

# 30 Good Rules for Connection Design

By Carol Drucker, S.E.

Practical tips for designing economical, fabricator- and erector-friendly steel connections

**W**hen working on a steel construction project, the Structural Engineer of Record (SER) can create an economical structure that is fabricator and erector friendly, especially with good communication and teamwork on the project design and construction teams.

If the fabricator designs the connections, sealed calculations usually need to be submitted to the SER for review. Also, the load criteria need to be given on the contract documents. For moment and axially loaded connections, the actual design load or a percentage of the member's capacity is given. For shear connections, one of three methods (or a combination) is used:

- The actual beam shear loads are given.
- A beam shear-load schedule for different beam depths are given.
- The SER specifies that the shear load will be determined based on a percentage of the beam's uniform distributed load capacity (UDL).

If a beam shear-load schedule or a percentage of UDL is given, the actual criteria specified by the SER should satisfy the required loading. However, the criteria should not be so conservative as to unnecessarily create the need for excessive typical shear connections (more than two vertical rows of bolts or more than full-depth connections).

For the most economical structure, it is preferred that shear loads be given on drawings for every beam. There are also conditions where a beam shear-load schedule or the UDL method do not apply and actual loads should be given:

- At beams with concentrated loads near the beam end.
- At beams sized for serviceability requirements rather than strength requirements.

- At infill beams that support a relatively small area.
- At beams with axial load.

At relatively short beams, the UDL method usually gives overly conservative shear loads, and an upper-limit shear load or the actual beam shear load should be given.

Below are some recommendations for cost-effective connection design that also could reduce problems in the field:

## 1. At bolted flange-plated connections, the flange plate should not be the same width as the beam flange.

Allow at least a 1/2" difference on each side of the flange plate at bolted flange-plated connections. If bolt holes misalign in the field, there will be a sufficient shelf dimension to place longitudinal fillet welds to compensate for the missing bolts.

## 2. Use oversized holes in beam moment and brace connections.

At flange-plated moment connections or brace connections with slip-critical bolts, oversized bolt holes are often preferred over standard bolt holes. Although standard holes give a greater bolt capacity, oversized holes permit more erection tolerance and reduce field problems. Typically, oversized holes are used in the detail material and standard holes in the main member.

## 3. 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.

## 4. Limit the number of bolt diameters to one size (a maximum of three sizes if necessary).

This reduces field errors as well as fabrication errors with drill lines that have manually changed spindles. Some fabri-

cators/erectors keep at least a 1/4" difference between bolt sizes to prevent uncertainty.

## 5. Avoid different grade bolts with the same diameter.

## 6. Use bolted connections instead of field-welded connections.

Bolted connections are quicker to erect and less costly. Welded connection delays result from the absence of a qualified welder or welding platforms, or cold windy weather.

## 7. Try to use fillet welds instead of partial-penetration or full-penetration welds.

Fillet welds are less expensive, since no base-metal preparation is needed. Also, partial- and full-penetration welds usually require more weld metal and inspection.

## 8. Limit the maximum fillet weld size to 5/16" (especially in the field).

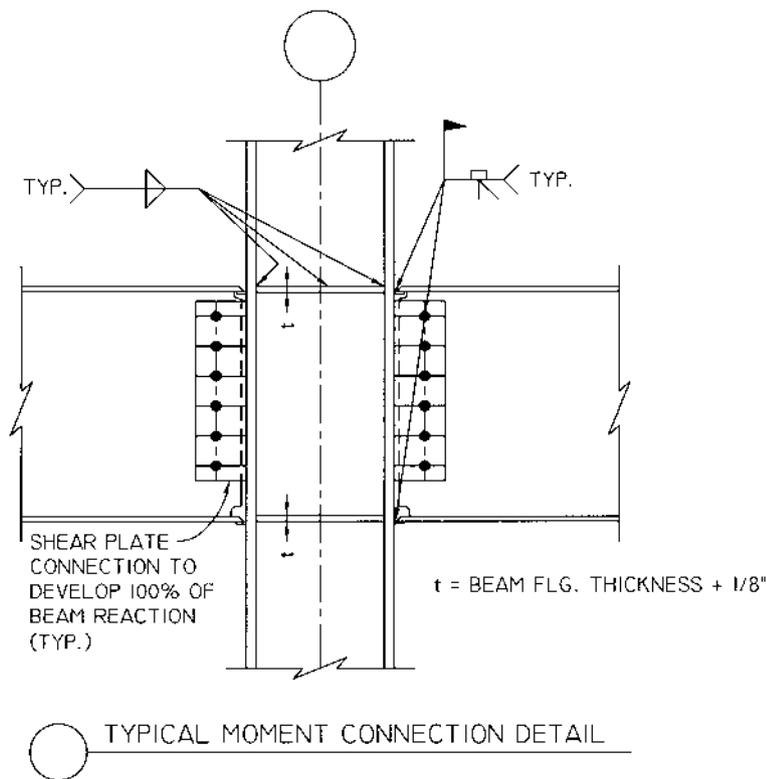
This is the maximum-size weld that can be completed in a single pass using the shielded-metal arc-welding (SMAW) process. Smaller, longer welds are preferred over larger, shorter welds.

## 9. 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.

## 10. Use full-penetration welds only when necessary.

Full-penetration welds cost more than partial-penetration or fillet welds due to increased material preparation, testing requirements, weld-metal volume and material distortion. Use fillet welds (pro-



○ TYPICAL MOMENT CONNECTION DETAIL

Use shear plate connections at beam-to-column flange-plated moment connections (see tip 20).

vided that the fabricator does not determine them excessively large) or partial-penetration welds when joint strength is adequate for the applied load. Full-penetration welds at HSS members are difficult due to the need for backing bars.

## 11. 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  $d-2t_f$  for beam overrun in depth and variation in beam-flange thickness. Per AISC *Specification* Section M, gaps not exceeding  $1/16$ " are permitted in bearing connections.

## 12. Avoid excessive connections.

Connections should be designed to actual load requirements. The SER should not specify that connections be designed for the members' full capacity if not nec-

essary. Excessive connections increase construction cost and the possibility of problems in the field. Over-welding can damage the steel.

## 13. Specify only prequalified welds.

The welding procedure specification (WPS) for a weld not American Welding Society (AWS) prequalified must be tested. To avoid the cost, time and confusion of qualification, use only prequalified welds.

## 14. Minimize weld volume at full- and partial-penetration welds.

The weld configuration with the least weld volume is the most economical. For weld configurations with double-sided preparation, consider the additional cost of material preparation, since it could offset the cost savings of less weld metal. For full-penetration welds, it is economical to prep one side of plates with a thickness less than 1" and to prep both sides of plates with a thickness greater than or equal to 1".

## 15. Provide proper clearance for bolted and welded connections.

AISC's *LRFD Manual of Steel Construction*, 3rd Edition, Parts 7 and 8, give the necessary clearances for bolted and welded

connections.

## 16. Specify slip-critical bolts only when necessary.

Slip-critical bolts are not always needed at moment and brace connections, and bearing bolts might be acceptable in standard holes. The RCSC (Research Council on Structural Connections) Specification, Section 4.3 gives criteria for slip-critical joints, which are more costly than bearing joints due to additional installation, inspection and faying-surface preparation. Also, more or larger bolts are needed for reduced bolt strength. In certain conditions, fabricators/erectors might prefer slip-critical bolts for oversized holes and greater erection tolerance.

If slip-critical joints and bolts are needed, the steel detailer must indicate them on the erection and shop drawings. The detailer should indicate masking requirements in the bolt-hole area to keep the faying surface free of coatings from inadvertent overspray of unqualified paint.

## 17. Allow bearing bolts at the shear connection of flange-welded moment connected beams.

At directly welded moment connections or flange-plated moment connections welded to the supported beam, bearing bolts can be used in the beam-web shear connection. Since the flange weld and the web bolts are not on the same faying surface, the web bolts do not need to be slip-critical. (See AISC's *LRFD Specification Commentary*, Section J1.9). Also, since the web bolts will be installed prior to the completion of the flange weld, the web bolts should be in bearing.

## 18. Limit the maximum bolt diameter to $1\frac{1}{8}$ " diameter.

Bolts with diameters greater than  $1\frac{1}{8}$ " are difficult to tighten, not commonly used, and costly. ASTM A490 bolts greater than 1" diameter have a disadvantage in slip-critical connections with oversized or slotted holes. Hardened washers with a  $5/16$ " minimum thickness (standard washers are  $5/32$ " thick) are required to cover oversized and short-slotted holes in outer plies. See *RCSC Specification* Section 6.2.

## 19. 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.

## 20. Use shear plate connections at beam-to-column flange-plated moment connections.

If flange plates need to be welded to a column flange, a shear-plate connection is preferred. Since the column needs to be positioned in the shop for the flange-plate welds, the shear plate can be readily welded.

At fully restrained moment connections, a typical shear-plate connection can be designed for shear only (and eccentricity in the shear connection can be ignored). The joint rotation is considered resisted by the flange connections.

## 21. Use double-angle connections at directly welded beam-to-column moment connections.

To avoid double-cutting the beam, double-angle connections (bolted to the column) are preferred over shear-plate connections at directly welded moment connections. At directly welded moment connections, the beam flange typically is set back  $\frac{1}{4}$ " from the face of the column flange for welding purposes. If a shear-plate connection is used, the beam web needs to be set back  $\frac{1}{2}$ " to clear the shear-plate welds, and the beam needs to be double-cut. However, for double-angle connections, the beam-web set-back can be reduced to permit a single beam cut.

## 22. Allow for field adjustment in one direction only at bolted connections.

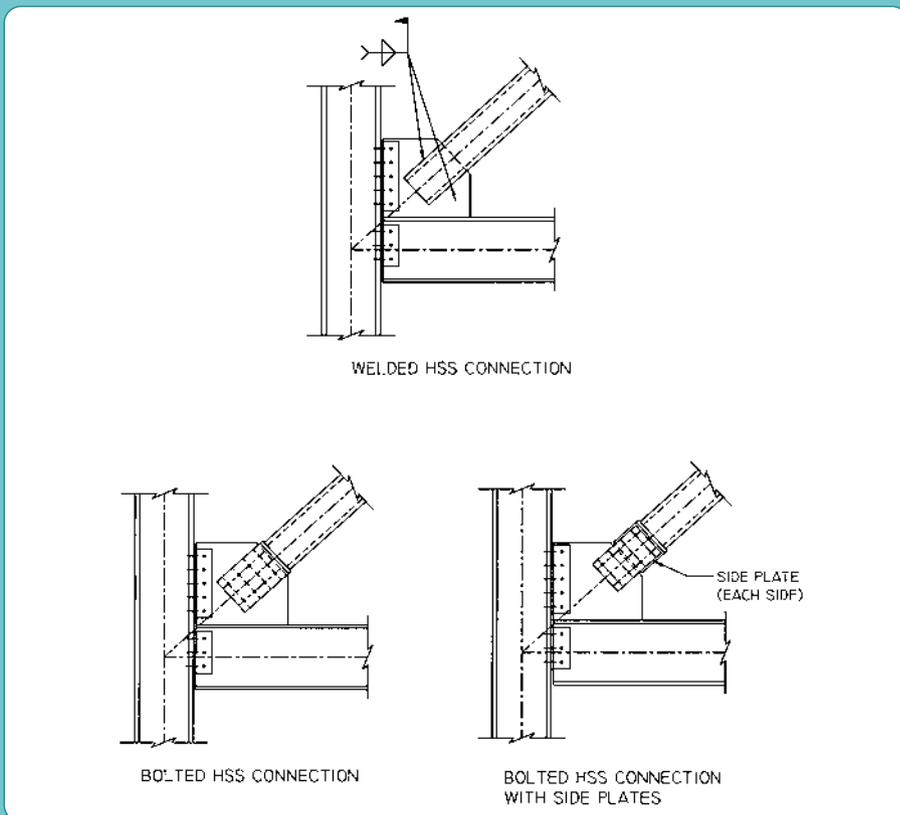
If slotted holes are needed at a bolted connection for field adjustment, the adjustment should be in only one direction. This permits adjustment in one direction while the adjustment in the other direction is made.

## 23. Cope or block beams instead of cutting flush and grinding smooth.

It is less costly to cope or block a beam. If the beam flange must be cut flush, then specify grinding smooth only when needed.

## 24. For shear-plate connections to HSS columns, use face-mounted single-plate connections instead of through-plate connections.

Avoid through-plate shear connections, which cost more than face-mounted single-plate connections. AISC's *Hollow Structural Sections Connections Manual* discusses face-mounted single-plate con-



Consider bolted HSS bracing connections (see tip 29).

nections and how to evaluate them. Face-mounted single-angle connections are also economical shear connections to HSS columns.

## 25. 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 through-plate 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.

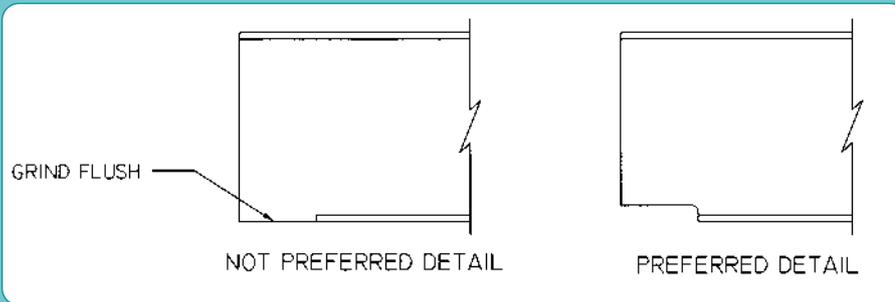
## 26. Clearly show column stiffener and doubler requirements.

The SER should check columns for stiffeners and doublers, and if possible, increase column sizes to eliminate required column reinforcement (especially doublers). If the fabricator is responsible for checking the columns, the actual beam dead, live and wind moments are needed at the time of bidding to determine the required reinforcing or alternate column sizes.

Sometimes typical details on the SER's structural drawings can include problems, such as:

1. Stiffeners might be provided unnecessarily.
2. It is unclear if the stiffener material is Grade 36 or Grade 50.
3. The stiffener weld size is not given.
4. The full-penetration field welds could be excessive.
5. It is unclear if the SER checked if column doublers are needed.

If no information is shown on the contract documents, it might be assumed incorrectly that no column reinforcing is needed.



Cope or block beams instead of cutting flush and grinding smooth (see tip 23).

**27. For simple shear connections, allow the fabricator to choose the AISC standard shear connection that is most economical for their shop.**

Each fabricator has a preference for a standard shear connection, based on shop equipment, ease of erection and specified shear reactions. Usually, any AISC standard shear connection is acceptable. Concerns about shear-plate connections or single-angle connections at laterally unbraced beams are due to the low torsional resistance of these connections. However, most beams are laterally braced due to the attachment of metal deck or perpendicular beams.

**28. At W-shape brace connections, connect only the brace flanges to the gusset plate.**

For ease of erection and fabrication, it is preferred that only W-shape flanges connect to the gusset plate. If the gusset plate and the web of the brace are not the same thickness, then filler plates are necessary at the web connection. If the web needs to be connected, connect it on one side so the brace can be erected easily from it. If only the brace flanges are connected, the section must be checked for shear lag.

**29. Consider bolted HSS brace connections.**

HSS braces commonly are shown slotted and welded to the gusset plate. To eliminate the need to field-weld the HSS brace, the HSS brace can be bolted to the gusset plate. A welded tee end or a slotted HSS/gusset-plate connection could be used. Design procedures for both connections are in AISC's *HSS Connection Manual*. To eliminate connection eccentricity, side plates also can be used.

**30. Whether using ASD or LRFD, consider information in AISC's *LRFD Manual* that can simplify connection design and increase connection capacity.**

1. Appendix J gives the fillet-weld strength-increase equation for welds loaded at an angle relative to the longitudinal axis of the weld.
2. Eccentrically loaded bolt-group and weld-group tables for loads applied at an angle from the vertical position. ★

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For more information on connection design, see the article "57 Tips for Reducing Connection Costs" by Victor Shneur, P.E., in the July 2003 issue of *Modern Steel Construction* (available online at [www.modernsteel.com](http://www.modernsteel.com)).

Also, check out AISC's new Detailing Cards, available by calling 800.644.2400 or by visiting [www.aisc.org/bookstore](http://www.aisc.org/bookstore) for \$30 for AISC members (\$60 for non-members). AISC Detailing Cards are a quick and handy reference for the shape, gage, and strength properties of steel sections and connectors.