A collection of some of the decade's most useful concepts gleaned from the pages of *MSC*.

# Best Tips of the 21st Century: Connections

BY HEATH MITCHELL, P.E., AND MATTHEW BRADY, P.E.

**TOOLS, TECHNIQUES, MATERIALS,** and personnel change over time. Technology never stands still; tasks and responsibilities change; and new people continually enter the workforce. Thus even the unquestioned rules of thumb, the elements of common knowledge, can usually bear repeating.

*MSC* regularly recruits experienced and knowledgeable practitioners from across the industry to share their accumulated wisdom with regard to structural steel fabrication, design and construction. With the first decade of this century now complete, we asked AISC's Heath Mitchell and Matthew Brady to reiterate what they view as some of the most useful concepts from this 10-year span.

**Source Material for Best Tips of the 21st Century—Part 1**

- "57 Tips for Reducing Connection Costs," by Victor Shneur, P.E. (July 2003 *MSC*)
- "30 Good Rules for Connection Design," by Carol Drucker, S.E. (May 2004 *MSC*)
- "24 Tips for Simplifying Braced Frame Connections," by Victor Shneur, P.E. (May 2006 *MSC*)
- "59 Tips & More for Economical Design," compiled by Geoff Weisenberger (January 2008 *MSC*)
- "In the Moment," by Victor Shneur, P.E. (June 2009 *MSC*)
- "98 Tips for Designing Structural Steel," by James M. Fisher, P.E., Ph.D., and Michael A. West, P.E. (September 2010 *MSC*)

All articles are available as free downloads on the *MSC* website, **www.modernsteel.com/backissues**.

## 17 Connection Tips That Are Worth Repeating

Heath Mitchell, P.E., is AISC's director of technical assistance. He joined the AISC Steel Solutions Center in November 2010 and coordinates all of the answers to technical questions submitted to AISC through the Steel Solutions Center.

Mitchell previously worked for AISC from 1999 to 2001. Since then he has been employed by PCS Structural Solutions, Tacoma, Wash., while maintaining his involvement with AISC as a committee volunteer. He also has worked part-time on the AISC technical assistance panel over the last year.

Here are 17 of Mitchell's favorite tips on connections, gleaned from *MSC* articles published since 2000. The source article is noted at the end of each tip.

1. Always provide complete load paths (including transfer forces) where there are axial forces. Keep load paths simple. (Shneur, 2003)
2. Simplify as much as possible. For example, make column-base details symmetrical, use the same spacing for expansion anchors and minimize the number of sizes of slab edge closures/pour-stops. This expedites the fabrication and erection process and greatly reduces the number of possible mistakes—and repair costs. (Shneur, 2003)
3. Use single-pass fillet welds where possible. A ⅜-in. fillet weld requires 44% more material and 100% more labor, but it is only 20% stronger than a 5⁄16-in. fillet weld. (Shneur, 2003)
4. Avoid the "weld-all-around" symbol. It is expensive, and in a lot of cases it's not required—and sometimes it's even prohibited. (Shneur, 2003)
5. Group similar connections rather than have several different connections. Connections on a project should be as uniform as possible to save fabrication time and reduce the possibility of errors. (Drucker, 2004)
6. Avoid overhead welding. The preferred welding positions are flat and horizontal. Overhead welding is difficult, costly and generally yields lower quality welds. For single-pass SMAW fillet welds, it can take four times as long as welding in the flat or horizontal position. (Drucker, 2004)

7. Consider finishing to bear. For connections with high compressive loads, it could be more economical to finish the steel to bear and provide AISC's minimum-required weld size instead of transferring the compressive force through large fillet or groove welds. When steel is to be finished to bear, it must be indicated on the connection detail. The detail also should call for the beam flanges to be square to the beam web. In detailing, stiffeners might need to be longer than $d\text{-}2t_f$ for beam overrun in depth and variation in beam-flange thickness. Per AISC *Specification* Section M, gaps not exceeding 1/16 in. are permitted in bearing connections. (Drucker, 2004)
8. At beam-to-HSS column moment connections, use direct moment connections when possible. Moment connections in which the beam's flanges or flange plates are welded directly to the face of the HSS column are the most economical moment connection to an HSS column. It is preferred over cut-out plate (doughnut) or throughplate connections. If the resistance of the direct moment connection is insufficient, then a cut-out connection is preferred to an expensive through-plate connection. The limit states for direct moment connections are given in the HSS *Specification* in AISC's *LRFD Manual of Steel Construction*, 3rd Edition. They include effective flange width, and yielding, crippling, and punching shear and buckling of the side walls. The HSS *Specification* does not include the limit state for yielding of the HSS face given in equations (5-2) and (5-3) in AISC's *HSS Connections Manual*. Based on conversations with AISC, this limit state was omitted due to limited testing and does not need to be considered. (Drucker, 2004)
9. Don't use fully restrained moment connections to resist torsion. Typically, a 5/16-in. or 3/8-in. end plate shop-welded to both flanges or bolted flange angles will provide adequate strength. Note that connection flexibility can be provided by keeping bolts at the end plate between the flanges, or using snug-tight bolts in the slotted holes in horizontal legs of flange angles. (Figures 1 and 2 illustrate these connection concepts.) (Shneur, 2009)
10. Shop-weld short cantilevers to the column as shown in Figure 3. This will make the erection much safer. (Shneur, 2009)
11. At cantilever-to-beam connections, when the bottom flange is always in compression, use an end-plate connection extended below the bottom flange as illustrated in Figure 6, on the following page. In this case, top-flange tensile force will be resisted by a CJP weld or flange plate, and bottom-flange compressive force will be resisted by bearing. Any field connection (CJP weld or flange plate) is eliminated at the bottom flange. The same concept can be applied to:
    - Cantilever and backing beam-to-column moment connections when the bottom flange is always in compression.
    - Field splices for beams and plate girders when the top flange is always in compression. (Shneur, 2009)
12. When rolled beams and plate girders need to be field-spliced, use end-plate connections described in AISC *Steel Design Guide 16, Flush and Extended Multiple-Row Moment End-Plate Connections*, when possible. (Shneur, 2009)
13. Moment connections to embedded plates in concrete require special details because of the different tolerances for steel and concrete. When designing these connections:
    - Make embedded plates larger than required for connections to allow for concrete tolerances.
    - Size embedded plate thickness conservatively; it may be moved from the design position, and flange tensile force will not be applied at the theoretical location. (Shneur, 2009)

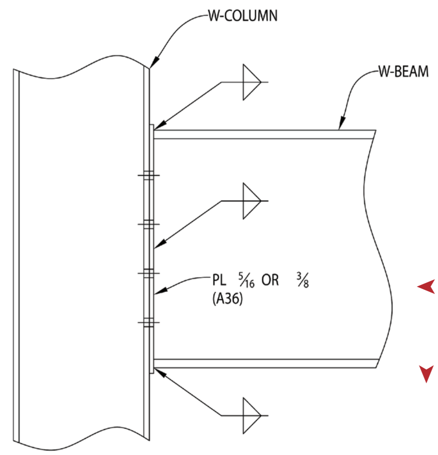
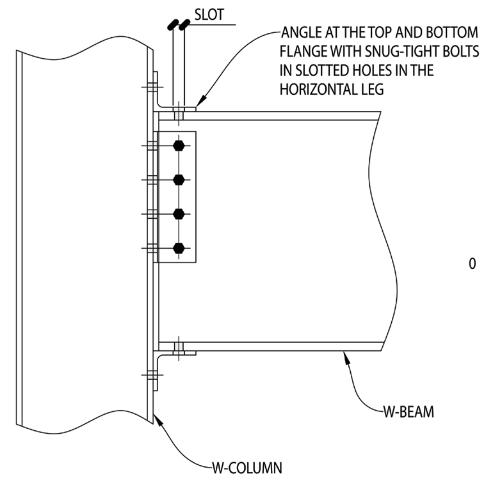



**Fig. 1:** End-Plate Connection for Torsion

**Fig. 2:** Flange-Angle Connection for Torsion



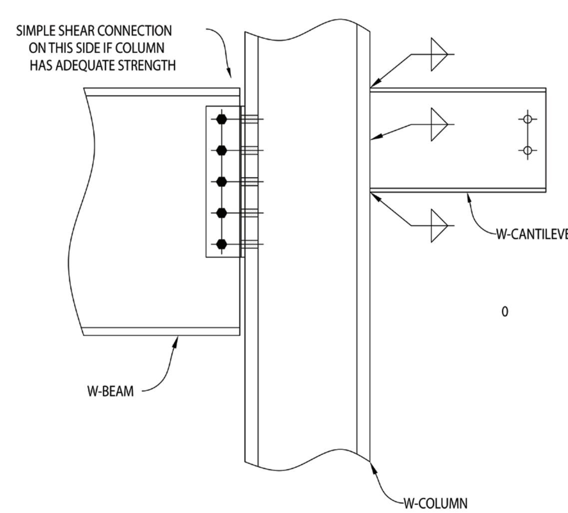




**Fig. 3:** Shop Welded Short Cantilever

*Heath Mitchell, P.E., is AISC's director of technical assistance.*

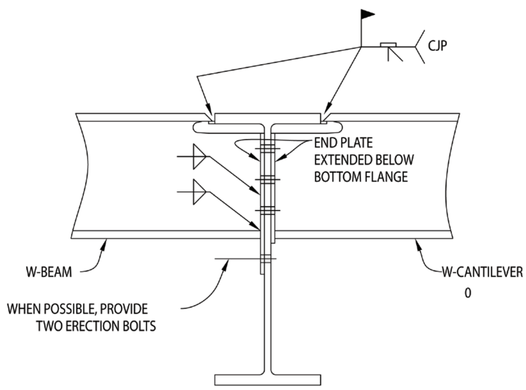
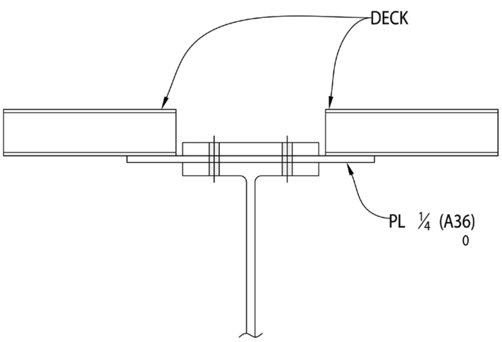



**Fig. 6:** Cantilever Moment Connection at W-Beam when End Moment is not Reversible (Bottom Flange is Always in Compression)

**Fig. 7:** Deck Bearing at Bolted Flange-Plate Connection




- Headed studs are preferable to transfer beam flange tensile force. When large moments need to be resisted and long anchors/rebars are required, consider using anchors that are field attached to the plates or field-screwing anchors into the couplers shop-welded to the plates. This will make fabrication and installation easier.
- All connection material needs to be field-welded to the embedded plates because of interference with formwork.
- Flange-plated connections field-welded to both the beam and embedded plate are preferred because of much tighter tolerances for steel than for concrete members. (Shneur, 2009)

*Matthew Brady, P.E., is an advisor in the AISC Steel Solutions Center.*

**14.** When using a field-bolted top flange plate, make a note to provide deck bearing at the flange connection. A ¼-in. shim between plate and flange can be extended providing support in lieu of a standard deck angle. Figure 7 shows an example with a ¼-in. shim. (Shneur, 2009)

**15.** Make embedded plates a minimum 6 in. to 8 in. larger than required for connections as a rule of thumb. Field fixes for embedded plates that are mislocated are time-consuming and expensive. (Weisenberger, 2008)

**16.** Maximize work requiring intermittent rather than continuous inspection. As codes and standards have evolved, the amount of third party inspection has increased. These inspections are in addition to the quality control work of the contractors and can impose a significant burden on the project. The types of connections used will affect the amount of third party inspection work that has to be performed in the field and the associated costs. Inspection is covered in Chapter 17 of the *International Building Code* and in Chapter N of the 2010 AISC *Specification for Structural Steel Buildings*. For a discussion of inspection terminology and requirements, see "Quality Time," *Modern Steel Construction*, March 2010 (available at **www.modernsteel.com/backissues**). (Fisher and West, 2010)

**17.** Minimize the need for stiffeners and doubler plates. This greatly reduces costs (see table). (Fisher and West, 2010)

| Estimated Steel Requirements for Stiffeners | |
|---|---|
| 1 pair of fillet welded stiffeners | 300 lb |
| 2 pair of fillet welded stiffeners (at top and bottom girder flanges) | 600 lb |
| 1 pair of groove welded stiffeners | 1,000 lb |
| 2 pair of groove welded stiffeners (at top and bottom girder flanges) | 2,000 lb |
| 1 doubler plate | 350 lb |

Table data for Fisher & West No. 81

## 10 Things to Keep in Mind About Structural Steel Connections

Matthew Brady, P.E., is the newest member of the AISC Steel Solutions Center team. In addition to providing conceptual studies to decision makers on a wide variety of building projects looking to utilize structural steel as their framing system, he also answers incoming technical questions. Prior to joining AISC in December, Brady worked in Chicago designing buildings at Holabird & Root, and bridges for Alfred Benesch, as well as working for Lockheed Martin on FAA-related projects. He also is company commander for the 631st Engineer Support Company of the Illinois Army National Guard.

Here are 10 key ideas, from *MSC* articles published over the last 10 years, each with additional related points, that he recommends keeping in mind. The source for each is noted in parentheses.

**1.** Review the member sizes for connection economy:

a. Preferably, a supporting beam should have at least the same depth as the supported beam. (Shneur, 2003)

b. Don't frame W8 beams into the webs of heavy W-shapes or plate girders. The thick flanges of the heavier shapes will require excessive—and sometimes impossible—copes in the W8. (Shneur, 2003)

c. Favor W12 and W14 sections (especially for typical gravity columns) whenever possible. The distance between flanges makes web connections easier. Unless architecturally required, avoid W10 and W8 columns because they have very limited space between flanges, which makes connections more difficult. (Shneur, 2003)

d. Consider using heavier member sizes (especially in column sections) to eliminate reinforcement (stiffeners and doublers). Chapter 3 in AISC *Steel Design Guide 13, Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications*, provides suggestions and cost comparisons. (Shneur, 2003)

e. Provide adequate flange width at perimeter members. Provide beams that frame slab openings to support deck and pour stops, and to weld studs. (Shneur, 2003)

f. Least weight is NOT always least cost! Select member sizes with sufficient depth to provide reasonable connections. For example, use a W16×26 rather than a W14×22, especially if the member is coped. (Weisenberger, 2008)

g. Do not reinforce beam web penetrations unless absolutely necessary. (Weisenberger)

**2.** Make embedded plates a minimum 6 in. to 8 in. larger than required for connections to allow for concrete tolerances. Field fixes for embedded plates that are misallocated are time and money consuming. (Shneur, 2003)

**3.** Use single pass fillet welds where possible:

a. Limit the maximum fillet weld size to 5⁄16 in. (especially in the field). This is the maximum-size weld that can be completed in a single pass using the shielded-metal arc-welding (SMAW) process. Smaller, longer welds are preferred over larger, shorter welds. (Drucker, 2004)

b. Select fillet welds over partial-penetration groove welds when possible. Select partial-penetration groove welds over full-penetration groove welds when possible. (Weisenberger, 2008)

c. Larger weld sizes generally require multiple passes and may require increased levels of inspection and thus add cost. (Fisher and West, 2010)

**4.** Avoid the "weld-all-around" symbol.

a. For column base plates, fillet welds that wrap around the flange ends (flange toes) and web-to-flange fillets take additional time because of changing weld positions and may lead to cracks due to high residual stresses in the welds. Also, these welds add very little to the strength. (Shneur, 2006)

b. Try to stay with welds on both sides of the web and flanges, if possible. (Shneur, 2006)

c. Gravity columns to base plates should rarely need welding all around. Normally welding one side of the web, the outside of one flange and the inside of the other flange is sufficient. (Weisenberger, 2008)

**5.** At bolted flange-plated connections, the flange plate should not be the same width as the beam flange. Allow at least a ½-in. difference on each side of the plate at bolted flange platted connections. If bolt holes misalign in the filed, there will be sufficient shelf dimension to place longitudinal fillet welds to compensate for the missing bolts. (Drucker, 2004)

**6.** Avoid slotted holes in plates thicker than the bolt diameter. Slots in thick plates are hard to punch and must be flame-cut, which is difficult and costly. Standard holes or oversized holes are preferred. (Drucker, 2004)

**7.** Do not over-economize connections:

a. If the overall connection configuration is virtually the same, reducing the amount of weld or bolt count in a single non-repetitive connection, by even a large percentage (e.g., in excess of 25% to 30%), will probably increase the overall time and expense of the project. Repeating connections will reduce connection design, detailing, layout, fabrication, and erection costs due to the reduced learning curve. (Weisenberger, 2008)

b. Time spent on connection design should be consistent with time spent on analysis and member design. Remember that the majority of shop and field labor is in the connections. Approximately 30% of the cost of structural steel relates to material costs. The rest is highly dependent on connection costs. Spend time thinking through the connections. (Fisher and West, 2010)

**8.** When showing stiffeners or other plate material, use popular flat bar sizes and UM plate sizes:

a. (usually 3⁄16 in., ¼ in., 5⁄16 in., ½ in., ⅝ in., ¾ in., and 1 in. thicknesses and widths 1 in. through 6 in., 8 in., 10 in., and 12 in.). (Weisenberger)

b. Bars make more sense than handling a 96-in. by 20-ft plate just to cut a few fittings. (Weisenberger, 2008)

c. Also, avoid extremely large and thick angle sizes when dealing with a small amount of fittings. (Weisenberger, 2008)

**9.** Read ASTM 6:

a. An understanding of steel mill production tolerances will play a major role in the design and detailing of steel frames and their connections. Designers should be familiar with mill tolerances when selecting members and designing connections. Essential parameters are permissible variations in overall depth, flange tilt, and the position of the web in the wide-flange shape. Recognizing these allowable variations from square and true will guide designers to design connections that are complete and relatively easy to fit up. Allowance for the variations must be included in the connection design. This is usually accomplished with gaps for shims, and oversized and slotted holes. (Fisher and West, 2010)

b. Always design and detail connections for the tolerances. At every moment connection, the web and both flanges of the framing beam are connected to the supporting member. Disregarding tolerances may make connections unworkable and lead to costly modification. Refer to ASTM A6/A6M/AISC *Code of Standard Practice for Steel Buildings and Bridges*, and AWS D1.1 for the allowable mill, fabrication and erection tolerances. Depending on actual connections, there are a number of different ways to provide for tolerances. For example, for directly welded flange-to-plate connections at column webs, specify connection plates thicker than the flanges; use slip-critical bolts in oversized holes for flange-plated connections, etc. (Shneur, 2009)

c. And never forget constructability and clearances for welds and bolts. For example, when a directly welded moment connection is made to a column web, locate the bolt group for the web connection outside of the column flanges. This simplifies erection and bold pretensioning and reaming, if required. (Shneur, 2009)

**10.** Be aware of OSHA Subpart R rules for steel erection. OSHA 1926.756(c)(1) prohibits double connections at columns and/or beam webs overall column where all the bolts are common to both connections, unless means is provided to secure the first beam erected from falling away when the second beam is erected. Figures 2-13 through 2-17 in AISC *Detailing for Steel Construction*, Third Edition, provide common solutions that conform to the rule. If staggered connections are used, check that the T-dimension can accommodate the extra row of bolts. Other OSHA Subpart R rules are cited in this article and the complete list is available at **www.osha.gov**. (Fisher and West, 2010) MSC