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

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 nd 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.

Yes. No one tracks the use of various connection configurations, so the number of extended single-plate shear connections in service cannot be quanti ed. However, the use of these connections certainly predates the formal procedure presented in the AISC *C s M* (available at). The extended con guration of the single-plate shear connection rst appeared in the 13th Edition of the *M* . 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 *M* also show pictorially what look to be extended single-plate shear connections, though no design procedure was presented.

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Section F1.(b) of the AISC *B s* (ANSI/AISC 360, available at) 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 and should be evaluated relative to the beam, not the extended single-plate shear connection. Part 2 of the *M* states: "In general, adequate lateral bracing is provided to the compression ange of a simple-span beam by the connections of in ll 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 satis ed.

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No. Both suf cient strength and stiffness must exist at points of support in order to apply the provisions in of Chapter F of the . The stabilizer plate checks shown in the *M* only consider strength. In fact, the derivation of these checks—presented in the Second Quarter 2011 *E J* article "On the Need for Stiffeners for and the Effect of Lap Eccentricity on Extended Shear Tabs" ( )—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 *M* .

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 *E J* article "Design of Unstiffened Extended Single-Plate Shear Connections," available at). The confusion arose from the fact that rather than writing new procedures to address the stability of the plate, it was decided that we 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 *J* *E*). However, in the 13th Edition *M* , the range of cope dimensions was extended by employing a general exural 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 *E* *J* article "A Direct Method for Obtaining the Plate Buckling Coef cient for Double Coped Beams," available at).

In the meantime, the provisions of F11 addressing the exural strength of rectangular bars were added. In the 15th Edition *M* , the design procedures for copes based on the elastic model are replaced with procedures based on the F11 provisions being modi ed 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.

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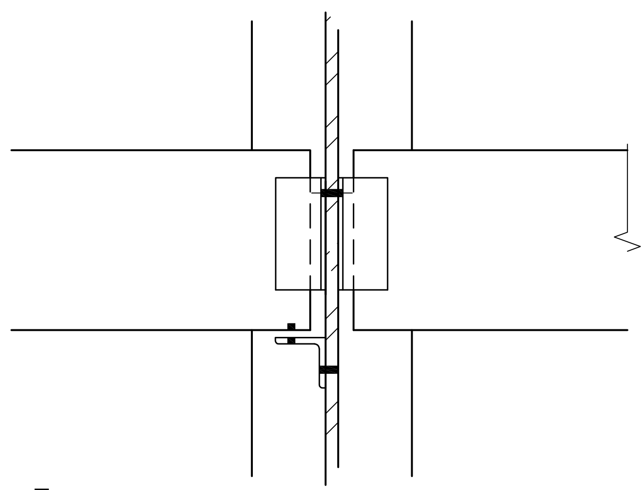
Yes. The Second Quarter 2009 *E* *J* article mentioned previously includes a discussion of serviceability and erection consideration when attaching to only one side of a support beam. It also includes a design example for a beam. The *M* statement refers only to a column, since, owing to the low torsional strength of wide- ange beams, no economy would be gained by transferring some moment into a support beam.

More generally, the absence of a speci c con guration in the *M* or Design Examples is not intended to discourage or prohibit its use. The *M* cannot address every condition that might be encountered in practice.



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[illegible] **10** 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 *M* also assume that the cope is braced at both ends of the cope. This has always been the case and has been clari ed in the 15th Edition *M* . 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 exural strength of the beam is to be determined using Chapter F of the , 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 insuf cient strength and/or stiffness, then this must be accounted for in the design of the beam. Neither the nor the *M* address this problem.

Yes, but there may also be other considerations. Bracing is not mentioned in the *M* 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 suf cient 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 suf cient restraint if it attaches to a coped section that does not possess suf cient strength and stiffness.

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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 (⅝). 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 ⅜-in. weld and four passes per weld up to a ½-in. weld. The number of passes for welds can be estimated from *M* Table steelwise