domestic producer, is called Direct Forming. As the name implies, the slit coil is directly formed into approximately the final shape (round or rectangular) prior to welding, thus eliminating most of the cold working.

Figure 12: Continuous Forming HSS Shapes

2.8.1 Uses

HSS are most commonly used as columns and lateral braces in commercial buildings. Compared to an open section such as a wide flange, which has both a strong and a weak axis, round and square HSS members have the same strength in both axes, which is often a benefit for compression elements. Because of the closed cross section, HSS members have relatively high torsional strengths, and therefore, are efficient when used for curved or eccentrically loaded flexural members, such as an exterior beam supporting a cladding load. HSS are favored by architects for their aesthetics, and are often used in applications of Architecturally Exposed Structural Steel (AESS) roof screens, canopies, skylights, roof/exposed trusses and exterior wall framing and supports such as wind girts. The closed section shape can also be necessary for clean rooms or food processing facilities, as the members do not have areas for dust or other contaminants to accumulate in. Similarly, HSS members have low surface to area ratios, which can be valuable if expensive coatings are used.

2.8.2 Tolerances

ASTM A500/A500M has relatively tight mill tolerances resulting in dependable cross-section shapes for fabrication and detailing. Rectangular HSS naturally have slight convex curves on each face due to the manufacturing process described above, especially for larger and/or thicker-walled sections, refer to Figure 13. Tolerances for convexity are outlined in ASTM A500/A500M (ASTM, 2018a), which unfortunately, does not clearly express these tolerances in a simple way. Table 2 communicates the information more clearly by including ASTM A500/A500M-18 (ASTM, 2018a) along with the footnotes. Note that these tolerances also include any concavity or convexity.

Table 2: Outside Dimension Tolerances for ASTM A500/A500M (STI 2015)

Large Outside	H/B < 1.5	1.5 ≤ H/B ≤ 3.0	H/B > 3.0
H ≤ 2.5 in	H: ± 0.020 in	H: ± 0.020 in	H: ± 0.020 in
2.5 in < H ≤ 3.5 in	H: ± 0.025 in	H: ± 0.025 in	H: ± 0.025 in
3.5 in < H ≤ 5.5 in	H: ± 0.030 in	H: ± 0.030 in	H: ± 0.030 in
H > 5.5	H: ± 0.01H	H: ± 0.01H	H: ± 0.01H

For most commonly used shapes, this tolerance does not present a problem other than the cumulative issues presented in the prior section. For example, an HSS12x4 rectangular section can vary from 3.82 to 4.18 inches in its small dimension (section A-A in Figure 23). Its larger dimension can vary from 11.88 to 12.12 inches (section B-B in Figure 13).

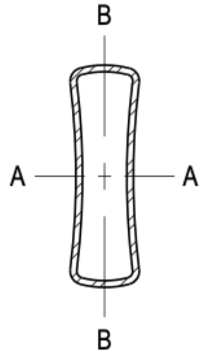


Figure 13: Tolerances for Rectangular HSS

However, for jumbo HSS (sections approximately HSS12x12 up to HSS24x12) the tolerances are more pronounced. In this case, it is recommended that directly welded connections not be used in order to allow for these convexity tolerances in the HSS sidewalls. Figure 14 below illustrates this. Please note that this is a very large, thick section and a very extreme example of bowing of the sidewalls of an HSS.

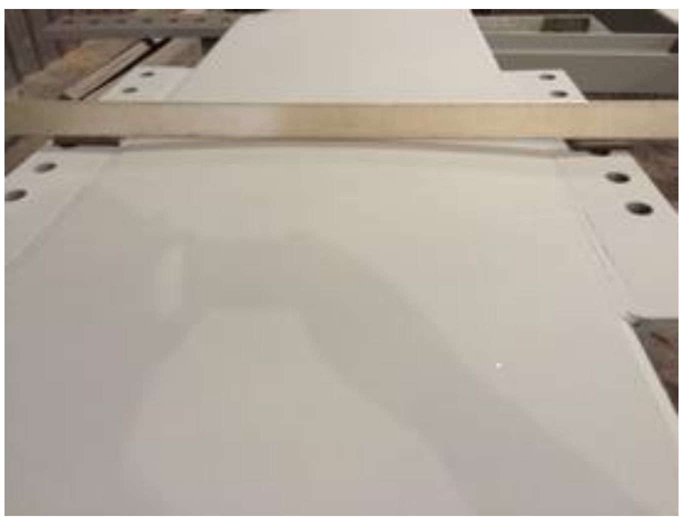


Figure 14: Tolerances in an HSS24x12x5/8

Round sections larger than 2 inches in Outside Diameter (OD) may have an OD differential of +/- 0.75%. Although the tolerance is small, it is unreasonable to expect a perfectly round section, especially if specifying a material other than A500 or A1085.

The straightness tolerance should also be considered, especially if the member is being "hidden" in a tall wall. The maximum "bowing" allowed in a member is $^1/_8$ inch per 5 feet of length. For example, due to this tolerance, it is not advisable to specify a 5-½ inch square tube inside a 6 inch stud wall as it leaves no tolerance for the straightness or convexity of the member.

2.8.4 Practical Tips for HSS

A500 rounds should be specified for structural applications rather than A53 pipe. A53 (ASTM, 2018b) is the standard specification for pipe steel that is coated with black lacquer, available in welded or seamless steel pipe. A53 is intended for use in mechanical and pressure applications like conveying steam, gas, or water. A500 is the standard specification for carbon steel structural tubing. A500 has a higher yield strength, lower cost, and is intended to be used for structural applications.

Labeling rounds is different for pipe versus HSS as well. A53 pipes are designated using a nominal pipe diameter in inches, plus one of three scheduled wall thickness (e.g. 8 inch STD). They are sized this way because A53 pipes must work with standardized fittings and valves. HSS rounds are designated more precisely with the outside diameter and wall thickness in inches and carried to three decimal places (e.g. HSS8.625x0.322).

It should also be noted there are several other non-structural specifications in the marketplace for round sections. Most notably, when a large round section is desired that exceeds the limitations in A500, often an Oil Country Tubular Good (OCTG) product is substituted. Careful research should be done to determine the suitability of these products as they are not intended for structural use. Weldability, yield strength, straightness, wall thickness, and tolerances of the member provided should all be investigated prior to acceptance.

2.8.5 Connections

Designing connections to an HSS or between two HSS members is very different than designing connections to an open section. Connections to an open section, although not preferred, can usually be easily reinforced if the member does not meet the localized strength requirements but this is very difficult to accomplish when connecting two HSS shapes. Whether the EOR is designing the connections, or delegating the design to another engineer, it is imperative that the HSS connections be considered when sizing the member's wall thickness to avoid costly reinforcement or revisions late in the construction process. The EOR should provide utilization ratios for main members when delegating connections.

2.8.5.1 Butt Welding

The corner radius of a square or rectangular HSS is dependent on its wall thickness. Therefore, when splicing two members that have the same outside dimensions, but different wall thicknesses, care should be taken when detailing this connection. The amount of bearing area or contact between the two sections will be limited to the flat dimension of the thicker section times the thickness of the thinner section. Weld length should be limited to the flat of the thicker section as well, shown below in Figures 15 and 16.

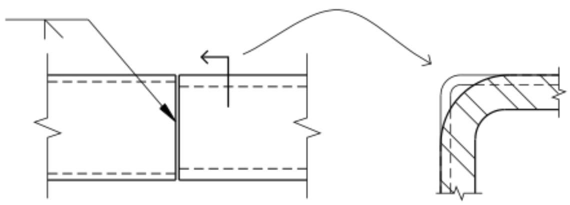


Figure 15: Splicing HSS of Equal Outside Dimensions but Unequal Wall Thickness

Figure 16: Splicing HSS of Unequal Wall Thickness

2.8.5.2 Welding at Corners

Since sections are cold rolled, residual stresses in the square and rectangular HSS corners are high. When possible, avoid welding to the radiused corners of HSS shapes. For example, when an HSS column is supporting a beam intersecting at a 45 degree angle, specify a single (or double) bent plate that can be welded to the flat of the adjacent HSS wall and bolted to the beam web, in lieu of welding a single plate to the corner of the tube.

2.8.6 Section Availability

The AISC Steel Construction Manual (AISC, 2017), and most design software applications, list all HSS that might be produced. This is not meant to imply all are readily available. It is advisable to check the Steel Tube Institute's Capability Tool to ensure that sections you are specifying are domestically produced, which can be found on the Steel Tube Institute website.

2.8.7 Galvanizing

Often HSS members may be exposed to weather and require hot dip galvanizing to prevent corrosion. If end plates on the HSS member are required, the EOR shall specify a vent or drain hole at each end of the member. This is to allow air in the member to escape so that the zinc can fill the cavity when dipped and for the zinc to drain out afterward. The EOR should specify both the location and the size of the hole permitted in coordination with the Galvanizer and Fabricator.

2.8.9 The 0.93 Factor

ASTM A500 has a wall thickness tolerance of +/- 10%. This tolerance is larger than what is specified for other sections that use the same resistance design equations. Therefore, section properties and wall thicknesses used in calculations must be reduced to take into account this tolerance. This is accomplished by reducing the nominal wall thickness of an A500 HSS by 7% as specified in AISC 360-10 and AISC 360-16 Section B4.2. It should be noted that all section properties in current versions of the AISC Steel Construction Manual (13th, 14th, and 15th Editions) have accounted for this reduction. A newer material specification, ASTM A1085/A1085M-15 (ASTM, 2015), tightens the wall tolerances such that the reduction is not necessary. This is indicated in AISC 360-16 Section B4.2 (AISC, 2016b) and section properties are published on the STI and AISC websites.

2.9 Dimensions

Accurate and clear dimensions are essential to successful steel construction. By establishing accurate and clear dimensions on the contract documents, the EOR can significantly influence the success of all the work that follows.

2.9.1 Accuracy

Utilizing the powerful capabilities of current design and modeling software can result in drawings with extremely accurate dimensions. However, this potential is dependent upon human competency and attentiveness. Accuracy early can result in accuracy downstream. Inaccuracy early guarantees inaccuracy downstream.

2.9.2 Clarity

In addition to accuracy, it is important that enough dimensioning is included on the contract documents to allow for the detailing to proceed without excessive RFI’s. This effort usually requires more attention as the complexity of the structural frame increases. Examples of increased complexity include skewed grid lines, discontinued grid lines, or multiple elevation changes at any one level. It is also often necessary to clarify the intent of a dimension string, such as “to face of channel”, or “to face of HSS”.

3.0 Steel and Concrete Constructability Considerations

The potential for connection and construction issues increases when more than one trade is involved on a project. In Colorado building construction, the following concrete and steel interfaces are common:

- Steel columns on concrete foundations.
- Steel girts to concrete columns.
- Steel beams to concrete core walls.

One challenge when different trades interface is understanding the differences in material tolerances. Steel fabricators and erectors are most familiar with steel-to-steel construction tolerances; discussed in the previous sections. While concrete tolerances vary based on construction type, the values are generally greater than those associated with steel construction. Concrete tolerances should be considered and may affect steel connection detailing. See Table 3 below for a general comparison of steel and concrete tolerances.

Table 3: Material Tolerances – Concrete vs Steel (ACI, 2010; AISC, 2016a)

Concrete			Steel		
Horizontal Tolerances	± 1"	H=20'	Column Plumb Tolerance		
	± 1"	H<84'			l/500
	± 6"	H>500'		Max +1", -2"	H<200'
Vertical Tolerance	± 1"		Vertical Tolerance of Horizontal Members		
				+ 3/16"	
				- 5/16"	
			(measured from upper finished splice line)		
Embed Location			Fabrication Tolerance	± 1/4"	
Vertical	± 1"				
Horizontal	± 1"				
Relative to face	± 1/2"				
Anchor Bolt Horizontal Location			Column Base PL Tolerance		
3/4" thru 7/8" Ø =	± 1/4"		Horizontal =	± 1/4"	
1" thru 1-1/2" Ø =	± 3/8"		Vertical (Bearing) =	± 1/8"	
1-3/4" to 2-1/2" Ø =	± 1/2"				

3.1 Concrete Tolerances

Generally accepted cast-in-place (CIP) concrete tolerances are discussed in detail in the ACI Specification for Tolerances for Concrete Construction and Materials, ACI-117 (ACI, 2010). The specification provides guidance on tolerances of horizontal out-of-plumbness, vertical elevations, thickness of elements, location of embeds, and placement of anchor bolts. Each of these factors will impact connections where steel and concrete elements meet. It is important to note that horizontal tolerances vary with building height. For example, Figure 17 shows the envelope of horizontal concrete tolerances, which increase with the height of a building. For building heights less than 83 feet, the concrete tolerance is about 1 inch. Compare that to typical steel erection tolerances of a ½ inch and it can be seen that concrete-to-steel connections require different treatment than steel-to-steel connections.

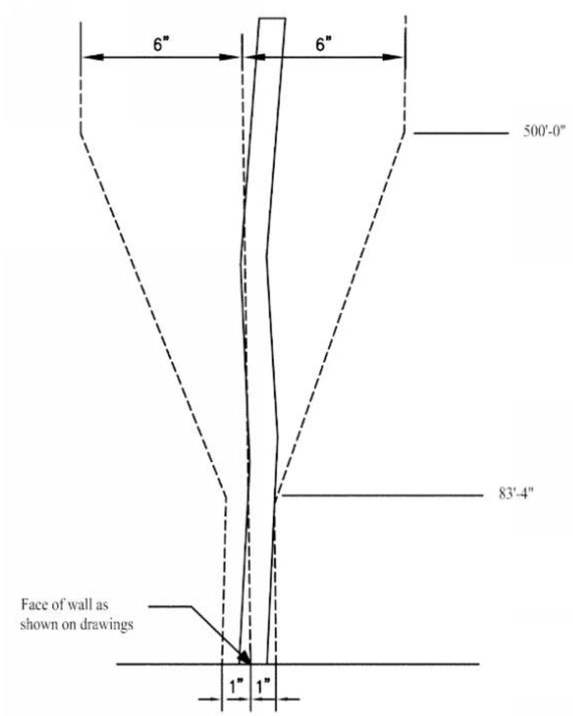


Figure 17: Horizontal Concrete Tolerance Envelope (ACI, 2010)

3.2 Column Bases

Steel columns bearing on concrete occur on almost every structural steel job. It is important to become familiar with commonly used column base plate details and to be aware of problematic details that can often lead to field issues. Column base connection design should take into consideration the variation of material tolerances as well as erection stability/safety.

3.2.1 Erection Stability

OSHA specifies that four anchor bolts/rods shall be provided at column bases to aid in initial erection stability. It also stipulates the foundation, base plate and anchor rods shall be designed to withstand the eccentricity of a 300 pound vertical load located 18 inches horizontally from the column face. However, it is suggested here as a general guideline that the column base be designed to withstand a 10 to 15 psf wind load applied to the face of the column. This wind load typically results in a larger overturning moment than the OSHA requirement and is a scenario frequently encountered during steel erection. Narrow anchor rod placement – such as placing rods inside the column flanges as shown below in Figure 18 – provides minimal overturning erection stability. By simply moving the anchor rods out beyond the column flanges and/or upsizing the anchor rod diameter, overturning erection stability can be greatly increased for added safety.

Figure 18: Narrow Anchor Rod Pattern Causing Erection Stability Issues

3.2.2 Base Plate Geometry and Anchor Rod Layout

When setting anchor rods, the tolerances of Section 7.5.1 of the AISC 303 COSP (AISC, 2016a) shall be met. If anchor rods are set to the required tolerances, and base plates are detailed with the special oversized anchor rod holes as shown in Table C-J9.1 of the AISC Specification Commentary (AISC, 2016b), common field issues can be reduced. Another simple way to avoid field issues is to keep anchor rod patterns symmetrical. In doing so, placement confusion is avoided and the possibility of anchor rods being accidently installed in a rotated position is eliminated. Additionally, it is ideal to reduce the number of different patterns so that anchor rod templates can be minimized, reducing field confusion and misplacement of anchor rods.

3.2.3 Base Plates of Similar Size to Concrete Pier/Foundation Wall

When column base plates are of similar size to a concrete pier or foundation wall below, it is especially important to pay close attention to anchor rod layout. Anchor rods placed near the concrete edge can often foul with rebar layout. When locating rods in these instances, keep in mind the installation tolerance of the concrete as well. Furthermore, anchors set near a free edge may not fully develop the tensile strength of the rod due to shear cone tear-out such as shown in Figure 19. This in turn reduces overturn capacity and possibly jeopardizes the erection stability of the column. Often, additional shim packs near the corners of the base plate can be utilized to increase the overturning stability of the column; however, if the concrete pier beneath is of similar size to the base plate, concrete spalling can prohibit the placement of shims at the corners. It's best to avoid these issues by enlarging the pier size, moving rods further inside the pier without compromising overturning stability with a too-narrow pattern, or shortening the bottom column shaft that bears on that pier or wall.

Figure 19: Base Plate of Similar Size to Concrete Pier with Anchor Rod Failure

3.2.4 Shear Keys

Shear keys are another item that often cause issues in the field. These can prevent the placement of shim packs below the column base due to spalling at the shear key opening. Moreover, shear key openings are often formed incorrectly and can foul with the plates, causing delays in steel erection. When possible, it is best to avoid shear keys and utilize anchor bolts or weld plates to transfer force from the steel superstructure to the foundation. It would also be preferred to lower the column base elevation, embedding it into the slab. When no other options exist and shear keys must be utilized, verify there is adequate grout spacing below the base plate to accommodate leveling nuts and washers.

3.2.5 Embed Plates

A column base detail where the base plate of the column rests directly on top of and is welded to an embed plate, similar to the detail shown in Figure 20, creates a range of installation issues. Grout is a key component that allows columns to be erected to AISC Code of Standard Practice (AISC, 2016a) plumb and vertical tolerances. Without a grout space, field adjustment opportunities are virtually eliminated. A base plate shop-welded to a column often "cups" upward due to the one-sided welding operation. Additionally, the base plate can be off the square axis of the column shaft but still within normal fabrication tolerances. These variations as well as the embed plate elevation and level placement tolerances would make it impossible to set the column to the required AISC COSP erection tolerances. For example, assume a 50 foot tall column with a 20 inch square base plate sitting directly on and welded to an embed plate. If the base plate and/or embed plate are only a $^1/_{16}$ inch out of level over the 20 inches width, the top of the column would be 1-$^7/_8$ inches out of plumb, which is greater than the 1 inch the AISC COSP (AISC, 2016a) plumbness tolerance allows. This demonstrates how critical it is to allow for field adjustments under column base plates.

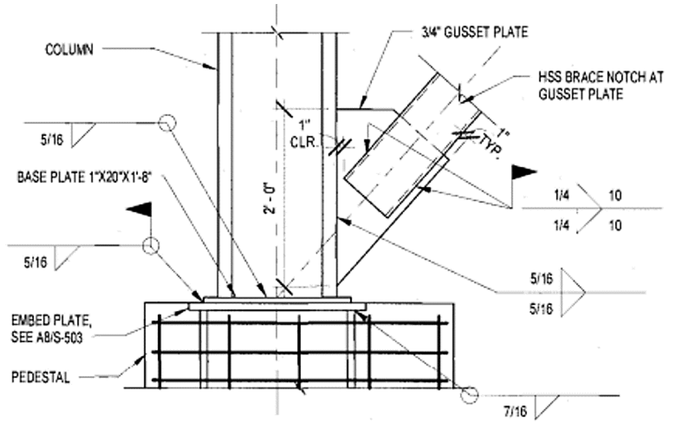


Figure 20: Column Base Sitting Directly on Embed Plate

If a column base plate detail incorporates a grout space between the embed plate and the column base plate, there are additional considerations to keep in mind. First, anchor rods welded to the top of the embed plate should not be pre-installed so that placement/layout issues are prevented. The erector should lay out and field-install these anchors. Second, when designing embed plates with weld-on anchors it is important to establish a direct load path from the anchors through the embed plate and into the headed studs or bars below. The embed plate is often too thin to transfer anchor tensile loads to embedded studs if there is not a direct load path. Figure 21 shows weak axis bending in the embed plate caused by tensile forces applied to the anchor rods during erection. This can have detrimental effects on the erection stability of the column.

Figure 21: Embed Plate Failure from Wind on Anchor Rod Tensile Loads

3.2.6 Anchor Rod Fixes

Occasionally anchor rods are damaged, are not set to the correct tolerances, or are set incorrectly, requiring field fixes to the anchor rods or base plate prior to erection. If anchor rods are not set within horizontal tolerances a common practice is to slot the base plate holes to shift the base plate back over the rods and move the column back on grid. This can become problematic if the base plate has been detailed with tight clearances to the column flanges and/or edges of the base plate. When nonsymmetrical anchor rods are set in a rotated position the base plate may need to be removed, rotated, and re-installed; however, it must be verified that the rotated position of the anchors do not foul with the column flanges. When the rotated rods foul with the column it's often required to remove the extruding portion of the rods and install epoxy anchors such as shown in Figure 22; or cut the base plate from the column and re-weld it in a rotated configuration. Once again this is an easily avoidable field issue by simply designing base plates with symmetrical anchor rod patterns. Anchor rod surveys completed early during construction may allow for any required base plate modifications to be completed in the shop fabrication process.

Figure 22: Epoxy Anchor Replacement for Anchors Installed in Rotated Positions

If anchor rods are damaged or set with inadequate projection, the use of a coupler may be required. Additional plate fillers may be required when utilizing couplers, see Figure 23. Cutting rods and welding on new extensions with a groove weld is another possible option when rods are of weldable material. Figure 24 shows a possible groove weld option for extending anchor rods. As the Designer, it's advisable to consider a larger diameter anchor of a more weldable grade (as applicable) to facilitate welded field fixes when issues arise.

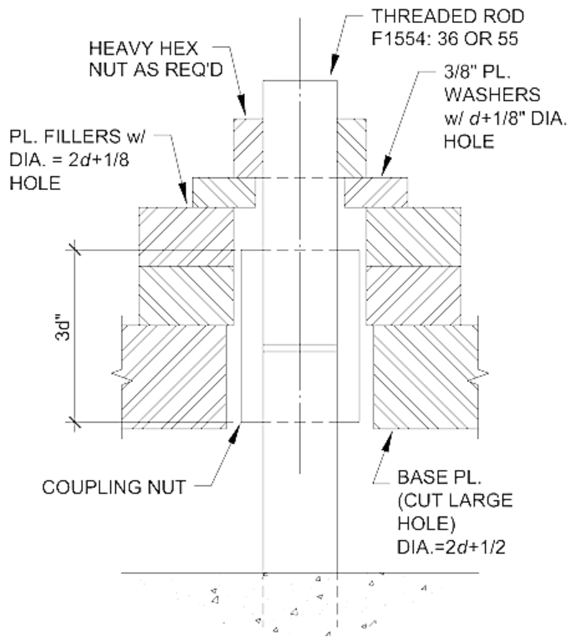


Figure 23: AISC Proposed Coupler Fix for Extending Anchor Rods when Projection is too Short (AISC, 2006)

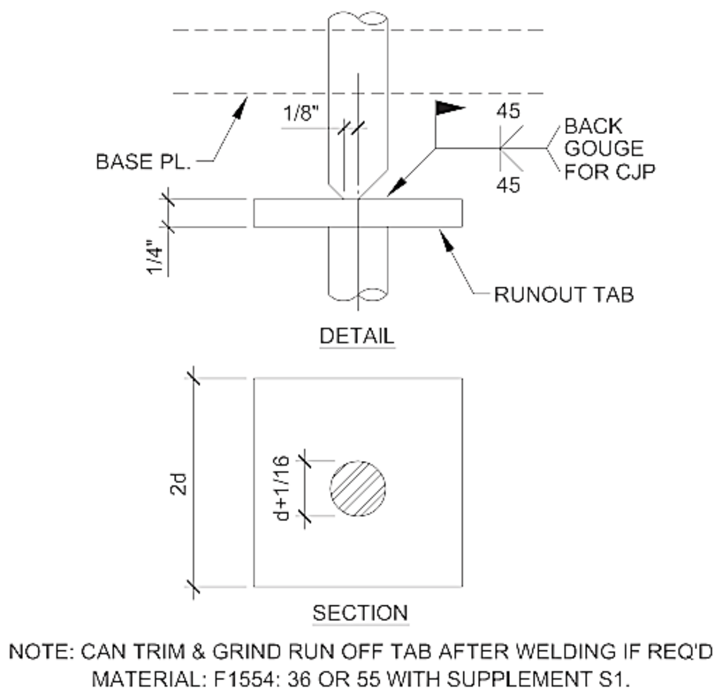


Figure 24: AISC Proposed Welded Option for Extending Anchor Rods when Projection is too Short (AISC, 2006)

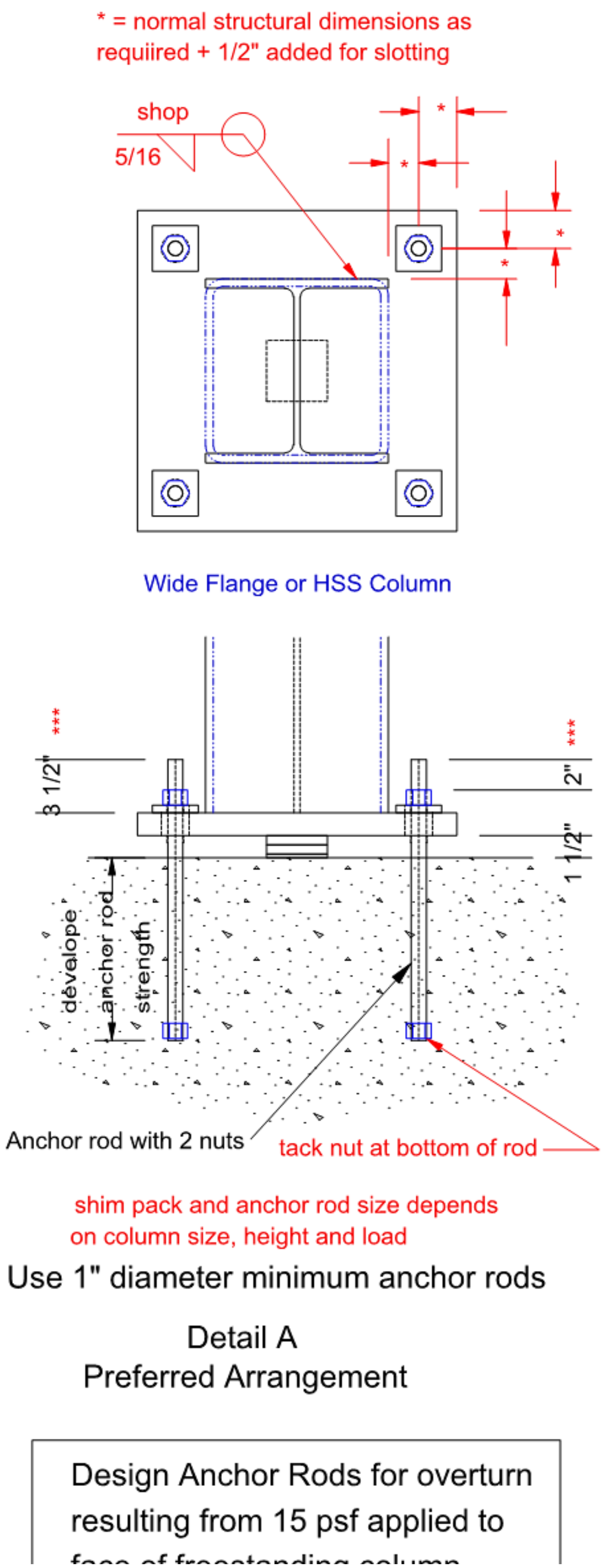


Figure 25: Erector Preferred Base Plate Details

3.2.7 Ideal Anchor Rod and Base Plate Arrangements for Erection Stability

In considering all the above scenarios the ideal base plate and anchor rod arrangement is a base plate with a minimum of four anchor rods, located outside the column flanges, in a symmetrical layout. A 2 inch grout space should be detailed below the base plate allowing a single shim pack to be set centered under the column base plate. This allows the erector to set the shim pack to the proper elevation prior to column erection. A single shim pack is also the simplest method of column plumb adjustment. The concrete pier should be sufficiently wider than the column base plate to prevent any spalling issues at shim packs when four corner shims are required, and the anchors should be far enough from concrete edges that the tensile capacity is not reduced and rebar conflicts are avoided. The designer may push for the Fabricator to provide anchor rod templates with anchor rod deliveries to aid installation of the anchors and reduce field misplacement. See Figure 25 for an ideal column base plate and anchor rod detail.

3.3 Beam Embed Plates

The most common approach to supporting a steel beam with a concrete element, is to cast a plate with rebar or stud anchorage into the concrete element. When the concrete forms are removed, that “embed plate” provides a weldable surface on which to support the steel beam. Post-installed anchors can be used as an alternative to embed plates to attach plates or angles to a hardened concrete surface. However, post-installed anchors do not provide the same strength as cast-in-place anchors or embeds and are therefore generally more expensive. Post-installed anchors are best used for special conditions or field fixes.

Per ACI 117 (ACI, 2010), the tolerance for locating an embed plate is +/- 1 inch. For the Designer or Detailer, a good practice to accommodate this tolerance is to provide oversized embed plates. Adding a few inches to the width and height of an embed plate increases the chances that the welding surface requirements are met, even if the location of the plate is not perfect. When oversizing an embed plate, the Engineer must also verify the anchorage is designed for the proper eccentricity, assuming the plate is loaded a few inches off center.

3.4 Beam Length

Even assuming the embed plate is perfectly located, relative to the formwork, there are still tolerances with out-of-plumbness and concrete element thickness to consider. Per ACI 117 (ACI, 2010), for a building under 83' tall, the tolerance for horizontal out of plumbness is +/- 1 inch, see Table 3 for reference. As the building height increases, this tolerance also increases, up to 6 inches maximum. For the common example of a steel beam supported by a perpendicular concrete wall or concrete beam, this tolerance can impact the connection and the steel beam length. The beam should be detailed short, and the connection be detailed such that it accommodates the range of construction tolerance. For a building under 83 feet, with +/- 1 inch of tolerance, the end connection needs to accommodate 2 inches of possible variation (1 inch short or 1 inch long). Consider another ¼ inch of steel fabrication tolerance, and a connection that accommodates 2-½ inches of possible variation is necessary. Furthermore, the tolerance for concrete element thickness should also be considered. The thickness of a concrete beam or wall, with a nominal thickness more than 12 inches, has a dimensional tolerance of +/- ½ inch. In practice, connections are not detailed for the extreme case. Contractors should plan their work to minimize these tolerances, and Engineers should detail connections to accommodate realistic variations. Including this topic in the preconstruction meeting further improves the chances for successful construction.

3.5 Beam Connection

Steel beam-to-concrete connections need to provide adjustability. For example, a difficult detail would show a steel beam welded directly to an embed plate. A detail like this should almost always be avoided. While it may be architecturally pleasing, perfect fit-up isn't achievable in a world of standard construction tolerances. When possible, steel-to-concrete connections should be made with plates or angles and should be field welded or bolted. Note that +/- 1 inch of concrete tolerance exceeds the travel of a typical long-slotted hole. Bolted connections may require special slot lengths.

Figure 26 shows a girt connection with field welds and a maximum gap of 1-¼ inches. This suggests the beam should be fabricated $^5/_8$ inch short of the intended embed plate location, providing a tolerance of $^5/_8$ inch. If the Detailer wants to provide +/- 1 inch of tolerance, to match the concrete out-of-plumbness, then the maximum gap should be detailed at 2 inches. This also requires the Engineer to design for a larger eccentricity.

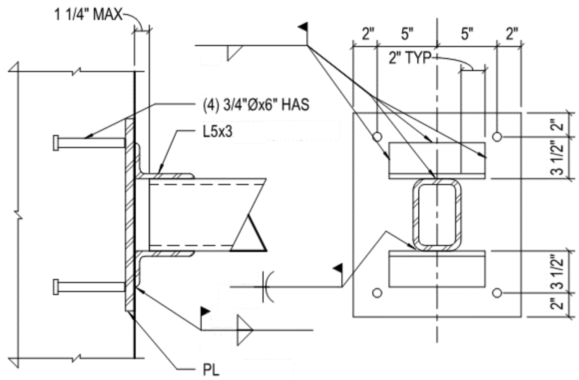


Figure 26: Possible Girt to Embed Connection

See Figure 27 for a different concrete embed detail concept. In this case, the size of the gap, and the intended beam length are dimensioned more explicitly. The detail provides about 1 inch of tolerance. It is up to the Engineer to choose a gap they think will provide adequate tolerance, while balancing design efficiency. The Contractor should review these details and suggest changes when beneficial to the project.

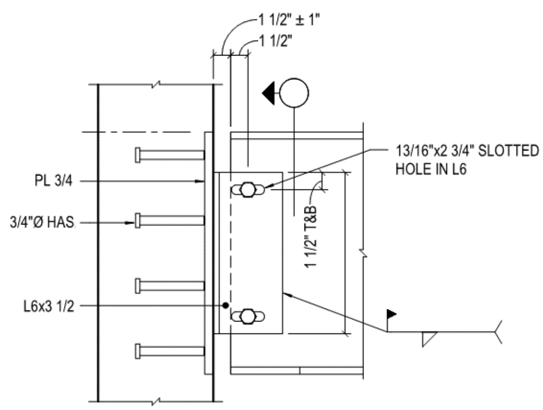


Figure 27: Typical Beam Embed Connection

3.6 Welded-Bolted Double Angle

In steel-to-steel connections, welded-welded double angle connections are used for heavily loaded beams with large reactions. Designers should be cautious specifying the same connections at concrete embed plates. The design of the angle welds at embed plates is different due to higher eccentricities, and results in lower capacities.

4.0 Steel and Masonry Constructability Considerations

The interface between steel and masonry is very common in low- to mid-rise construction such as schools, hospitals, prisons, warehouses, and other “big box” projects. Masonry walls may be used in bearing to support gravity loads from floors or roofs and/or as shear walls to resist lateral loads. Steel elements that often connect to masonry walls, columns, and pilasters include structural steel beams, open web steel joists, trusses, and metal decking.

4.1 Tolerances - General

As is the case with other interfacing trades, the relative tolerances between steel and masonry are not always compatible. Masonry construction is installed on site using modular blocks that are interconnected using mortar joints, grout and rebar. Location, plumbness, and elevation are all subject to the quality of the installation. Masons are able to adjust element location throughout construction by slightly altering the thickness of bed and head joints and the actual placement of individual blocks. Structural steel is a primarily shop-fabricated system with tolerance allowances provided by slotted holes and field-adjusted connections.

Table 4: Material Tolerances – Masonry vs Steel (ACI, 2011; AISC, 2016a)

Masonry			Steel	
Variation from plumb	± 1/4"	in 10 ft	Erection Tolerance	± 1/2"
	± 3/8"	in 20ft		
	± 1/2"	max		
	± 3/4"	Non-LB		
Location of Wall	± 1/2"	in 20 ft		
(horizontal)	± 3/4"	max		
Top of Wall	± 1/4"	in 10 ft		
(vertical)	± 1/2"	max		
Embed Location			Fabrication Tolerance	± 1/4"
Vertical	± 1"			
Horizontal	± 1"			
Relative to face	± 1/2"			
3/4"Ø Anchor Bolt Location				
In plan	± 1/4"			

One instance of the impact of these tolerances is the case where a beam may span between two masonry walls. It is possible that each wall could be ½” from its theoretical location in opposite directions. This would result in each beam end having its bearing location short by a ½ inch, resulting in the beam being a total of 1 inch too short. These tolerances not only impact the feasibility of a connection, but the eccentricity of load from the masonry member’s centerline may also be underestimated. If the design of the wall is tight, this added eccentricity could overstress the wall. It is important to provide sufficient bearing length and pocket depth to allow for variance in wall location and beam length, allow for additional eccentricity in the design, and document what the allowable eccentricity is on the design documents. If both ends of a beam bear on masonry that extends above the level of the steel, erection of the beam may be a challenge as the beam would need to be placed through the pocket on one end in order to set the other end. If beams are required in the same plan location on opposite faces of a masonry wall, consider running one beam over the wall and connecting the other beam to the continuous beam as shown in Figure 28.

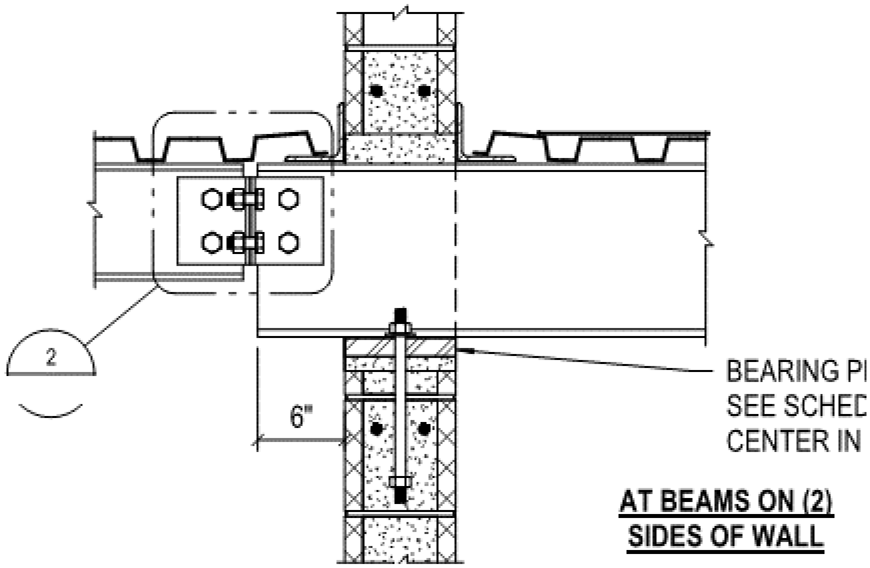


Figure 28: Connection of Beams on Two Sides of Wall

4.2 Lintels

Another common steel-to-masonry interface example is a steel lintel used in a masonry wall to support the wall above an opening such as a door or window. The steel shape must fit within the plane of the wall. If the wall is nominally eight inches wide (actually 7-$^5/_8$ inches +/- ¼ inch), the lintel must be less than 7-$^5/_8$ inches wide. Steel lintels usually require a bearing plate at either end that, if properly detailed, will be as wide as or narrower than the wall and will have a thickness less than or equal to the mortar joint thickness (typically $^3/_8$ inch). If headed studs or rebar are welded to the top of the steel lintel, care must be taken to ensure the spacing and location corresponds with the masonry block cell spacing.

4.3 Grout Drop

The interface between CMU walls/columns and the elements that bear on them is also affected by “grout drop” – a phenomenon in masonry construction where the elevation of the grout within the masonry units lowers shortly after it is placed due to bleed water from the grout being wicked into the masonry units. While easily addressed by the mason, it can add a level of complication in maintaining the precise position of embedded objects like anchor bolts and embed plates.

4.4 Thermal Expansion

Accounting for a variation in the elevation of beam bearings or the top of masonry walls can be accomplished by specifying a lower elevation and adding leveling shims and grout under a beam bearing plate. If this type of detail is utilized, one may also easily avoid "locking" the beam into the masonry before the space is conditioned to avoid thermal expansion and contraction of the steel. Thermal effects can easily damage the masonry at a connection if the steel is "locked in"; especially where the masonry construction is rigid in the direction of the beam, as might occur at a corner, intersection or the end of a wall. To avoid this, specify a threaded stud welded to the bearing plate and bolted to the beam in a slotted hole for the temporary erection condition. Once the building has been enclosed, a field weld from the beam to the bearing plate completes the connection. The beam pocket may be grouted at this time. See Figure 29.

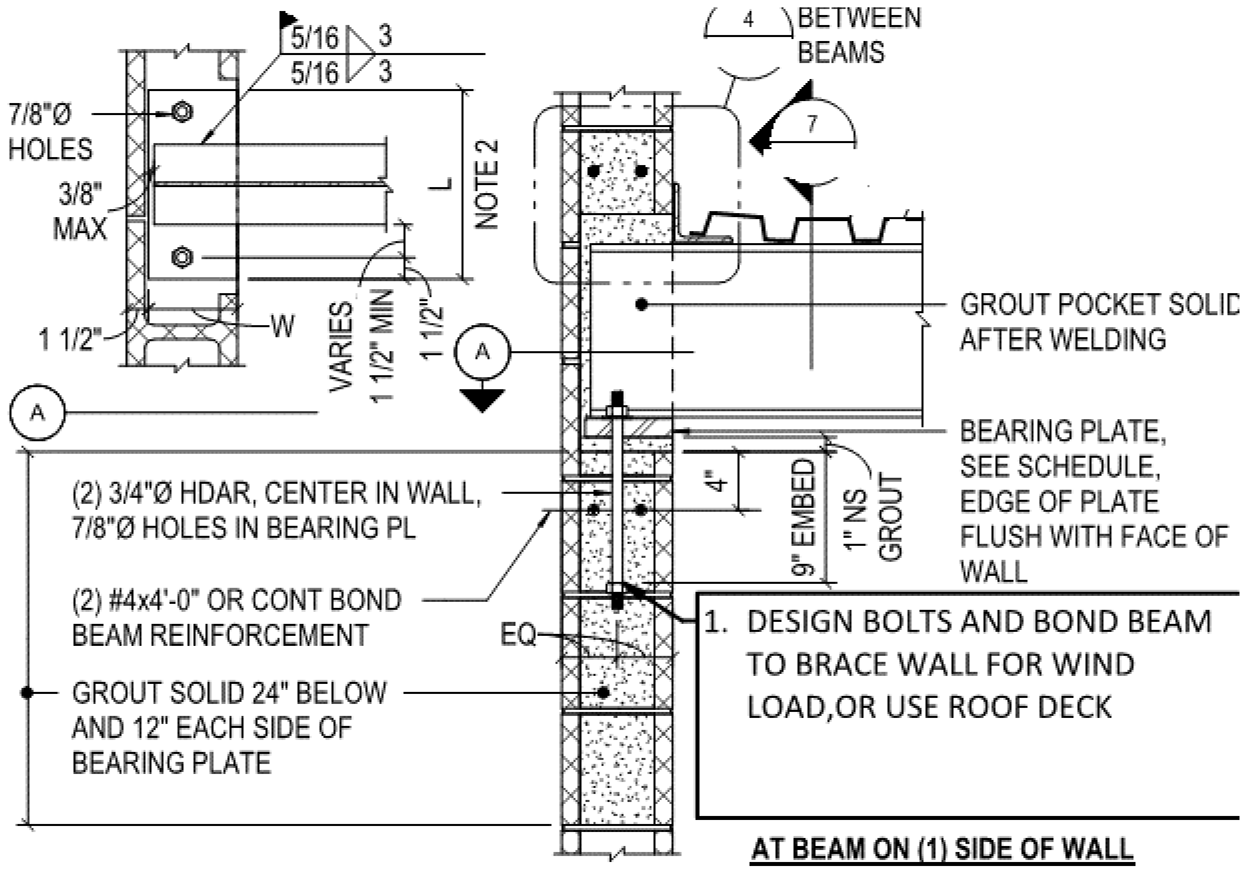


Figure 29: Typical Beam Bearing on Masonry Wall

4.5 Steel & Masonry Columns

Occasionally, a steel column is wrapped in masonry in lieu of using the masonry itself as the compression element. This may ease some construction obstacles such as scheduling and material tolerances, however several factors must be considered. The column must be small enough to fit inside the masonry cavity, while taking into account the tolerances for both the steel and the CMU. It's also necessary to consider the column base plate and bolts and their interaction with the bottom of the masonry wall and its bearing condition.

4.7 Embed Plates

Similar to cast-in-place concrete construction, the Designer should consider reasonably oversizing embed plates to allow for tolerances. Along with reasonably oversizing, the Design should consider designing to include horizontal/vertical tolerances. It is also beneficial, where possible, to locate them such that they course with the CMU blocks, i.e. every 8 inches. A final consideration is to make the embed plates symmetric to mitigate field errors.

5.0 Open Web Steel Joists and Steel Deck Constructability Considerations

5.1 Steel and Concrete Masonry Units

When steel joists bear on masonry, the reaction point is critical for both the masonry design and the joist design. The reaction point must be over the required steel bearing plate in order to avoid creating an eccentricity in the wall and a moment in the joist top chord end panel. The eccentricity in the wall needs either to be avoided or accounted for in the design.

Joist bearings for K-Series joists are 4 inches long. At least 2-1/2 inches of the 4 inches must bear on steel. For LH, DLH, and Joist Girders, bearings are 6 inches long. Of the 6 inches, the minimum bearing length is given in Table 5 below. Any extensions of the bearing angles are not part of the joist bearing and cannot be included as part of the bearing length. The joist base length is defined as end-to-end of the joist bearings. The reaction point is the intersection of the neutral axes which occurs 2 inches in from the end of the base length (end of joist bearing). This is illustrated in Figures 30, 31, and 32, below. Extending the bearing materials beyond the standard bearing length does not change the reaction (intersection) location.

JOIST SECTION NUMBER[1]	STANDARD CLEAR BEARING LENGTH	MINIMUM BEARING LENGTH ON STEEL
K1-12	4” (102 mm)	2 ½” (64 mm)
LH02-06	6” (152 mm)	2 ½” (64 mm)
LH07-17, DLH10-17, JG	6” (152 mm)	4” (102 mm)
DLH18-25, JG[2]	6” (152 mm)	6” (152 mm)
(1)Last digit(s) of joist designation shown in Load Table. (2) **Joist Girders** with a self weight greater than 50 plf (0.73 kN/m).		

Table 5: Minimum Bearing Length, SJI Table 5.4-1 (SJI, 2015)

The first step in avoiding eccentricities in CMU walls is using the correct dimensions. Actual block dimensions should be shown in details (e.g. 7- $^{5}/_{8}$ inches, not 8 inches). SJI Specification Section 5.4.1.3 (SJI, 2015) states the steel bearing plate shall be located not more than ½ inches from the inside face of the wall and continues that “special consideration” shall be given if it is not. Multiple tolerances for the masonry, construction errors, and the +/- ¼ inch allowance for the joist length can also create conditions that cause eccentricities. If the plate is not within ½ inch from the interior face of wall due to preference or design details that allow for tolerances, the reaction point must be moved further onto the wall when detailing the joists and increasing the bearing plate dimension. The reaction point distance from the interior face of wall needs to be shown on the structural details.

JOIST SECTION NUMBER[1]	STANDARD BEARING SEAT DEPTH	STANDARD CLEAR BEARING LENGTH	SPECIAL MINIMUM BEARING SEAT DEPTH[2]
K1-12	2 ½" (64 mm)	4" (102 mm)	0.6 x (RP + 2 ½" (64 mm))
LH02-17, DLH10-17	5" (127 mm)	6" (152 mm)	0.6 x (RP + 4" (102 mm))
DLH18-25	7 ½" (191 mm)	6" (152 mm)	0.6 x (RP + 4" (102 mm)) + 2 ½" (64 mm)
JG	7 ½" (191 mm)	6" (152 mm)	RP + 4" (102 mm)

(1) Last digit(s) of joist designation shown in Load Table.
(2) RP is equal to the distance the reaction is to occur from the face of the wall or leading edge of support member. The equation is not applicable for the high end of a sloped joist or Joist Girder.

Table 6: Special Minimum Bearing Depth, SJI Table 5.4-3 (SJI, 2015)

Moving the reaction inward from the inside face of the masonry wall can only be achieved by moving the intersection (again, not by extending the joist bearing materials). This can result in fouling between the wall and the first diagonal joist web member. The solution is to lower the wall in order to provide more clearance for the web to clear. This is done by increasing the joist bearing depth and adding a note on the structural drawings identifying where the reaction is to occur. See Figure 32. Guidance for this is provided in Table 6 (SJI, 2015). The minimum joist bearing depth for joists bearing on masonry should be 3-½ inches. Joists sloping $^{3}/_{8}$ inches or more will require additional bearing seat depth.

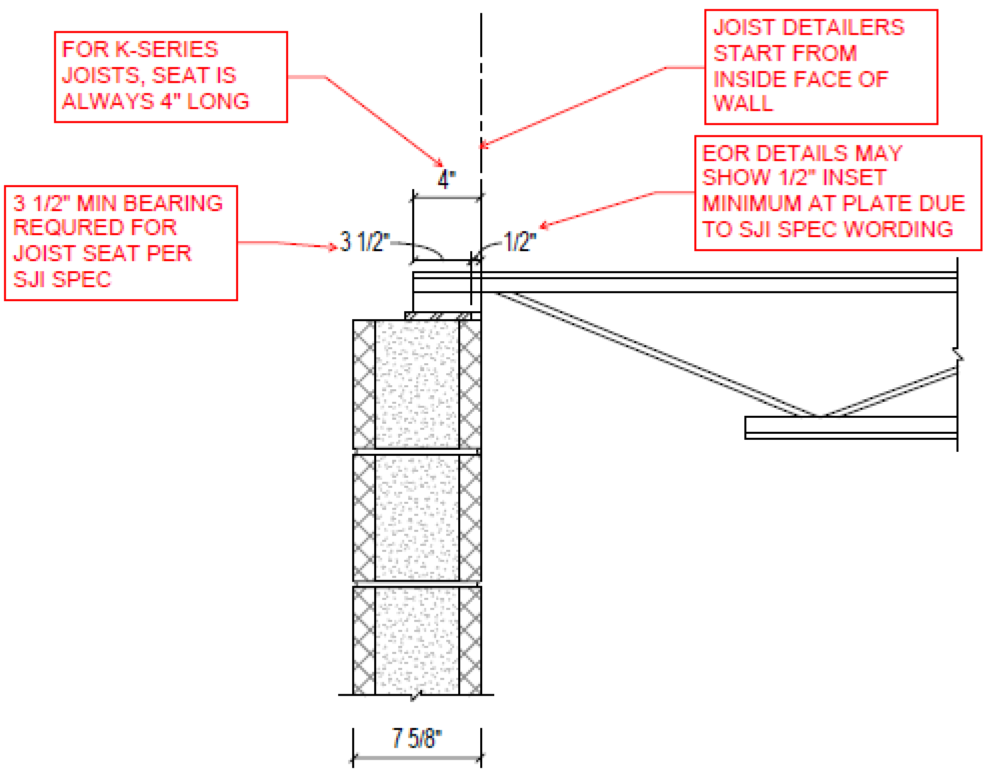


Figure 30: Joist Bearing - As Detailed

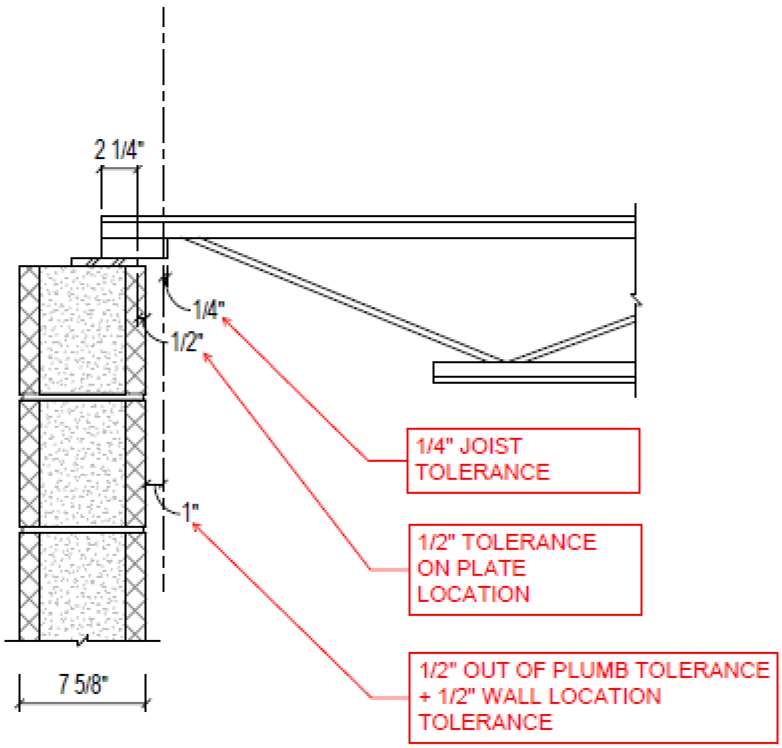


Figure 31: Joist Bearing - As Built Within Tolerance

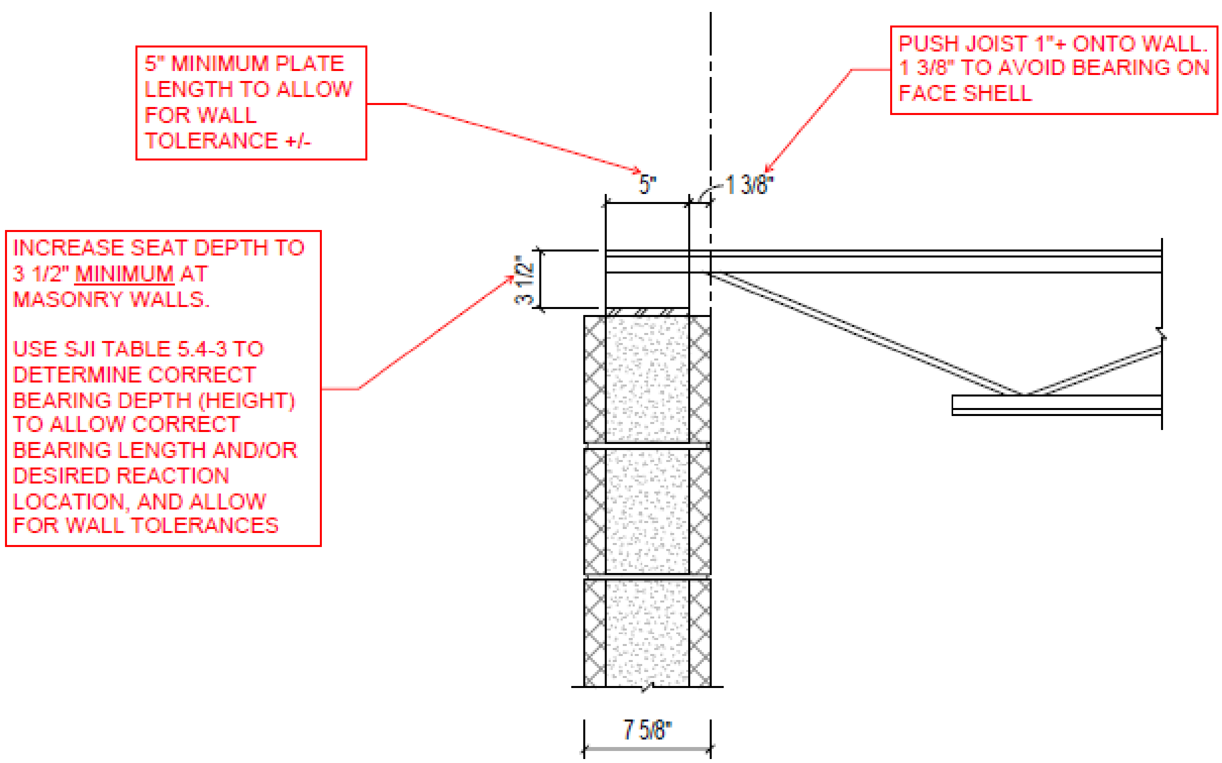


Figure 32: Joist Bearing - Suggested

5.2 Camber in Roof Members Adjacent to Hard Points

It's important to know that open web steel joists always have standard camber unless specified otherwise. Standard camber is a radius of 3,600 feet for joists up to 100 feet and L/300 for longer. Any other camber, including "No Camber", is a special camber.

Joists with non-standard or no camber are more costly for joist manufacturers to fabricate, however this additional cost is well spent when field issues are avoided by strategically modifying the joist camber. When joists with standard camber are adjacent to other framing that is not cambered, such as walls and steel roof beams or trusses, the camber can prevent the deck from bearing on the non-cambered member, see Figure 33. Other examples of issues due to joist camber are when bay sizes change and the joists are near a column or a bearing wall that is not near the end of the joist, see Figure 34. Good examples are a school gym with an adjacent stage that is shorter, a major jog in the wall such as two adjacent gyms of different size, or when a longer bay is split into two bays part-way through the building.

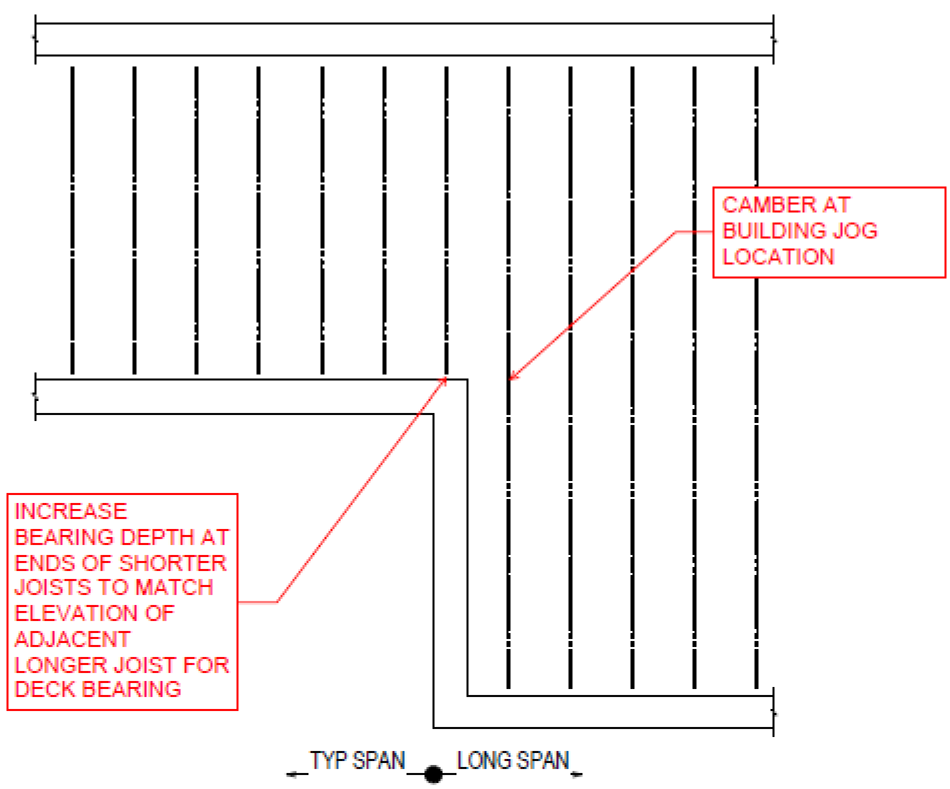


Figure 33: Camber - Jog in Bearing Wall

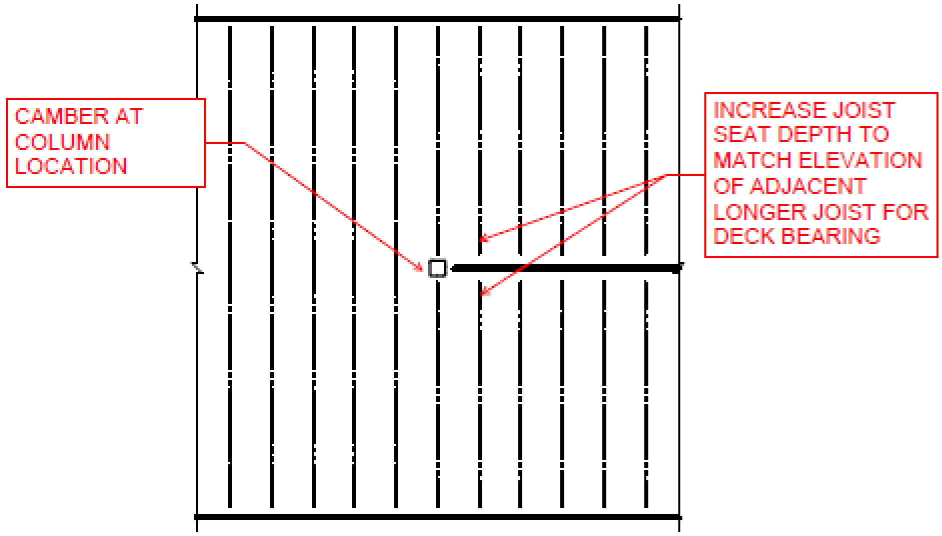


Figure 34: Camber - Hard Support

In the case of walls, beams and trusses, suggestions include:

- Allowing the deck hats to be cut. This requires attaching the deck to both chord angles and requiring the joist manufacturer to provide a load path between the chords
- Decreasing both the camber and live/snow load deflections in order to be close enough to matching elevations to bend the deck down to attach.
- Attaching the deck to a ledger that has been installed curved to match the standard camber of the joist.

In the case of hard points such as columns and large wall jogs, adjust bearing depth of shorter joist(s) to match expected residual camber of the adjacent longer joist at the point.

Cases where "No Camber" should be specified include joists with non-parallel chords with a pitch of ≥ 3":12" and for joist girders supporting joists sloping more than 1":12", see Figure 35.

Note that for all these conditions, differential deflection under any post-construction loads needs to be considered.

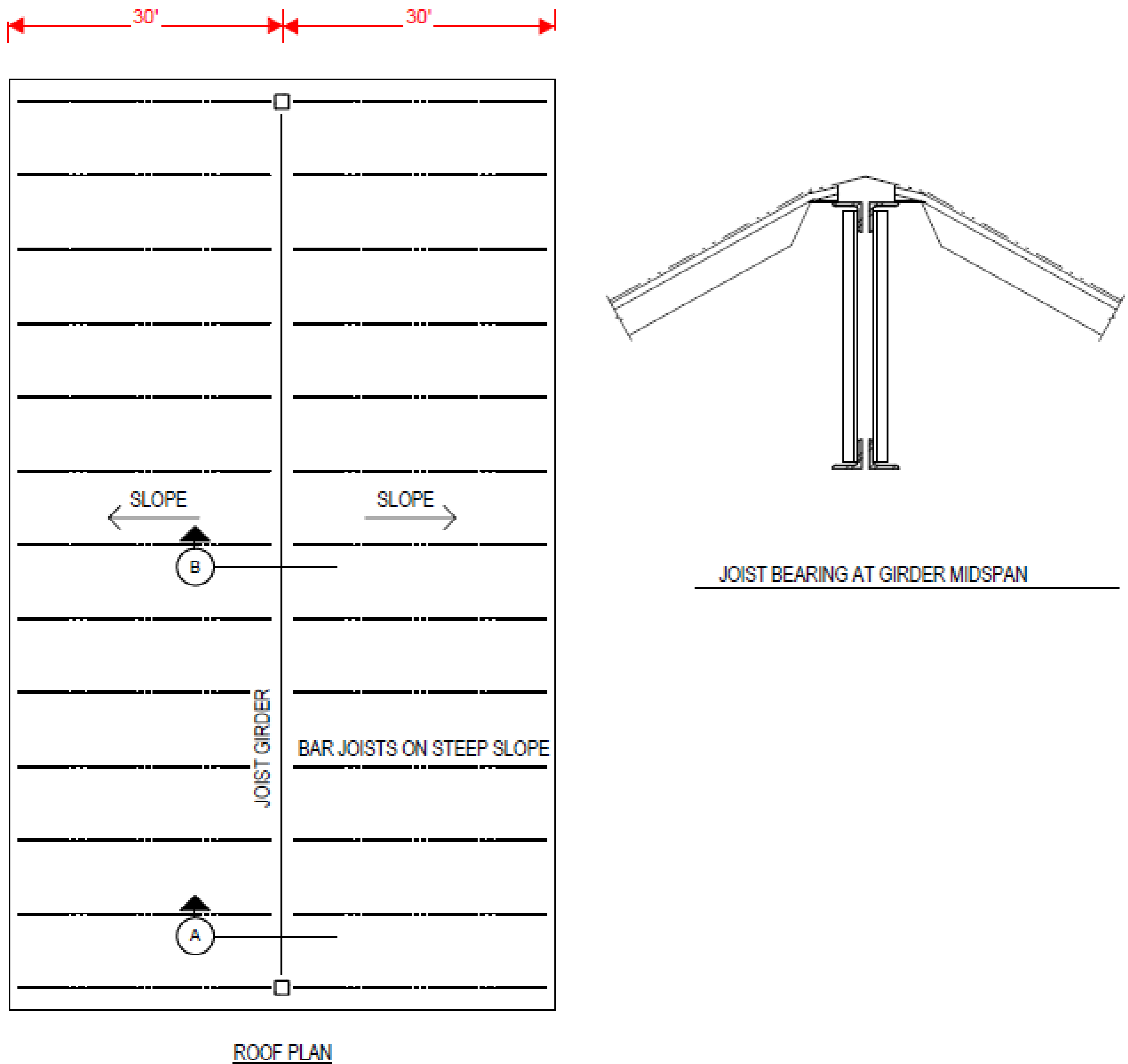


Figure 35: Cambered Joist Girder at Steep Roof

5.3 Exterior Beam Flange Widths

Bearing floor deck on an exterior beam can create conflicts with other materials also bearing on and attached to the beam. These include deck closures, shear studs, bent plates, other anchors or reinforcing for attachment of curtain walls, etc. These combined with the deck minimum bearing lengths of 2 inches to 3-½ inches require enough beam flange width to accommodate it all.

The deck must bear flat on the beam in order for the shear connector to be welded to the beam properly. Bearing on a bent plate is not feasible since a load path for shear from shear connectors is not available, see Figure 36. Assuming a minimum bearing length for the bent plate of 2 inches and 2 inches of bearing for the deck, a ¼ inch fillet weld for the bent plate already consumes more than a 4 inches wide beam flange. Adding tolerances for beam sweep, steel fabrication and erection, and deck length compounds the problem, see Figure 37. Since the edge angle is normally installed first, the deck can end up bearing on the plate or the weld instead of the beam. Deck erectors agree, “It happens all the time”. The typical solution heard is to “pound the deck down”, however, this damages and reduces the shear capacity of the deck.

Consider 5 inches as a minimum for the perimeter beam flange width; preferably 6 inches wide. Also consider non-composite beams at the below details to allow less congestion and conflict with other attachments such as welded horizontal dowels and curtain walls.

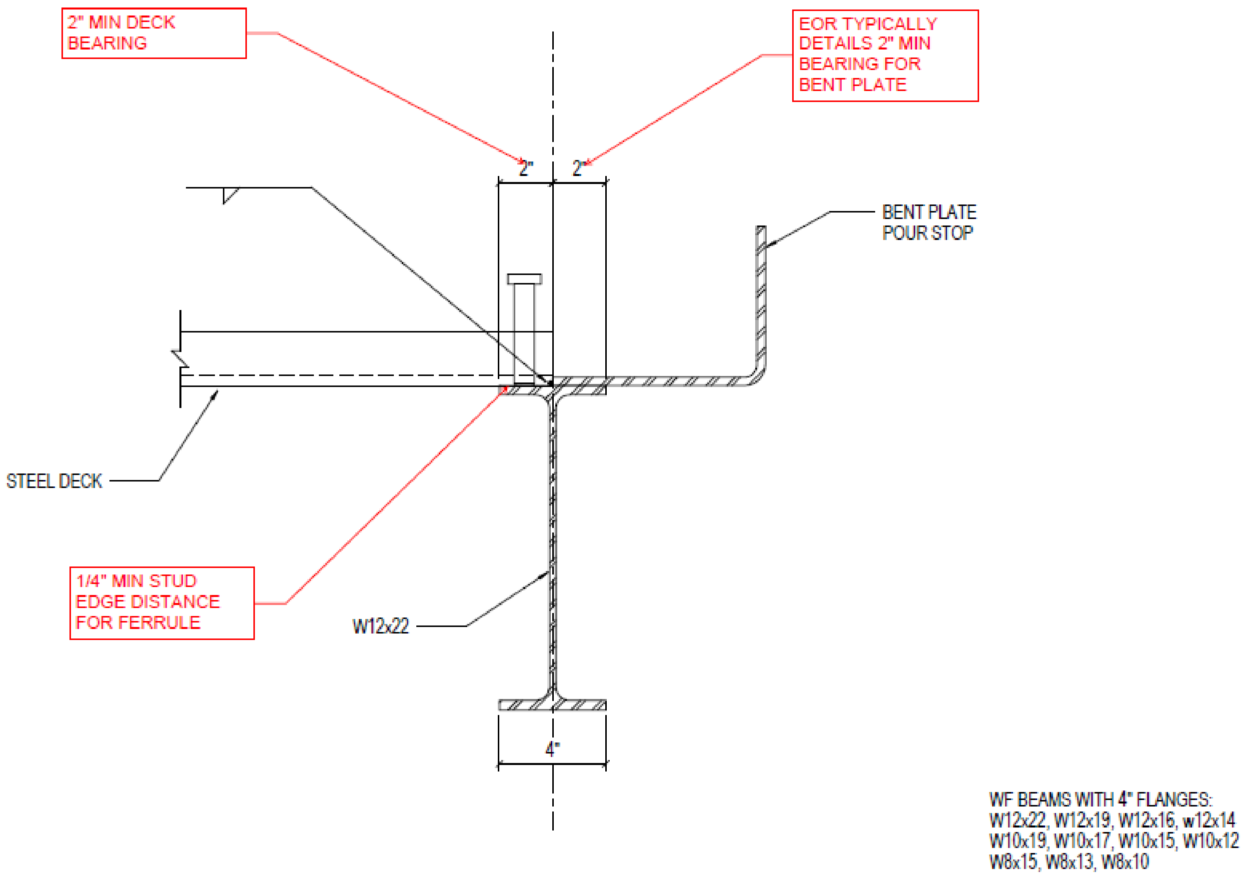


Figure 36: Deck Bearing at Composite Perimeter Beam - As Detailed

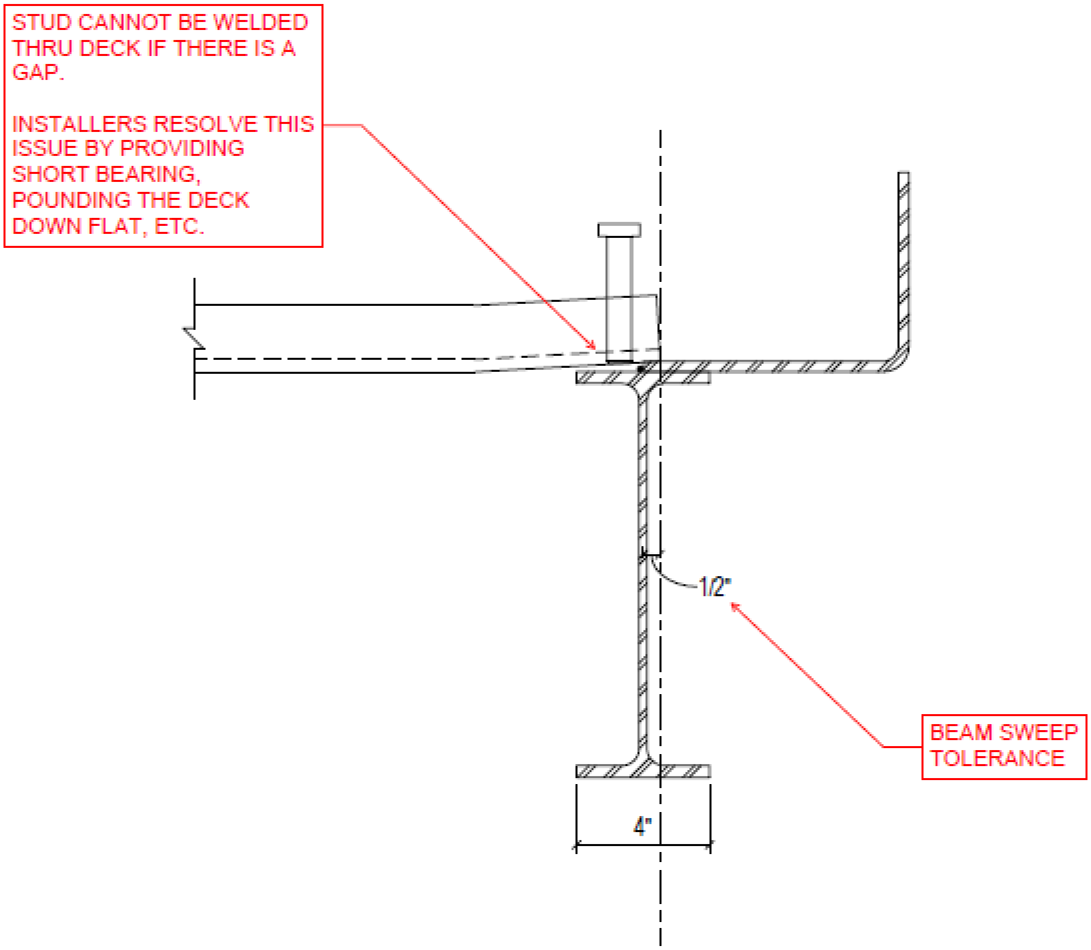


Figure 37: Deck Bearing at Composite Perimeter Beam - As Built Within Tolerance

6.0 Weld Clearance – An Illustrated Example

On the subject of weld clearance, there is less guidance and fewer recommendations than would be expected. There are recommendations in the AISC Specification (AISC, 2016b), but the best, most succinct advice is this from H. M. Priest: “The welder must be able to see his work clearly” (Priest, 1943).

Figure 38: Welder Attempting Access to Plates with Poor Clearance

Figure 38, is an excellent example of what Priest was talking about.

Here the welder clearly cannot see his work. Admittedly, this is an extreme example, but that is why it makes such a good demonstration of the point. While the calculation for this weld was easy to perform, between stiffener plates and a column, and it can be detailed very cleanly on a computer and shows no conflict in the model, there is no way for a welder to accomplish it. Unlike the old riddle of how to get everyone across the river in the row boat that can only carry so much weight, no welding sequence will solve this dilemma.

Unfortunately, by the time this was discovered, large plates (PL 1 ¾” x 14” x 3’-0”), had already been ordered, cut, and received a double bevel for a CJP weld. Since this detail occurred in several locations, there was a significant loss of material and labor, was as well as a potential negative impact to the schedule.

The inability to perform this work was overlooked by the steel estimators, detailers, several shop order personnel, as well as the engineer of record

The welds required between the plates and the web of the column (here a W24x730), require many passes. It is critical for the Welder to have good clearance to lay down the passes cleanly. The Inspector also requires visibility and clearance to be able to properly inspect and test the weld.

The solution, arrived at by discussions between the Fabricator and the Engineer of Record, involved fewer, larger plates, with a larger gap between them (see Figure 40). New 2-½ inch plates had to be ordered and processed before fabrication could proceed.

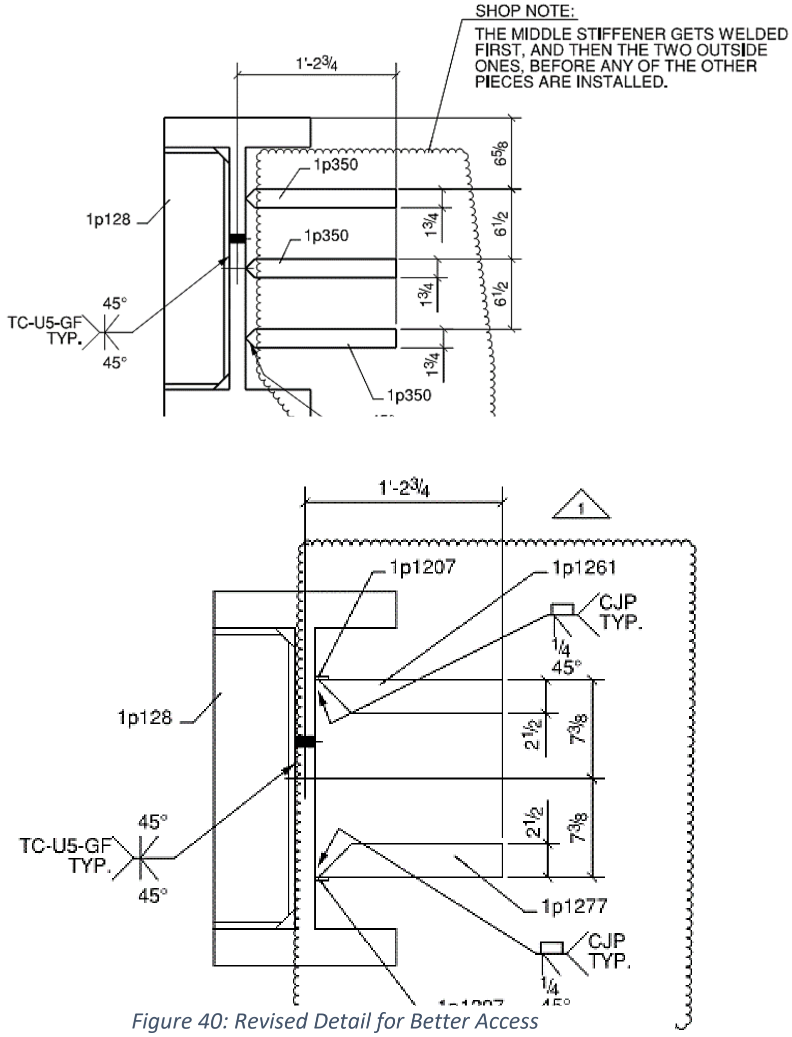


Figure 40: Revised Detail for Better Access

Recommended: c_{min} = min(0.6*b*, 5 in.) Minimum: c_{min} = min(*b*/2, 4 in.)	c_{min} = ¾ in. ⁵⁄₁₆ in. < t ≤ ⅝ in.: c_{min} = min(*b*/2, 2 in.) ⅝ in. < t: Recommended: c_{min} = min(b/2, 3½ in.) Minimum: c_{min} = min(b/2, 2½ in.)

Figure 41: Recommended Clearances for FCAW and GMAW Welding (Dowswell, Smith, 2017)

The article, "Clearance is CRITICAL" (Dowswell, Smith, 2017) explains the different types of welding, and the recommended clearances for the various types. References are available for these recommendations are cited in the article. This article is much more extensive and technical than what is presented in this paper.

The above-mentioned article also discusses an AISC-funded research project designed to determine proper clearance requirements for welded joints. This research yielded the recommendations for FCAW and GMAW welding shown in Figure 41.

When reviewing the plates in the previous example, with the recommendations shown in Figure 41 Case 1, a value of, c_{min}=min(14*0.6, 5 in.) is recommended when welding the second plate. The controlling value is 8.4 inches. The minimum value would be 7 inches. In the above example, the clearance provided was 9-¾ inches, which did prove to be adequate.

However, due to the plate length and depth, a custom track welder (modified Bug-O track torch) was developed to provide consistent control of the weld while still allowing the Fabricator good visibility (hands were not blocking the view), shown in Figure 42.

However, removal and repair of weld, if found to be defective, would still be very difficult. Fortunately, the welds were made successfully and no correction or repair was required. The track welder enabled the fabricator to monitor the weld and closely control placement and speed during the many passes that were necessary to accomplish this CJP.

When designing welds with tight clearances, it is critical to provide the necessary access for the Welder and Inspector, for a good outcome. The AISC study (Dowswell, Smith, 2017), and the recommendations in the AISC Steel Construction Manual (AISC, 2017) can be utilized to help make this determination. If the specifier is still unsure, contact a Fabricator to get their feedback on whether the weld is feasible or economical.

Figure 42: Modified track torch

7.0 References

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