steelwise

FIELD FIXES

BY MARGARET A. MATTHEW, P.E.

Minimizing fixes in the field starts in the design phase.

WHEN A PHONE CALL comes in from the field, it's typically not good news.

Most likely it involves a problem of some sort that needs to be fixed. These calls are inevitable on any construction project, but they can be minimized with some helpful tips and best practices on the design side.

Anchor Rods

Let's start with anchor rods. At first glance anchor rods seem fairly straightforward—but they can cause significant problems in the field when they are located incorrectly or set to the wrong elevation. A good place to start is familiarity with tolerances. Section 7.5.1 of the AISC *Code of Standard Practice* calls for a tolerance of ±⅛ in. between anchor rods in a group and ±¼ in. between groups of anchor rods. However, ACI 117 allows larger tolerances based on specific anchor rod sizes (and keep in mind that the larger ACI anchor tolerances are so large that they may not work with the fabrication tolerances):

- ¾ in. dia. - ⅞ in. dia. = ±¼ in.
- 1 in. dia. - 1½ in. dia. = ±⅜ in.
- 1¾ in. dia. - 2½ in. dia. = ±½ in.

If you are already taking tolerances into account in your design, which standard do you use? The key is to make sure the concrete contractor that is setting the anchor rods is making the same assumptions you are. You should either require that the anchor rods be placed to the tolerances in the AISC *Code of Standard Practice* or specify special hole sizes and washers to accommodate the ACI 117 tolerances. Make a point of communicating this clearly in your project specifications and drawings.

Why is this important to the engineer? OSHA (Occupational Safety and Health Administration) has requirements that cover anchor rod placement in OSHA 1926.75. OSHA requires that anchor rod repair, replacement or field modification cannot be done without the approval of the structural engineer of record, and any approval given must state if the repair or modification requires guying or bracing of the column. The contractor is also required to provide written notification to the erector of any repair or modification.

Here are some more guidelines will help you avoid some of the more avoidable anchor rod issues:

- Provide a design with ample rod projection length and ample rod thread length.
- Use ASTM F1554 grade 36 or 55 in larger diameters if necessary before using grade 105 (high-strength steel limits the solution options when things go wrong.)
- Place anchor rods in symmetric patterns where ever possible.
- For large mat concrete foundations, consider using drilled-in anchor rods if possible; it's difficult to place and hold large or long conventional anchor rods in alignment while concrete is being placed.
- Require grouting before column base plates are loaded to prevent the base plate from punching through leveling nuts.
- Provide adequate edge distance for anchor rods.
- Place anchor rods inside the reinforcing steel in a concrete pier or wall to prevent breakout.

In addition, AISC's *Design Guide 1 – Base Plate and Anchor Rod Design* is a great resource for more information on the design, detailing and specification of anchor rod connections.

Columns

Moving up from base plates, let's take a look at columns. Columns can be too long or too short, they can be out of plumb or the column splice might not fit right. These issues can be avoided by being clear with information on the design drawings. Use uniform base plate elevations whenever possible, and flag any exceptions on the drawings. Clearly show the elevations at the tops of columns, this is especially important when the steel elevation changes to accommodate the roof slope.

Column splices, where possible, should be a standard AISC splice detail like the one shown in Figure 1 (on the next page). These details allow the adjustment necessary to properly plumb the building. Gaps may occur at the splice due to fabrication or plumbing tolerances. In reviewing if a fix is required, it is important to remember that gaps up to 1⁄16 in. are permitted by AISC *Specification* Section M4.4 and that gaps exceeding 1⁄16 in. but less than ¼ in. require shims to be used to pack out the gap.

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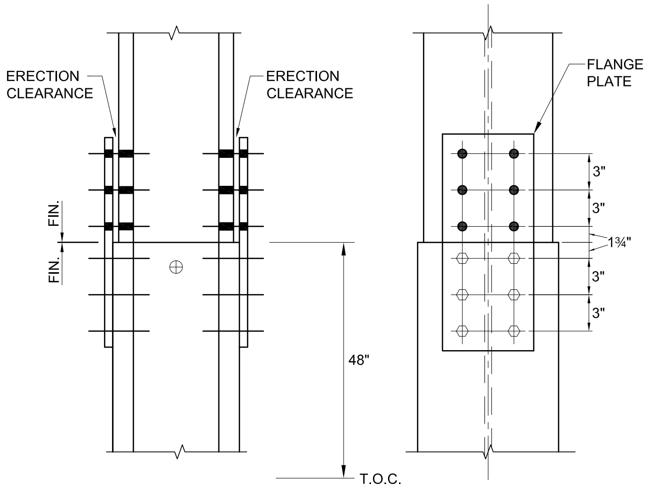


Figure 1. Standard AISC splice detail.

Cantilevered Beams

Cantilevered beams present a number of issues to be aware of, including erection stability. OSHA Subpart R Section 1926.756 states that "a competent person shall determine if more than two bolts are necessary to ensure the stability of cantilever members; if additional bolts are needed, they shall be installed." Although erection stability is a part of means and methods and not the responsibility of the engineer of record, there are some things that can be done in design to help:

- Use bolted connections where possible for ease of erection and worker safety.
- Allow short cantilevers (up to about 4 ft) to be shop installed.
- Indicate if camber or a preset elevation at the end is required.
- For longer cantilevers consider making the beam continuous into its back span by stacking the columns.
- Provide a note on the drawings requiring the cantilever to be supported until the final moment connection is made.

Provide the actual shear and moment for the design, especially at complete-joint-penetration groove welded joints in a weak-axis moment connection. While the beam is usually ASTM A992, the stiffener plate may be ASTM A36, which creates a mismatch. The strength of the weld is based on the strength of the plate material and if it is less than the strength of the beam material, the full strength of the beam cannot be developed.

If the project delivery method allows for collaboration with the fabricator and erector during the design phase, getting their input on connection details like this is an invaluable resource to minimize problems in the field.

Bolt Holes

In a perfect world, bolts fit perfectly in their intended holes, and edge distance requirements are always met. In reality, bolts sometimes don't fit and edge distances can get compromised. To preempt the issue of bolts not fitting in their intended holes, consider designing with oversize holes (and slip-critical joints) for complex framing. Using short slots normal to the direction of loading can also provide extra erection clearance. Often, issues with edge or end distances can be avoided by simply adding an extra ⅛ in. to the minimum distance requirement when developing connection details for the design drawings. Remember that edge distances in Table J3.4 of the AISC *Specification* can be reduced based on the required strength at these locations.

Camber

Cambered members are another instance where the interface between steel and concrete can create a tolerance issue. Section 6.4 of the AISC *Code of Standard Practice* stipulates a camber tolerance for beams of -0 in. to +½ in. for beams up to 50 ft long; an additional ⅛ in. is allowed for each additional 10 ft of beam length in excess of 50 ft. It also requires that camber be measured with the beam in the unstressed position in the shop because there is no way to practically and accurately verify shop camber after it is received on-site.

Sometimes, *not* using camber is the best way to prevent field issues. These are situations when camber is not recommended:

- Beams with moment connections. They should already be stiff enough and if cambered, special provisions would be required to accommodate the end slope of the beam at the connection.
- Spandrel beams supporting fascia. They should be designed to be stiff enough to support the façade.
- Beams subject to torsional loads. Like beams with moment connections, camber will require special design of flange connections.
- Beams with bracing connections. Any camber could make the connection fit-up difficult or impossible.
- Beams less than 25 ft long. This limitation is a practical one based on the length of typical mechanical presses in fabrication shops.
- Beams with webs ¼-in. thick or less. There's a good chance the thin web will buckle if mechanically cambered.
- Beams that require ½ in. or less of camber.

Common camber issues are too much or too little camber, deck bearing problems or difficulties with steel studs. It is important to correctly specify the amount of camber. Generally it is recommended that camber should be specified for the concrete dead load only, however some engineers include an allowance for concrete shrinkage and the variation in load due to deck support conditions.

Also watch out for when a cambered steel beam will be adjacent to an un-cambered beam or support. The steel deck might not end up bearing properly on the intended support (as shown in the following photo). To prevent this situation, specify transitions in the beam camber and/or provide for a deck splice. It's also good practice to design the deck for a conservative simple span condition to ensure adequate capacity.

No one wants to see steel studs sticking out of their finished floor. Good design practice is to select a stud length at mid-range of the permitted stud height, which is at least 1½ in. above the top of the deck and ½ in. below the top of the concrete elevation.

Deck not bearing on un-cambered beam adjacent to cambered beam.

Braced Frames

Remember to consider the connections when selecting member sizes to eliminate member reinforcement due to forces at the connection. This is especially true when designing bracing connections (And AISC has a great new resource to help in designing braced connections, Design Guide 29: *Vertical Bracing Connections-Analysis and Design*.)

In industrial buildings and other applications with exterior braced frames and girts, watch out for interference issues. The key is to think about bracing and girts in the same plane of the wall during the design phase, and either design and detail the girts in conjunction with the bracing to avoid the interference or provide details that allow for compatibility.

Steel-to-Concrete Connections

Steel-to-concrete connections, such as those for a concrete core in a steel frame, are another area where specifying the construction tolerances on the drawings is critical. Here, again, differences between ACI tolerances and AISC tolerances must be considered and the requirements specified in the contract documents.

ACI 117 Section 4 allows for a 1-in. tolerance on vertical alignment for heights not exceeding 100 ft; for heights exceeding 100 ft the tolerance is 1⁄1,000 times the height but not greater than 6 in. Lateral alignment tolerance is 1 in; these are larger than the tolerances allowed for the steel framing in the AISC *Code of Standard Practice*.

Structural steel connections, where possible, should be designed and detailed to accommodate these tolerances, except where the concrete contractor or the project specifications provide for smaller tolerances. Embedded materials such as plates are often located incorrectly and can vary significantly even when placed within ACI 117 tolerances. They should be designed and detailed to allow for these concrete tolerances as a minimum; providing nail holes to fasten the plates to the forms can help the installation.

When the actual length at a beam intended for connection to a concrete wall is long or too short, an adjustable connection detail will help to minimize issues. Figure 2 shows an example of an adjustable welded/bolted detail and Figure 3 shows an example of an adjustable welded/welded detail. It might also be a good idea to have a standard field-fix preapproved for the conditions that vary so much that the holes do not align even with the use of these slots.

Figure 2 – Adjustable welded/bolted connection.

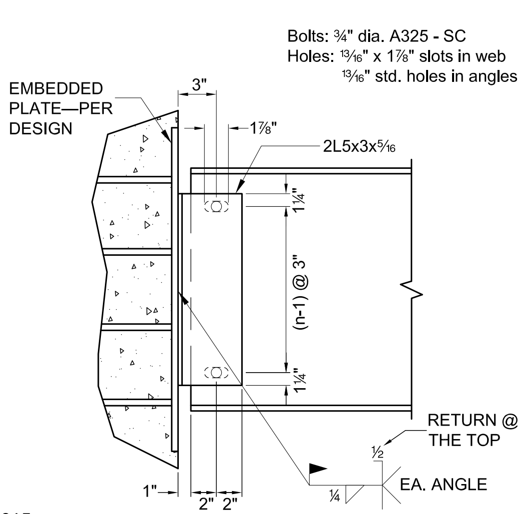
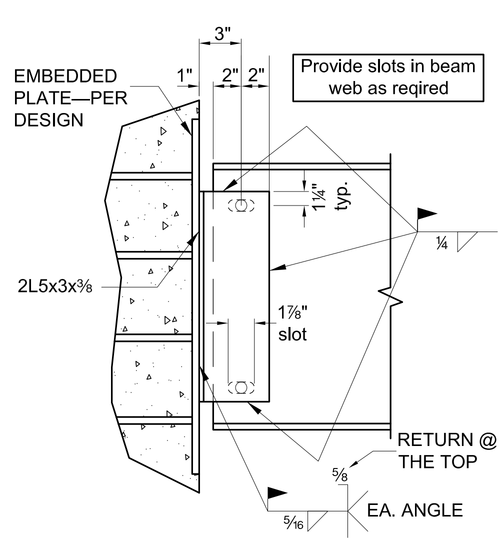


Figure 3 – Adjustable welded/welded connection.



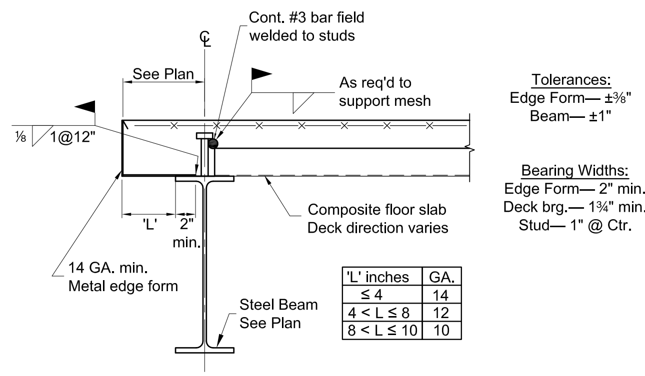
Façade Framing

Next to anchor rods, façade support issues are the most common problems encountered in the field. To avoid detailing and tolerance issues with façade framing, it is imperative that the responsibilities of all parties involved are understood at the beginning of the design process, as outlined in Design Guide 22: *Façade Attachments to Steel-Framed Buildings*. Because it is common for the façade framing to be designed by a specialty contractor's engineer, the engineer responsible for the main structure could be working to a different set of assumptions than the engineer actually designing the façade framing. Clear communication between parties will facilitate a problem-free façade installation.

Bearing width can be a big issue when dealing with façade framing. As shown in Figure 4, spandrel beams are often specified as wide-flange sections with a 4-in.-wide flange. When minimum bearing width requirements are taken into account along with the addition of erection tolerances, a 4-in. flange will typically not be wide enough. The beam flange must accommodate the 2-in. minimum bearing requirement for an edge form and the 1¾-in. minimum bearing requirement for steel deck, while allowing 1 in. for the steel stud bearing at the center of the flange. The beam flange must allow for these requirements and still provide ±⅜-in. tolerance for the edge form on a beam that is part of a steel frame with a plumb tolerance of up to 1 in. Flanges 5 in. or less in width will typically require field fixes.

Thinking ahead is always the key! Being aware of potential constructability issues during design will go a long way to preventing them from ever happening. ■

*This article is based on the 2014 NASCC: The Steel Conference presentation "Field Fixes: Common Problems in Design, Fabrication and Erection: Solutions and Prevention" by Larry Kloiber, P.E., structural fabrication consultant with LeJeune Steel. You can view the presentation at **www.aisc.org/2014nascconline**. All the Design Guides mentioned are available for free to AISC members at **www.aisc.org/designguides** and AISC's complete ePubs collection is viewable at **www.aisc.org/epubs**.*



'L' inches	GA.
≤ 4	14
4 < L ≤ 8	12
8 < L ≤ 10	10

Figure 4 – Typical deck bearing at slab edge detail.