steelwise

A SHEAR CONNECTION EXTENDS ITS REACH

The extended configuration of single-plate shear connections: questions and answers.

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THE EXTENDED CONFIGURATION of the single-plate shear connection is growing in popularity and use.

Designers like the ability to use a shear plate for "tight" framing conditions; fabricators find the connections to be simple and economical; and erectors generally love them due to ease of access and simplicity of erection. Here, we'll address some common questions related to the use of these connections and provide some additional guidance.

Does this connection have a history of acceptable performance? Yes. No one tracks the use of various connection configurations, so the number of extended single-plate shear connections in service cannot be quantified. However, the use of these connections certainly predates the formal procedure presented in the AISC *Steel Construction Manual* (available at **www.aisc.org/manual**). The extended configuration of the single-plate shear connection first appeared in the 13th Edition of the *Manual*. At this point, over a decade has passed since the 13th Edition was published (the 15th Edition was released recently) and I know similar connections were used for at least a decade prior to 13th Edition. Earlier editions of the *Manual* also show pictorially what look to be extended single-plate shear connections, though no design procedure was presented.

Item 6 under the Design Checks shown in Part 10 of the *Manual* states: "Ensure that the supported beam is braced at points of support." What exactly does this mean and why is it required? Section F1.(b) of the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360, available at **www.aisc.org/specifications**) states: "The provisions in this chapter are based on the assumption that points of support for beams and girders are restrained against rotation about their longitudinal axis." The design procedure assumes that this restraint need not be provided by the single-plate shear connection. Many beams encountered in practice are continuously braced. In such cases, the torsional strength and stiffness of the end connection are immaterial.

The brace must satisfy the requirements of Appendix 6 of the *Specification* and should be evaluated relative to the beam, not the extended single-plate shear connection. Part 2 of the *Manual* states: "In general, adequate lateral bracing is provided to the compression flange of a simple-span beam by the connections of infill beams, joists, concrete slabs, metal deck, concrete slabs on metal deck and similar framing elements." If such elements can be considered to provide continuous bracing relative to the design of the beam, then Item 6 can be assumed to be satisfied.

Are the checks in the 14th Edition *Manual*, under the heading "Requirement for Stabilizer Plates," intended to ensure that Item 6 is satisfied? No. Both sufficient strength and stiffness must exist at points of support in order to apply the provisions in of Chapter F of the *Specification*. The stabilizer plate checks shown in the *Manual* only consider strength. In fact, the derivation of these checks—presented in the Second Quarter 2011 *Engineering Journal* article "On the Need for Stiffeners for and the Effect of Lap Eccentricity on Extended Shear Tabs" (**www.aisc.org/ej**)—assume that a slab is present. It should also be noted that these checks, though conservative, will rarely govern. In fact, the stabilizer plate check does not appear in the 15th Edition *Manual*.

The AISC Design Example addressing this connection has been revised several times. The determination of the flexural strength of the plate, as shown in the Design Example, has sometimes been based on a plastic section modulus and sometimes based on an elastic section modulus. Which is correct? The plastic section should be used. The design procedure was developed to use the plastic section modulus of the plate (see the Second Quarter 2009 *Engineering Journal* article "Design of Unstiffened Extended Single-Plate Shear Connections," available at **www.aisc.org/ej**). The confusion arose from the fact that rather than writing new procedures to address the stability of the plate, it was decided that we

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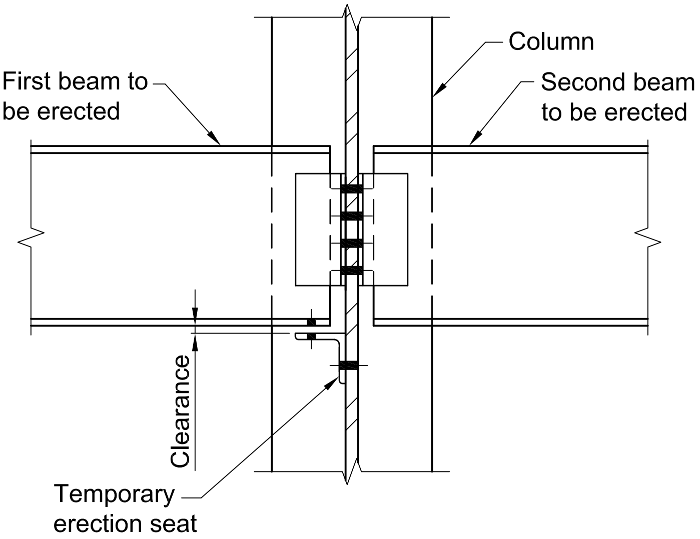
would simply reference the procedures described for double-coped beams. These procedures were based on work by Cheng and Yura, which was developed at a time when the use of the elastic section modulus was still very common (see "Local Web Buckling of Coped Beams" in the August 1986 issue of ASCE's *Journal of Structural Engineering*). However, in the 13th Edition *Manual*, the range of cope dimensions was extended by employing a general flexural buckling check. Though this check assumed a plastic distribution of stress for consistency with the other checks, it was applied with the elastic section modulus (see the Third Quarter 2017 *Engineering Journal* article "A Direct Method for Obtaining the Plate Buckling Coefficient for Double Coped Beams," available at **www.aisc.org/ej**).

In the meantime, the *Specification* provisions of F11 addressing the flexural strength of rectangular bars were added. In the 15th Edition *Manual*, the design procedures for copes based on the elastic model are replaced with procedures based on the F11 provisions being modified to account for the boundary conditions at either end of the cope or extended single-plate connection. Not only will this eliminate a source of potential confusion, but it will also ensure that the potential for lateral-torsional buckling of the plate is properly considered.

The *Manual* includes a figure (Figure 10.2, at right) showing a column as the support and states: "The design procedure for extended single-plate shear connections permits the column to be designed for an axial force without eccentricity. In some cases, economy may be gained by considering alternative design procedures that allow the transfer of some moment into the column." Can extended single-plate shear connections be used with support beams as well as columns? Yes. The Second Quarter 2009 *Engineering Journal* article mentioned previously includes a discussion of serviceability and erection consideration when attaching to only one side of a support beam. It also includes a design example for a beam. The *Manual* statement refers only to a column, since, owing to the low torsional strength of wide-flange beams, no economy would be gained by transferring some moment into a support beam.

More generally, the absence of a specific configuration in the *Manual* or Design Examples is not intended to discourage



or prohibit its use. The *Manual* cannot address every condition that might be encountered in practice.

If the end of the beam is not braced, is the design procedure in Part 10 still applicable? No. The design procedure assumes that the end of the beam is braced. The beam can be braced by an actual brace or by the slab, deck or other suitable means. The cope checks in Part 9 of the *Manual* also assume that the cope is braced at both ends of the cope. This has always been the case and has been clarified in the 15th Edition *Manual*. Since the design procedure for the extended single-plate shear connection references the cope checks, it must satisfy the same assumptions.

Also, as stated previously, if the flexural strength of the beam is to be determined using Chapter F of the *Specification*, then there must be adequate torsional restraint at the supports. If the beam is not braced at its end, then the strength and stiffness of the plate must be evaluated. If there is insufficient strength and/or stiffness, then this must be accounted for in the design of the beam. Neither the *Specification* nor the *Manual* address this problem.

Table 8-12
Approximate Number of Passes for Welds

Weld Size* in.	Fillet Welds	Single-Bevel Groove Welds (Back-Up Weld not Included)		Single-V Groove Welds (Back-Up Weld not Included)		
		30° Bevel	45° Bevel	30° Groove Angle	60° Groove Angle	90° Groove Angle
3/16	1	–	–	–	–	–
1/4	1	1	1	2	3	3
5/16	1	1	1	2	3	3
3/8	3	2	2	3	4	6
7/16	4	2	2	3	4	6
1/2	4	2	2	4	5	7
5/8	6	3	3	4	6	8
3/4	8	4	5	4	7	9
7/8	–	5	8	5	10	10
1	–	5	11	5	13	22
1 1/8	–	7	11	9	15	27
1 1/4	–	8	11	12	16	32
1 3/8	–	9	15	13	21	36
1 1/2	–	9	18	13	25	40
1 3/4	–	11	21	13	25	40

*Indicates plate thickness for groove welds.

If my beam is not sufficiently braced at the end, should I opt for a torsionally stiff connection configuration? Yes, but there may also be other considerations. Bracing is not mentioned in the *Manual* for any of the other shear connections discussed in Part 10. It has long been established practice to provide a connection that is at least half the depth of the beam and implicitly assume that there is sufficient torsional restraint. However, the presence of a cope could invalidate this assumption. Also, as stated previously, the cope checks in Part 9 assume a brace point at the end of the cope. Even the strongest and stiffest connection will not provide sufficient restraint if it attaches to a coped section that does not possess sufficient strength and stiffness.

Why does the section "Requirement for Stabilizer Plates" no longer appear in the *Manual*? The checks that were included in the 14th Edition *Manual* were rational but conservative and will rarely govern. They also were misinterpreted by some engineers as checks on the stability of the beam. This was not the intent, as discussed above. If an engineer wants to check the suitability of the extended tab for unusual conditions, then they can refer to the original paper, which is still referenced in the *Manual*. For typical conditions, there is no need to perform the checks. These are among some of the reasons the checks were removed.

When the strength of the extended single-plate shear connection is insufficient to carry the design loads and is governed by buckling, is it more economical to increase the thickness of the plate or to add stiffeners? It is generally more economical to increase the thickness of the plate. One consideration is that the weld between the plate and column is determined from the plate thickness ($^5/_8 t_p$). Up to a ½-in.-thick plate, the weld size will be $^5/_{16}$ in. or less on each side of the plate. This is a single-pass weld, the most economical arrangement. As the plate thickness increases beyond ½ in., the number of passes will increase to approximately three per weld up to a $^3/_8$-in. weld and four passes per weld up to a ½-in. weld. The number of passes for welds can be estimated from *Manual* Table 8-12 (above). Since it is not common to use a $^9/_{16}$-in. plate for a plate design thickness beyond ½ in., a ¾-in.-thick plate would be used, requiring four passes per weld. A 1-in.-thick plate will require $^5/_8$-in. welds and six passes. Therefore, plates greater than ½ in. thick will cost more than four times as much as a comparable but thinner plate. This cost increase must be compared to the increased cost of providing stability plates, which will involve cutting, fitting and welding two additional pieces.

Providing a thicker plate to satisfy stability requirements can lead to concerns when attaching to a relatively light support. The best way to avoid such complications is to design the connections based on the actual loads, as opposed to indirect methods such as reactions determined from the maximum uniformly distributed load that can be supported by the beam. However, even when the actual loads are used, a 1-in. plate might attach to a $^3/_{16}$-in. support web. Meeting the $^5/_8 t_p$ weld size recommendation might result in a $^5/_8$-in. weld to either side of the $^3/_{16}$-in. web. In such cases, it is important to remember that the model assumes only shear is transmitted at the support and that the supporting beam has already been checked against this demand. It is also important to realize that steel generally does not fail in the through-thickness direction. If a check of the supporting member is to be performed, it should consider only the vertical shear reactions. It should also be noted that the changes made relative to the cope checks in Part 9 of the *Manual* (discussed above) should allow for thinner plates, likely making this less of a concern than it was in the past.

When substantial stiffeners are provided, must the support be designed for additional eccentricity? In the First Quarter 2016 *Engineering Journal* article "Analysis and Design of Stabilizer Plates in Single-Plate Shear Connections" (www.aisc.org/ej) the authors consider several different arrangements. Their preference is to keep the stability plates small and flexible so that any effect they may have is clearly negligible. It is more common to simply use stiffeners that roughly fill the area between the column flanges, similar to typical stiffeners used at beam-to-column moment connections. The article and subsequent presentations indicate that the authors feel the additional restraint could be detrimental to the column.

Whether the effect of this additional restraint needs to be considered in the design of the column is a matter of engineering judgment. The discussion in Part 9 of the *Manual* titled "Eccentric Effect of Extended Gages" argues that while potentially adding moment to the column, the increased flexural stiffness will also add restraint. These are, to some degree, offsetting effects, and the *Manual* seems to suggest that the eccentricity need only be considered in the connection. Another justification for this approach is that adding material or restraint should not weaken the structure. Though there are exceptions to this rule, it generally holds true for inherently ductile materials like steel.

Must the $\frac{5}{8}t_p$ recommendation for weld size always be met? No. If the weld is sized based on $\frac{5}{8}t_p$ (and all of the other recommendations are followed) then it can be assumed that the *Specification* requirements have been met. However, meeting the *Specification* requirements may not require that the weld size equals or exceeds $\frac{5}{8}t_p$. Section B3.4a of the *Specification* states: "A simple connection transmits a negligible moment. In the analysis of the structure, simple connections may be assumed to allow unrestrained relative rotation between the framing elements being connected. A simple connection shall have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure." The weld size recommendation assures that an end rotation of about 0.03 radians can be accommodated in a ductile manner. This is a very large end rotation, and beams producing this level of end rotation will likely be unserviceable due to large deflections.

For some conditions, rotations may be deemed negligible. This might occur for deep beams with short spans, such as transfer girders, or for beams that are part of a vertical brace connection (assuming large seismic drifts are not a consideration).

It should also be noted that the rotation determined by the analysis of the structure need not be accommodated through flexural yielding of the plate alone, as is assumed in the *Manual* procedure. When connecting to a flexible support such as a beam or column web with no connection on the opposite side, rotation may be accommodated through local deformation of the support. Movement of the bolts within slotted holes or plowing of the bolts may also accommodate end rotations, as is accounted for in the design procedures for conventional single-plate shear connections. However, slots become less effective when there are two or more vertical rows of bolts in the connection.

There are other rational approaches that can be used as well. The *Manual* provides a simple and relatively foolproof design procedure applicable to a wide range of conditions; other approaches can be used for specific conditions.

Design Considerations

Keep these points and recommendations in mind when designing with extended single-plate shear connections:

- At this point, it is safe to say that thousands of extended single-plate shear connections are in service and performing well in the United States.
- An extended single-plate shear connection can be used with either a beam or a column as the supporting member.
- Extended single-plate shear connections are often a good choice when the unstiffened plate is up to ½ in. thick. Beyond this thickness, the choice of connection type should be more carefully considered.
- Extended single-plate shear connections can be used, without concerns related to stability, at points of support whenever continuous bracing can be assumed, which is often the case when designing typical buildings.
- Extended single-plate shear connections may not be the right choice when torsional end reactions exist.
- Extended single-plate shear connections may not be the right choice when there is uncertainty about the stability of the beam. This may occur when no diaphragm exists or the diaphragm is not sufficiently connected to the beam (e.g., some conditions with metal deck). ■