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In the Moment

BY VICTOR SHNEUR, P.E

Here are 60 tips for simplifying fully restrained moment connections for W-shapes.

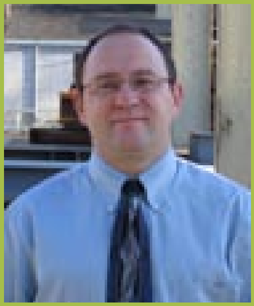
MANY BUILDINGS HAVE MOMENT CONNECTIONS FOR lateral frames and/or cantilevers. Even though they don't encompass the majority of the connections, it is important to give moment connections special attention since they require more work and may be a safety consideration during erection.

Here are a few (well, 60) suggestions for making moment connections easier to design, detail, fabricate, and erect, along with a few recommendations for avoiding problems:

1. Moment connections for cantilevers require special attention from the erector. They always must be completed, including moment connection for backing beams, before the cantilever is released. Otherwise, adequate temporary support should be provided. Also, per OSHA requirements, a competent person should supervise cantilever erection.

2. If moment connections are required for the lateral load resisting system, select an R of 3 or less whenever possible. When $R > 3$ the AISC *Seismic Provisions* must be applied, which has a significant associated cost implication.

3. When heavy rolled W-shapes are required at moment connections with complete-joint-penetration (CJP) groove welds, don't forget about special requirements for the material covered in the AISC *Specification*, Section A3.1c.



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4. When moment connections are not designed by the EOR, provide all end reactions, including vertical end reactions and moment envelops. The fabricator can then select the most efficient connections and check columns for reinforcement.

5. For non-domestic sections, consider using A913 steel to substantially reduce preheat requirements at welds (see Table 3.2 in the AWS D1.1:2008) and possible column reinforcement at moment connections.

6. As the engineer of record, request removing backing bars *only* when required by the governing code or architectural reasons. (This is an expensive procedure.)

7. Do not fill weld access holes with weld material for cosmetic or corrosion-protection reasons. In addition to the cost, it creates undesirable triaxial stresses. Using mastic materials is preferable to welding.

8. Avoid weak-axis moment connections at W-columns.

9. For moment frames, consider using partially restrained or flexible moment connections in lieu of fully restrained connections whenever possible.

10. Carefully examine cantilever framing for reducing the number of members with moment connections. This is the best way to save money on moment connections. Potential increase in material weight can be well justified by savings in labor and safer erection.

11. When a direct-welded flange moment connection is made to a column web, extend connection plates at least ¾ in. beyond the column flanges to:
   - ➜ avoid intersecting welds
   - ➜ provide for strain elongation of the plates
   - ➜ provide adequate room for runout bars

12. When possible, consider using a deeper W-shape to reduce flange forces and possibly eliminating stiffeners at columns. The increase in material weight is typically offset by eliminating stiffeners and using a less expensive/lighter moment connection (e.g., an extended end-plate connection in lieu of a directly welded connection if end moment allows).

13. When moment connections are made to a column or beam web, use beams with the same depth on both sides of the web where possible.

14. Don't specify fully restrained moment connections to resist moderate beam axial forces. Double-angle connections with thicker angles for perpendicular framing and shear end plate connections with thicker plates for skewed framing often can be designed for combined shear and tension forces.

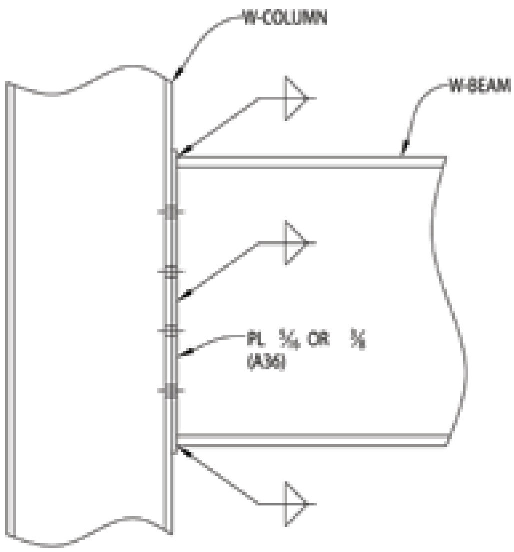

Figure 1. End Plate Connection for Torsion

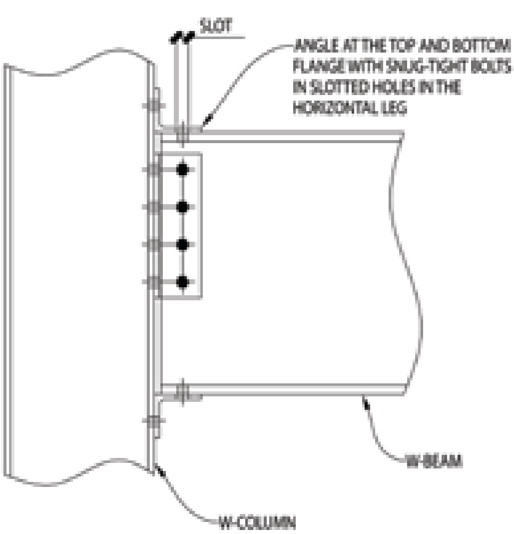

Figure 2. Flange Angle Connection for Torsion

15. Don't use fully restrained moment connections to resist torsion. Typically, a $\frac{5}{16}$-in. or $\frac{3}{8}$-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.)

16. Check section dimensions (depth, flange width, flange thickness) at moment connections. Choosing sections that fit correctly may simplify the connections (e.g., CJP welding detail at the bottom flange when flanges match each other).

17. Remember that eccentricity can be neglected in the web shear connections. As explained on page 12-3 in the 13th edition AISC *Manual*, it is permissible "since, by definition, the angle between the beam and column in a fully restrained moment connection remains unchanged under loading."

18. Bearing bolts in standard or horizontal short-slotted holes (perpendicular to the line of force) are permitted in the web shear connections. This can reduce number of bolts and avoid special requirements for faying surfaces.

19. If beams with moment connections frame into both column flange(s) and web, try to use the same depth for all beams. This eliminates interference where stiffeners are required.

20. Skewed moment connections at columns, especially for beams framing into a column web, can be difficult to make. Modifying the framing, rotating the column, or slightly moving the beam end can greatly simplify these connections.

21. Pay attention to sloping beam-to-beam moment connections; they require special load analysis due to the vertical component of the flange force. Also, the connection layout is typically more complex.

22. Avoid cambering beams with moment connections, because moment connections provide end restraint and reduce deflection. As L.A. Kloiber, P.E. explained in the article "Cambering of Steel Beams" (MSC, 1989), "Moment connections such as end-plate connections, top-and-bottom-plate connections, and direct-welded connections will not fit up properly unless the connection face is fabricated vertical. This requires special layout and cutting after cambering and is an added expense."

23. Shop-weld short cantilevers to the column as shown in Figure 3. This will make the erection much safer.

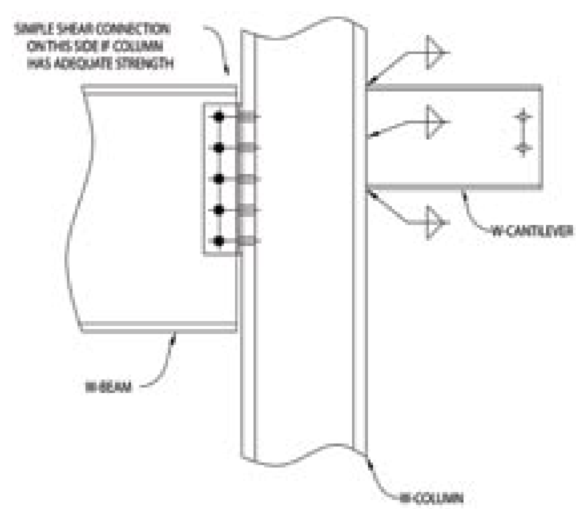

Figure 3. Shop Welded Short Cantilever

24. Cantilever framing to a column doesn't need a backing beam with a moment connection on the other side of the column when the column has adequate strength to resist the cantilever moment.

25. If the cantilever moment needs to be balanced, review the effect of the backing beam moment connection. A partially restrained moment connection may be used to reduce the end moment delivered to the column.

26. When the cantilever is required at the roof, making the beam continuous over the column will eliminate the moment connections for the cantilever and backing beam. This will make fabrication and erection easier and safer.

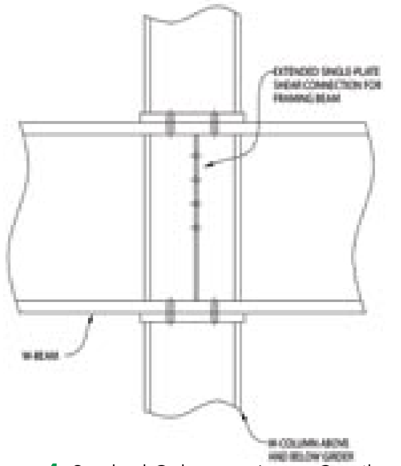

Figure 4. Stacked Column at Large Cantilever

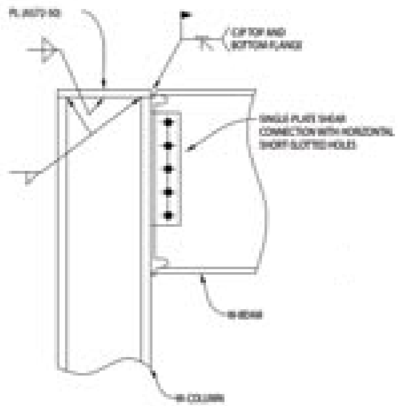

Figure 5. Directly Welded Moment Connection at Top of Column

27. For a large floor cantilever/beam, consider stacking the columns as illustrated in **Figure 4**. This may be easier than reinforcing the column supporting the cantilever and having two large moment connections in the field.

28. When a directly welded moment connection is made at the top of the column in lieu of welding a beam top flange to the column flange and providing two stiffeners between column flanges, make one cap plate and weld the top flange directly to this plate (see **Figure 5**).

29. When possible, favor extended end-plate moment connections over directly welded moment connections. AISC Design Guide 4, *Extended End-Plate Moment Connections: Seismic and Wind Applications* provides design procedures and recommendations. Extended end-plate moment connections make the erection much simpler and safer, eliminating CJP welds at flanges in the field.

30. At extended end-plate moment connections for non-seismic applications, it is acceptable to weld flanges with fillet welds on both sides in lieu of all-around welding, when adequate strength is provided.

31. 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**. 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.

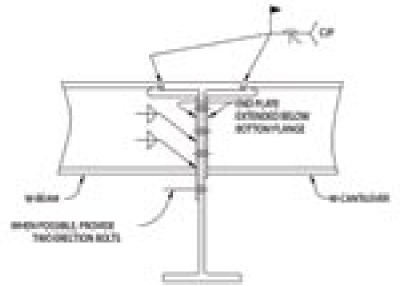

Figure 6. Cantilever Moment Connection at W-Beam when End Moment is not Reversible (Bottom Flange is Always in Compression)

32. Consider using heavier column sections to eliminate the reinforcement (stiffeners and doublers) at moment connections. Chapter 3 in AISC Design Guide 13, *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* provides suggestions and cost comparisons.

33. Layout welds to reduce restraint, especially for large welds. This lowers the possibility of lamellar tearing.

34. Favor fillet welds over groove welds.

35. Allow full-strength connections in lieu of CJP groove welds in statically loaded structures (e.g., for welding stiffener plates at columns). Fillet welds up to ¾ in. are more economical.

36. Avoid field-welded moment connections for galvanized members, especially when end moments are not large. Galvanizing requires special ventilation in closed areas and usually needs to be removed and restored.

37. Normally, a directly welded moment connection is preferable at rectangular hollow structural section columns. If possible, increase HSS column wall thickness and/or use a deeper W-shape to reduce flange forces to eliminate expensive through plates at W-flanges welded directly to the HSS wall.

38. If, however, horizontal through plates are required due to large moments and moment connections are made to different sides, use same-depth beams to eliminate multiple through plates at bottom flanges.

39. Consider flange-plated moment connections to round HSS/pipe columns or to the corner of rectangular HSS columns. Directly welded moment connections may create erection clearance problems when top and bottom flanges are prepared to match the supporting column shape (unless the connection at the other end allows bringing the beam in).

40. When rolled beams and plate girders need to be field-spliced, use end-plate connections described in AISC Design Guide 16, *Flush and Extended Multiple-Row Moment End-Plate connections* when possible.

41. 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.
   - ➜ 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.

42. Choose the geometry of preparation for CJP groove welds to minimize weld metal volume. This reduces labor, shrinkage, and the possibility of discontinuities.

43. Design moment connections to elimi-

nate overhead welds in the field. For example, when moment flange plates are field-welded, make the top flange plate narrower than the flange and the bottom flange plate wider than the flange (see recommended minimum shelf dimensions as shown in Figure 8-11 in the AISC *Manual*).

44. 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.

45. Remember that a CJP groove weld in a directly welded flange connection can be expected to shrink from $\frac{1}{16}$ in. to $\frac{3}{16}$ in. in the length dimension of the beam when the weld cools and contracts. It is especially important when a multi-bay moment frame with CJP groove welds is used. This should be coordinated with the fabricator and erector to establish the appropriate connection detail and erection procedure.

46. Mill tolerances for beams and columns may cause significant misalignments of holes in flange-plated connections when bolts groups are large. Consider shipping the flange plates loose for field welding. This also eliminates additional shimming at these plates.

47. 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 ASMT 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.

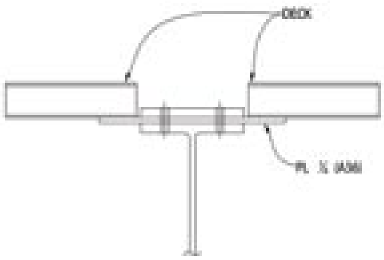

Figure 7. Deck Bearing at Bolted Flange-Plate Connection

48. 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 bolt pretensioning and reaming, if required.

49. As the engineer of record, ask a local fabricator or erector for his/her advice in cases of special situations. This can save time and money down the road, especially for repetitive connections.

50. Remember that inspection immediately drives up the cost and needs to be specified carefully and only as needed. For example, welds that are subject to low stresses or are in compression don't need the same inspection as welds subject to high tensile stresses.

51. As the engineer of record, unless they can be justified for unique conditions, avoid specifying more stringent requirements than established by standard practice and included in the current specifications, standards, codes, and provisions. All procedures have been developed to meet requirements per these documents, and more stringent requirements will lead to establishing new procedures and cost increases (especially when the number of bidders will be reduced).

52. When a new moment connection is made to an existing frame, carefully examine existing conditions including actual plan dimensions, elevations, member sizes, steel grades, steel weldability, etc. Keep in mind that: members could be substituted; moment connections are always very sensitive to the tolerances; and typically it is difficult to reinforce existing members.

53. When sequencing steel frame erection, consider moment frames for stability. This may allow savings for temporary bracing.

54. Use correct sequences when making fully restrained moment connections. For example,
    - ➜ At directly welded moment connections at columns, pretension bolts at web the connections *after* the welds at the flanges are made and allowed to shrink. Otherwise, the weld shrinkage would cause significant amount of preload in the bolts and welds. Provide horizontal short-slotted holes in the web connections.
    - ➜ At moment connections with CJP groove welds at the web and flanges (e.g., beam splice), weld the web first to reduce additional stress due to restraint.

55. The perfect design will not eliminate all mistakes. Good connections substantially reduce the number of shop and field problems, but remember: people make mistakes, actual tolerances may be larger than expected, and problems may arise. As the engineer of record:
    - ➜ Request an as-built report. It clearly shows the problem and eliminates misunderstanding or misinterpretation.
    - ➜ Discuss possible solutions with a fabricator or erector; chances are they already have a suggestion.
    - ➜ Consider the cost; labor is expensive and material is cheap.
    - ➜ Proceed with fast decisions and approvals to continue the work.
    - ➜ Remember that not every field problem requires correction.

We're down to the last five. Here are some suggestions for moment frame connections when seismic provisions must apply:

1. Use prequalified connections included in AISC 358 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*, included in the AISC *Seismic Design Manual* (Section 6.2).

2. When heavy rolled W-shapes are required at moment connections with CJP groove welds, don't forget about special requirements for the material covered in the AISC *Seismic Provisions*, Section 6.3.

3. Detail weld access holes at CJP groove welds to comply with Tables 1-1 and 1-2 in the AISC *Seismic Design Manual*.

4. Provide items required per the AISC *Seismic Provisions* and Section 5 in particular.

5. As for non-seismic applications, favor extended end-plate moment connections over directly welded moment connections. Refer to AISC 358 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* for design requirements for these connections.

Moment connections vary greatly, loads can be large, and framing conditions can be complex. However, as with all other connections, the best effect is achieved when the design, fabrication, and erection expertise is combined together. MSC