Design of Unstiffened Extended Single-Plate Shear Connections

LARRY S. MUIR and CHRISTOPHER M. HEWITT

Extended single-plate shear connections (Figure 1) offer many advantages that simplify the construction process. Because the connection to the supported member is moved clear of the support, coping of the supported member is not required and the only fabrication process required for the supported member is drilling or punching. Also, because bolted connections are only used in the connection to the supported member, there is no safety concern over the use of shared bolts through the web of the support. Additionally, in some instances, extended single-plate connections are the only practical solution to a framing problem, such as the case of a member framing into the weak axis of a column with continuity plates.

The rigidity of single-plate connections at the support has always been a gray area. Designers have often been concerned about a considerable, unanticipated moment that could be developed in the connection, which could then result in either a moment delivered to the column that the column has not been designed to resist or a sudden rupture of either the weld or the bolts. Section B3.6a of the AISC *Specification for Structural Steel Buildings,* hereafter referred to as the AISC *Specification,* requires that simple shear connections have sufficient rotational capacity to accommodate the required beam end rotation. This paper will address each of these concerns and will present a general design procedure for extended single-plate shear connections.

This paper outlines the background and development of the design procedure for extended single-plate shear connections presented in the 13th Edition AISC *Steel Construction Manual,* hereafter referred to as the AISC *Steel Manual.* While the method presented in this paper has been determined by the AISC Committee on Manuals and Textbooks to be suitable for all cases, other rational methods may be used at the discretion of the designer.

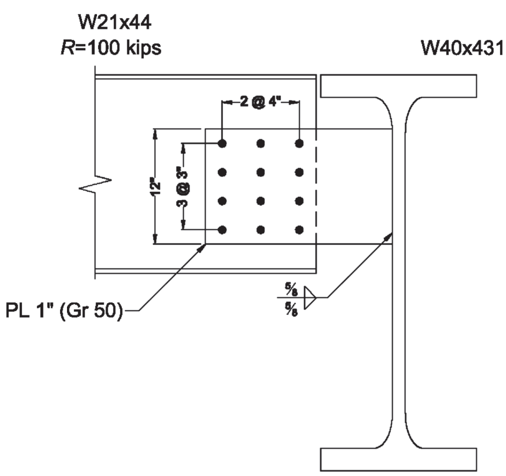

Fig. 1. Extended single-plate shear connection.

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HISTORY OF USE AND RESEARCH

Extended single-plate shear connections have a long history of use. Illustrations of the use of extended single-plate shear connections have been included in the AISC *Steel Manual* since 1992, and they have been used by designers for several decades. Despite a relatively long history of use, a well-defined, simple and rational design procedure has never been included in the AISC *Steel Manual,* and the design procedure was largely left to the discretion of individual engineers.

Fearing that the plate might buckle or that the weld might fracture, many designers have chosen to detail the connections with top and bottom stiffening plates or to extend the plate and connect it to the top and bottom flanges of a supporting girder (Figure 2). Ironically, testing has shown that extending the plate vertically in this manner could actually result in a lower plate buckling strength (Sherman and Ghorbanpoor, 2002) and is, in many cases, unnecessary.

Sherman and Ghorbanpoor (2002) conducted testing on extended single-plate shear connections and also proposed a design procedure. The procedure is predicated on the use of top stiffening plates and a single column of bolts. The bolts are designed based on an empirically derived eccentricity related to the number of bolts, similar to the contemporary design procedure for conventional single-plate shear connections. Strongly tied to the empirical test data, the procedure does not adequately address the needs of the practicing engineer for all cases.

ESTABLISHING A MODEL FOR THE CONNECTION DESIGN

A beam connected to a support by a single-plate shear connection is an indeterminate system. The moment applied to the connection will depend on many factors, including the distance that the connection extends from the support and the relative stiffnesses of the supported beam, the connection and the support. Though it might be assumed that the moments at the support are bounded by those predicted by the pinned-end beam model at the low end, and those predicted by the fixed-end beam model at the high end, even this is insufficient. There is a possibility that the support, or connection, at one end of the beam could be quite flexible, while the other end is extremely rigid. In such a case the stiffer end would be subjected to a moment greater than the fixed-end beam moment. The upper bound then becomes the moment developed at the fixed end of a propped cantilever. Even if the stiffnesses of the supports and the beam could be reliably predicted, determining the stiffness of the connection is problematic. Not only must the stiffness of the plate be determined, but factors such as bolt slip and bearing deformation must also be accounted for in order to accurately predict the distribution of the moments in the system.

Without an accurate prediction of the moment distribution, establishing a design procedure for the connection might seem intractable. Fortunately, instead of trying to predict the behavior of the connection in service, an assumed model can be established and then steps can be taken to control the behavior to support the assumptions. This approach is supported by the lower bound (static) theorem, which states:

> If an equilibrium state can be found which does not violate the yield condition, then however 'unlikely' that state may seem to be, the structure is safe. (Baker and Heyman, 1969)

Thus, the applied external forces in equilibrium with the internal force field are less than or, at most, equal to the applied external force that would cause failure, provided that all the limit states are satisfied and sufficient ductility exists to allow redistribution of the forces.

The most logical model to use is the pinned-end beam model (Figure 3) since it replicates the typical assumptions made during the main member design. The pinned-end model assumes that the connection delivers only the shear reaction from the supported beam to the support. Based on this assumption, the bolt group will be subjected to a moment equal to the shear reaction multiplied by the distance from the support to the center of the bolt group.

ANTICIPATED BEHAVIOR

As stated earlier, the lower bound theorem is predicated on sufficient ductility being present to redistribute the loads. To accomplish this, the plate, as the most ductile element in the connection, is used as a fuse to shed unwanted moments prior to rupture of either the bolts or the weld. To describe this behavior the system can be modeled as a fixed-end beam of varying cross section. As the beam is loaded moments will be produced at supports, resulting in a moment diagram similar to Figure 4.

As the load increases, the possibility exists that the connection will be subjected to a moment greater than its yield

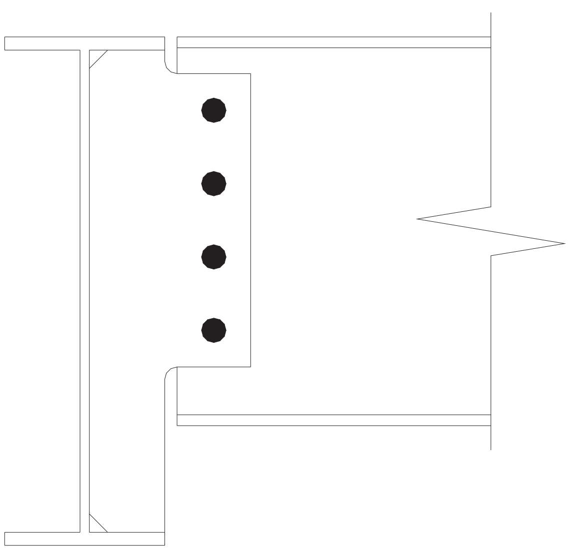


Fig. 2. Single-plate shear connection lengthened to connect to top and bottom flanges of support.

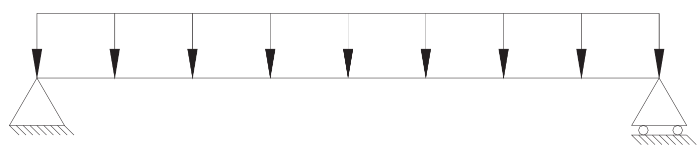


Fig. 3. Simply supported beam model.

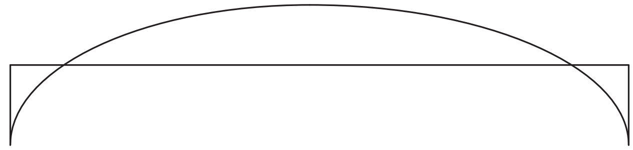


Fig. 4. Moments on a fixed-end beam.

strength. If this occurs, the plate will begin to yield and it will shed the excess moment to the beam, thereby relieving moment from both the connection and the support. The moment distributes to the beam because the yielding of the plate effectively lowers the plate's stiffness. This can be seen by examining the stress-strain curve for steel (Figure 5). When the plate is first loaded its modulus of elasticity, E, is 29,000 ksi. However, as it is loaded beyond the yield stress, the stress-strain curve is no longer linear, and the behavior is better predicted by the tangent modulus of elasticity, E_T, which will be significantly lower than E.

SIZING THE PLATE FOR STRENGTH AND DUCTILITY

In order for the plate to act as a fuse, as described earlier, it must yield without rupturing the bolts or the welds and must possess sufficient strength to support the required loads, thereby setting both a lower and an upper limit on the plate thickness.

Determining the minimum required thickness is straightforward. The limit states of gross shear yielding, net shear rupture, gross flexural yielding, and net flexural rupture all must be satisfied. The plastic section modulus is used in checking both flexural yielding and rupture. This is supported by recent test results (Mohr, 2005). Though not presented as a limit state in the AISC *Specification*, it is recommended that shear and bending interaction be checked. This is done by applying the von Mises yield criterion:

$$\left(\frac{f_v}{\phi_v 0.6F_y}\right)^2 + \left(\frac{f_a}{\phi_a F_y}\right)^2 \le 1.0$$

This can be rewritten in terms of the nominal reaction, R_n, as:

$$\left(\frac{\dfrac{R_n}{dt_p}}{\phi_v 0.6F_y}\right)^2 + \left(\frac{\dfrac{4R_n a}{d^2 t_p}}{\phi_a F_y}\right)^2 \le 1.0$$

$$R_n = \frac{F_y d t_p}{\sqrt{2.25 + 16\left(\dfrac{a}{d}\right)^2}}$$

where
ϕ = 0.90
Ω = 1.67

It should be noted that the current AISC procedure recommends calculating a reduced bending strength related to the applied shear. The AISC approach, also based on the von Mises yield criterion, is theoretically equivalent to the preceding procedure. However, since the AISC equation, $F_{cr} = \sqrt{F_y^2 - 3f_v^2}$, includes both resistance and load terms on the right side of the equation, it may be cumbersome to apply.

Block shear and net shear must also be checked in accordance with Sections J4.3 and J4.2b of the AISC *Specification*.

Additionally, since the edge of the plate in compression may buckle, a stability check must be performed using the relationship (Muir and Thornton, 2004):

$$\phi F_{cr} = 0.90 F_y Q$$

where
$Q = 1$ for $\lambda \le 0.7$
$Q = (1.34 - 0.486\lambda)$ for $0.7 < \lambda \le 1.41$
$Q = (1.30/\lambda^2)$ for $\lambda > 1.41$

$$\lambda = \frac{d\sqrt{F_y}}{10t_p\sqrt{475 + 280\left(\dfrac{d}{a}\right)^2}}$$

If λ is less than or equal to 0.7, the limit state of plate buckling will not control. It is advisable to size the plate to prevent this limit state. However, if plate bucking must be considered as a controlling limit state, it is recommended that the elastic section modulus be used as a conservative method of calculating of the applied bending/buckling capacity of the plate.

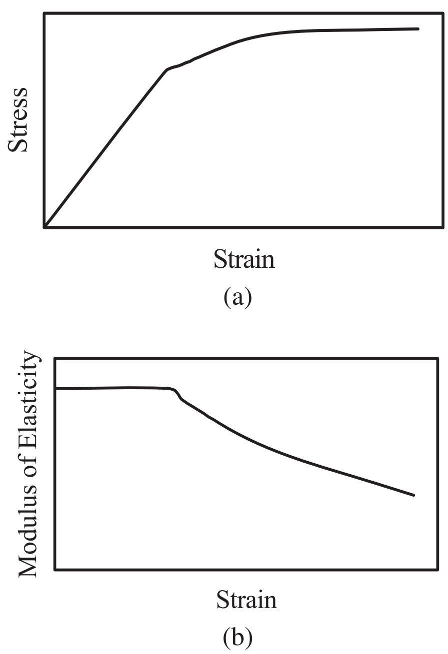


Fig. 5. (a) Stress-strain and (b) modulus of elasticity-strain curves for steel (based on tangent modulus).

Another possible concern is the torsional restraint provided by the extended connection. Most beam designs assume that the beam has full torsional end restraint. In terms of torsional restraint, the extended single-plate shear connection is very similar to a beam coped at the top and the bottom flanges. As with the case of a coped beam, if a slab is present, there should be little concern over the torsional restraint of the connection. However, where torsional restraint is a concern, it can be checked using the procedure presented by the Australian Institute of Steel Construction (Hogan and Thomas, 1988).

The flexural rupture strength of connection elements was investigated by Murray and Mohr (Mohr, 2005). It may be noted that during this testing, a restraining bolt was installed in the plates to prevent buckling prior to developing the full plastic strength of the member. A review of their data indicates that the values for λ varied from a low of 0.538 to a high of 0.685. The preceding theory would not have predicted the buckling in any of the plates. However, there is an important distinction that can be made between the Murray and Mohr tests and extended single-plate shear connections. The Murray and Mohr tests were designed to cause a uniform bending moment on the plates in order to test the moment capacity in the absence of any other factors. It has been shown that for a plate subjected to uniform bending, the critical length at which buckling will occur quickly decreases as the section progressively yields. For nonuniform bending, the decrease is much less pronounced and is negligible for cantilevers. (Baker, Horne, and Heyman, 1956). An extended single-plate shear connection will not be subjected to uniform bending throughout its length and, like a beam designed using the plastic section modulus, will rarely, if ever, experience appreciable yielding in service. Buckling was not a significant problem in either the Murray and Metzger (Metzger, 2006) tests or the Sherman and Ghorbanpoor (2002) tests. Buckling was given as a failure mode for only three of the tests in the Sherman and Ghorbanpoor regime, in two cases where the tab plate was extended to allow welding to the bottom flange of the support beam and a third where it is listed as "a secondary effect of twist" (Sherman, 2002).

As stated previously, the plate acts as a fuse to protect both the bolts and the welds from rupture, thereby allowing the moments to redistribute in an acceptable manner. In order to safeguard the weld, the plate must yield prior to the weld fracturing. Prior to the 13th Edition AISC *Steel Manual*, AISC required that the welds to the support be sized as at least ¾ of the plate thickness. This requirement was developed to ensure that the plate would yield before the weld yielded (Astaneh, 1989). In the latest procedure, this recommendation has been modified, and the new procedure recommends that the plate be sized to yield before the weld ruptures. This is a more logical approach, as weld yield is not a well-defined limit state, and joint separation will not occur until the weld ruptures. The modified requirement calls for the weld be equal to or greater than ⅝ of the plate thickness.

The ⅝ t_p requirement is derived here and was verified by testing (Metzger, 2006), as is discussed later in this paper. In the derivation, the rupture strength of the weld is assumed to be $\sqrt{2/3}\,F_{EXX}$. This is more conservative than the strength of a transversely loaded fillet weld reflected in the AISC *Steel Manual* of $1.5(0.6)F_{EXX}$, but results in a closed-form solution to the problem. It also matches well with the 2006 American Welding Society (AWS) requirement 2.24.1.3, which is intended to prevent the unzipping of welds at tubular connections. AWS states that the ultimate breaking strength of fillet welds with 60 ksi or 70 ksi tensile strength shall be taken as 2.67 times the basic allowable stress. This results in a stress of $2.67(0.3)(70) = 56.1$ ksi compared to $\sqrt{2/3}(70) = 57.1$ ksi with the more convenient value assumed in this paper to obtain a closed-form solution. William A. Thornton (unpublished) originally proposed this approach prior to the AISC *Steel Manual* adopting an increased strength for transversely loaded fillets. The required weld can also be derived using the strength specified in the AISC *Steel Manual*, but a closed-form solution cannot be obtained (though the solutions bound a somewhat smaller required weld, as would be expected.)

As stated previously, the rupture strength of the weld is assumed to be $\sqrt{2/3}\,F_{EXX}$. From this, the shear strength on the weld can be calculated as:

$$F_{shear} = \frac{1}{\sqrt{3}}F_u = \frac{1}{\sqrt{3}}\sqrt{\frac{2}{3}}F_{EXX} = \frac{\sqrt{2}}{3}F_{EXX}$$

The interaction equation for the weld is:

$$\left(\frac{R_n}{2\dfrac{1}{\sqrt{2}}wL_w\dfrac{\sqrt{2}}{3}F_{EXX}}\right)^2 + \left(\frac{R_n e}{2\dfrac{1}{\sqrt{2}}w\dfrac{L_w^2}{4}\sqrt{\dfrac{2}{3}}F_{EXX}}\right)^2$$

$$= \left(\frac{R_n}{\dfrac{2}{3}wL_wF_{EXX}}\right)^2 + \left(\frac{R_n e}{\dfrac{1}{2\sqrt{3}}wL_w^2F_{EXX}}\right)^2 \le 1.0$$

Solving for R_n yields:

$$R_n \le \frac{wL_wF_{EXX}}{\dfrac{\sqrt{3}}{2}\sqrt{3+16\left(\dfrac{e}{L_w}\right)^2}}$$

Similarly for the plate, the interaction equation is:

$$\left(\frac{V_u}{V_{np}}\right)^2 + \left(\frac{M_u}{M_{np}}\right)^2 \le 1.0$$

$$\left(\frac{R_n}{t_p L_w \frac{F_y}{\sqrt{3}}}\right)^2 + \left(\frac{R_n e}{\frac{1}{4} t_p L_w^2 F_y}\right)^2 \le 1.0$$

Solving for R_n yields:

$$R_n \le \frac{t_p L_w F_y}{\sqrt{3 + 16\left(\frac{e}{L_w}\right)^2}}$$

Since the plate must yield before the weld fractures, the following must be true:

$$\frac{t_p L_w F_y}{\sqrt{3 + 16\left(\frac{e}{L_w}\right)^2}} \le \frac{w L_w F_{EXX}}{\sqrt{\frac{9}{4} + 12\left(\frac{e}{L_w}\right)^2}}$$

Solving for the weld size, w, yields:

$$w \ge \frac{t_p F_y \sqrt{3}}{2 F_{EXX}}$$

Assuming $F_y = 50$ ksi and $F_{EXX} = 70$ ksi yields:

$$w \ge 0.619 t_p \cong \frac{5}{8} t_p$$

In a similar fashion, the plate safeguards the bolt group by yielding prior to bolt shear. To ensure that this takes place, a maximum plate thickness is determined based on the strength of the bolt group. To determine the moment strength of the bolt group, the instantaneous center of rotation method is used. To obtain the maximum moment capacity of the bolt group, a moment-only condition is assumed. The instantaneous center of rotation coincides with the center of gravity of the bolt group and the strength can be calculated as:

$$M_{max} = 1.25 F_{nv} A_b C'$$

where

$$C' = \sum \left[L_i \left(1 - e^{-\left(\frac{10 L_i \Delta_{max}}{L_{max}}\right)} \right)^{0.55} \right]$$

It should be noted that the bolt shear strength given in Table J3.2 of the AISC *Specification* includes a 20% reduction to account for the uneven force distribution that occurs in end loaded bolt groups (Kulak, 1987). Since this bolt group is not end loaded, the 20% strength reduction can be neglected. This is the origin of the 1.25 factor preceding the equation. It should also be noted that this is a check to ensure ductility and not strength; therefore, safety factors have not been applied to capacities of either the bolts or the plate. This is similar to the approach used for the weld ductility check shown previously.

Once the moment strength of the bolt group is determined, it can be compared to the flexural yield strength of the plate to obtain a maximum plate thickness:

$$t_{max} = \frac{6 M_{max}}{F_y d^2}$$

The elastic section modulus is used in this check because the plate will begin to redistribute stress after first yielding.

SIZING THE BOLT GROUP

Assuming a pinned-end beam model that delivers only shear to the face of the support, the moment that exists on the bolt group can be calculated as:

$$M_{bolt} = Re$$

The strength of bolt group can be calculated in the typical fashion, using the instantaneous center of rotation method. Though the bolt group is not end loaded, the 20% bolt shear strength reduction inherent in the AISC *Specification* cannot be neglected when designing for strength in practice. Accounting for the 20% reduction is a requirement of the AISC *Specification* as currently written, although from a theoretical standpoint the reduction is not necessary in this case.

SUPPORT ROTATION

Resistance of the support against rotation is often questioned when extended single-plate shear connections are used. In the current design procedure, because all of the connecting elements are designed to resist or otherwise accommodate the entire range of anticipated moments, support rotation and connection deformation from the plate and bolts are serviceability and not strength considerations.

When a rigid support condition exists, all significant rotation of the beam end is accommodated by deformation of the plate and the bolt group, and serviceability need not be considered. A rigid support is one in which the support and the connected beam tend to stay in place or rotate in the same direction. A rigid support may generally be assumed in any of the following cases:

Table 1. Vertical Deflections Due to Short Slotted Holes	
n	**Δ**
2	1.06
3	0.526
4	0.350
5	0.262
6	0.209
7	0.174
8	0.149
9	0.131
10	0.116
11	0.104
12	0.095

- The single-plate connection is attached to a column flange.
- Single-plate connections are framed to both sides of a girder or column web.
- A structural slab is present, attaching to both the girder and the beam flanges.
- The torsional resistance of the girder provides sufficient rotational restraint.

If the support is considered flexible, consideration must be given to the serviceable rotation limit for the connection. Serviceability merits greater concern when short slots are used at a flexible support. It should be noted, however, that the rotation allowed even by short slots is limited by the geometry of the connection. For a connection with a single column of bolts, the angle of rotation can be approximated as:

$$\theta = \tan^{-1}\left(\frac{\left(L_s - d_b\right)}{\left(n-1\right)s - 1/16}\right)$$

The vertical deflection caused by the rotation will be approximately equal to:

$$\Delta = \tan\left(\theta\right)e = \frac{e\left(L_s - d_b\right)}{\left(n-1\right)s - 1/16}$$

The resulting vertical deflections, given a 10-in. eccentricity and 1-in.-diameter bolts in short-slotted holes spaced at 3 in., are as listed in Table 1.

Assuming a ¼ in. of deflection can be tolerated, rotation problems do not exist as long as at least five rows of bolts are provided. The limit of ¼ in. is derived from the suitable performance history of standard shear tabs with short-slotted holes, which will result in a theoretical deflection of about 5⁄16 in. With two or more columns of bolts, the additional geometrical restraint reduces the rotation by a factor of 4, effectively eliminating the problem (Figure 6).

In typical cases, the slab, when cured, can provide the necessary resistance to rotation. Prior to curing, the rotation caused by the dead load must be resisted by the bolted connection alone acting as a slip-critical connection. For this reason, if excessive support rotation during erection is a concern, the bolts should be pretensioned and a minimum of a Class A faying surface should be present when short slots are used. It should be noted that the worst case for any serviceability problems is a connection utilizing a single column of bolts with two rows. As either the number of rows or columns of bolts are increased, the rotation allowed by the movement in the holes drops off quickly, so that there would be essentially no problems for any connections with a double column of bolts, and the rotation of connections with single columns of bolts is only a concern when less than about five rows are employed.

The upper bound of the moment to be resisted by the bolts is defined by the design resistance of the connection, which is designed for bearing and factored down to include only the dead load. The required relationship can be written as:

$$\frac{\phi R_{sc} C'}{e} \geq \frac{\phi R_v C}{1 + L/D}$$

The ratio of the slip resistance with a Class A faying surface to the shear strength of an ASTM A325X bolt is approximately:

$$\frac{\phi R_{sc}}{\phi R_v} = \frac{\phi \mu D_u h_{sc} T_b N_s}{\phi F_{nv} A_b}$$

$$= \frac{1.00\left(0.35\right)\left(1.13\right)\left(0.85\right)\left(0.70\right)\left(90\text{ ksi}\right)\pi\left(\frac{d_b}{2}\right)^2\left(1\right)}{(0.75)(60\text{ ksi})\pi\left(\frac{d_b}{2}\right)^2}$$

$$= 0.471$$

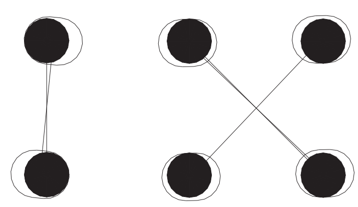


Fig. 6. Rotational restraint provided by (a) single column of bolts and (b) double column of bolts.

It should be noted that in this ratio, the bolt pretension is taken to be 70% of the nominal tensile strength of the bolt instead of the values given in the AISC *Specification*. This is done to provide a closed-form solution to the problem. The ratio for ASTM A490 bolts is similar. It can also be shown that the ratio of C' to C is approximately 10 for eccentricities of 9 to 10 in. From this, a maximum eccentricity, e, can be calculated as:

$$e = \left(1 + L/D\right)0.471(10) = 4.71\left(1 + L/D\right)$$

In the formulation of the original LRFD *Specification*, a live-to-dead load ratio of 3 was assumed. Using the same assumption here results in a maximum eccentricity of 17.8 in. Since this is greater than the typical eccentricity used in practice, the connection will experience no serviceability problems before the concrete is cured, so long as pretensioned bolts are used with a Class A faying surface. Even with a live-to-dead load ratio as low as 1.25, serviceability will not be a problem for eccentricities up to 10 in.

Of course, once the concrete has cured, the connection may be subjected to the full live and dead loading. Serviceability must be considered for this case as well, though a different model can be used. A couple consisting of a tension force at the bolt group and a compression force in the slab will resist the rotation caused by the eccentric gravity loads. Again considering an upper bound defined by the design resistance of the connection in bearing, the required relationship for a bolt spacing of 3 in. is:

$$n\left(\phi R_{sc}\right)\frac{\left(3\text{ in.}\right)n}{2} \geq \phi R_v Ce$$

Again recognizing the ratio of the slip resistance with a Class A faying surface to the shear capacity of an ASTM A325 bolt is:

$$\frac{\phi R_{sc}}{\phi R_v} = 0.471$$

We find that:

$$n^2 \geq 1.4Ce \Rightarrow e \leq \frac{n^2}{1.4C}$$

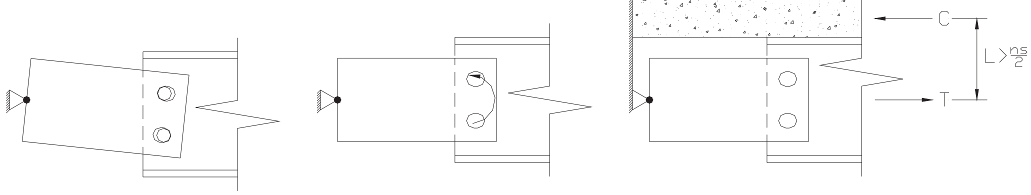
Table 2. Allowable Eccentricity with Cured Concrete Slab

e—no slab	*C*	*e*—with slab (in.)
2	1.18	2.42
3	0.88	3.25
4	0.69	4.14
5	0.56	5.10
6	0.48	5.95
7	0.41	6.97
8	0.36	7.94
9	0.32	8.93
10	0.29	9.85
12	0.24	11.9
14	0.21	13.6
16	0.18	15.9
36	0.08	35.7

Looking at the worst case, a two-row connection, the resulting eccentricities are shown in Table 2.

Table 2 demonstrates that the eccentricity that can be resisted assuming model (c) in Figure 7 is greater than or equal to the eccentricity that the bolt group would be designed to resist as a bearing connection. Therefore, excessive support rotation and vertical deflections are unlikely to occur in practice when the assumed conditions are met.

The preceding discussion emphasized the effects of support rotation as it applies to an unrestrained support. In such cases, the support is assumed to have no stiffness, and no moment is resisted by the support. The problem is strictly one of serviceability. However, the report issued by Sherman and Ghorbanpoor (2002) addressed a "web mechanism" limit state. The web mechanism was created when the moment resistance of the column was mobilized through deformation of the column web. Since the design procedure in the AISC *Steel Manual* sizes the components to resist the full



(a) (b) (c)

Fig. 7. Support rotation at short slotted holes. (a) Support restrained by bolts slipping into bearing. (b) Support restrained by slip resistance to moment. (c) Support restrained by slip resistance to tension and compression in slab.

eccentricity at the bolted connection, the rotational resistance of the support is not required to resist the design loads. The rotation is therefore limited to the simple beam end rotation (usually assumed to be on the order of 0.03 radian) and the rotation allowed by the bolt slip in the holes and bearing deformations in the plates and the bolts. Since this deformation is self-limiting, it is only a serviceability concern.

The limited rotation that results from the AISC procedure can be seen by examining a plot of shear versus rotation from the Sherman and Ghorbanpoor test 1-U (Figure 8). In this test, the girder rotated more than 9 degrees. This obviously exceeds the rotation predicted by simple beam end rotation and connection slip. However, when the predicted capacity from the AISC *Steel Manual* is shown (represented by the heavy horizontal line), it can be seen that the resulting support rotation is limited to less than 3 degrees. The combined rotation that could be expected from the simple beam end rotation, and the connection slip would be about 4 degrees. Limiting the applied load to the design capacity instead of the nominal capacity would result in a further decrease in the support rotation.

COMPARISON TO TEST RESULTS

There is data available from 13 tests performed on unstiffened, extended single-plate connections. Sherman and Ghorbanpoor (2002) presented the results of eight tests. Murray and Metzger (Metzger, 2006) performed five additional tests sponsored by Cives Steel Company. The plates in seven of the tests conformed to the parameters of the design procedure presented in this paper. However, all of the Sherman tests used a weld equal to ¾ of the plate thickness, and all of the Murray tests used a weld equal to ½ the plate thickness. The objective of the Sherman and Ghorbanpoor project

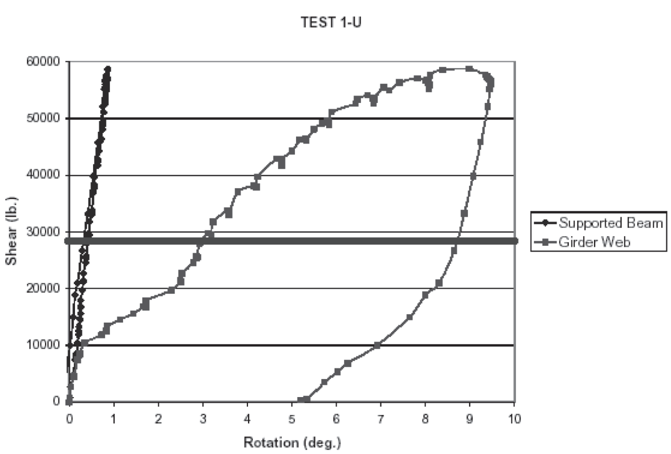


Fig. 8. Plot of shear versus rotation for the Sherman and Ghorbanpoor test 1-U.

was to develop a design procedure for extended single-plate connections. The objective of the Murray and Metzger testing program was to verify the procedures contained in the AISC *Steel Manual* for both conventional and extended single-plate connections. A special emphasis of the Murray and Metzger testing was to validate the reduction in required weld size from ¾ of the plate thickness to ⅝ of the plate thickness. Thus, the available test data represent a wide spectrum of conditions. The single-plate connections in the Sherman testing were all attached to the webs of either columns or girders and are representative of connections with flexible supports, where the bolts would tend to be the more critical than the welds. The Murray single-plate connections were all attached to the flange of a W14×90 and are representative of connections with rigid supports, where the welds would tend to be the more critical than the bolts.

As can be seen in Table 3, the design procedure contained in the AISC *Steel Manual* provides a good margin of safety. Though not typically considered in the design of shear connections, one failure mode emphasized in the work of Sherman and Ghorbanpoor, which is not explicitly considered in the current design procedure, is that of plate twist. Two tests, 2-U and 4-U, are reported as having failed primarily by twisting of the plate.

Work by Bennetts, Thomas, and Grundy (1981) investigated shear connections with the specific goal of developing an understanding of the torsional behavior of shear connections due to the eccentricity of load between the connecting plates and the adjacent beam web. This investigation included testing to determine the torsional stiffness of many types of shear connections, including extended single-plate connections. In that work, the researchers showed that single-plate connections maintain the majority of their torsional rigidity until the plate yields, at which time, as one might expect, the torsional rigidity of the connection decreases and the twist of the connection becomes apparent. To design for this effect, the limit state of twist in the new design procedure is implicitly checked considering the von Mises interaction of forces on the plate, which ensures that the plate does not yield in the presence of shear forces. Effects of twisting can be further mitigated by the lateral bracing of the beam when a slab is present. As can be seen in the test results from Sherman and Ghorbanpoor, the measured capacity was 82.9 kips for test 2-U and 98.7 kips for test 4-U. The limit predicted by the von Mises interaction for both conditions is 85.6 kips, which is accurate to within approximately 3% of the measured capacity of the connection and supports the assumption that twist effects are appropriately addressed by the limit state of yielding of the connection. Twist is listed as a secondary failure mode for tests 6-U and 6-UB, which resisted 138 kips and 136 kips, respectively. The von Mises limit state predicts a capacity of only 116 kips. Further work to refine procedures for considering torsional limit states in all types of shear connections may be appropriate.

Table 3. Summary and Comparison of Unstiffened Extended Single-Plate Connection Test Data												
Test	Columns of Bolts	Rows of Bolts	Depth of Plate (in.)	*a* (in.)	Maximum Allowed Plate Thickness (in.)	Plate Thickness Provided (in.)	Meets Criteria for Extended Shear Tab?	Meets Criteria for Standard Shear Tab?	Tested Capacity	Tested Predicted	Tested Design (Factor of Safety)	Tested Adj. Design* (Factor of Safety)
Sherman and Ghorbanpoor												
1-U	1	3	9	6.85	0.402	⅜	YES	NO	58.7	2.08	5.19	4.15
2-U	1	5	15	6.3	0.421	⅜	YES	NO	82.9	1.00	2.51	2.01
3-U	1	3	9	6.86	0.402	⅜	YES	NO	54.8	1.94	4.85	3.88
3-UM	1	3	9	6.86	0.402	⅜	YES	NO	58.6	2.07	5.18	4.15
4-U	1	5	15	10.04	0.421	½	NO	NO	98.7	1.79	4.48	3.59
6-U	1	6	18	10.04	0.428	½	NO	NO	138	1.77	4.43	3.54
6-UB	1	6	18	10.04	0.428	½	NO	NO	135.8	1.74	4.36	3.48
8-U	1	8	24	10.04	0.426	½	NO	NO	173.6	1.31	3.28	2.62
Murray and Metzger												
1	1	3	8.5	3	0.259	⅜	NO	YES	81	1.11	1.85	1.85
2	1	4	11.5	3	0.271	⅜	NO	YES	110	1.48	3.69	2.95
3	1	5	14.5	3	0.259	⅜	NO	YES	146	1.10	2.75	2.20
4	1	7	20.5	3	0.256	⅜	NO	YES	200	1.08	2.70	2.16
5	2	3	8.5	3	0.695	½	YES	NO	89	1.20	2.99	2.39
6	2	5	14.5	3	0.585	½	YES	NO	200	1.18	2.94	2.35
7	1	7	20.5	9	0.256	⅜	YES	NO	97	1.07	2.68	2.14
5A**	2	2	8.5	3	0.368	½	NO	NO	88	2.16	5.39	4.31

* Where bolt shear controls the design value, the 20% reduction for end-loaded connections has been removed.
** Connection failure did not occur.

Plate buckling was not listed as a failure mode for any of the unstiffened, extended single-plate connection tests. Interestingly when the plate was extended to the bottom flange of the supporting girder, as illustrated in Figure 2, the plate was more likely to buckle.

SUMMARY OF RECOMMENDED DESIGN PROCEDURE

The following procedure has been adopted into the AISC *Steel Manual* as a universally applicable method of designing single plate shear connections. A general extended single-plate shear connection is shown in Figure 9. The procedure, referred to as the extended procedure, is useful when the dimensional and other limitations of the conventional single-plate shear connection design method cannot be satisfied. This procedure can be used to determine the strength of single-plate shear connections with multiple vertical rows of bolts.

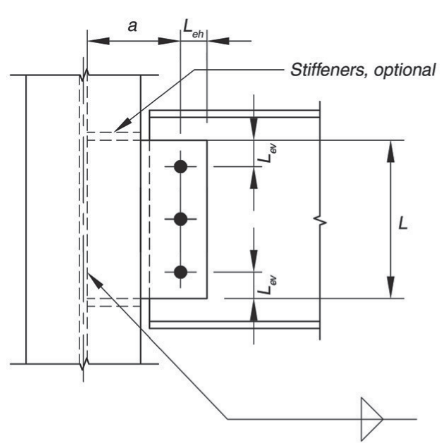


Fig. 9. General extended single-plate connection configuration.

Limitations

1. The use of holes must satisfy AISC *Specification* Section J3.2 requirements. The shear load can be considered to be applied transverse to the slots. The eccentricity does not require that the bolts be designed as slip-critical.

2. The horizontal and vertical edge distances, L_{eh} and L_{ev}, must satisfy AISC *Specification* Table J3.4 requirements.

Design Checks

1. Determine the bolt group required for bolt shear and bolt bearing with eccentricity, *e*, measured as the distance from the support to the center of the bolt group. Alternative considerations of the design eccentricity are acceptable when justified by rational analysis. For example, see Sherman and Ghorbanpoor (2002).

2. Determine the maximum plate thickness permitted such that the plate moment strength does not exceed the moment strength of the bolt group in shear, as follows:

$$t_{max} = \frac{6M_{max}}{F_y d^2}$$

where

M_{max} = $1.25F_v A_b C'$

$1.25F_v$ = shear strength of an individual bolt from AISC *Specification* Table J3.2, ksi, multiplied by a factor of 1.25 to remove the 20 percent reduction for uneven force distribution in end-loaded bolt groups (Kulak, 2002). The joint in question is not end loaded.

A_b = area of an individual bolt, in.^{2}

C' = coefficient from Part 7 of the AISC *Steel Manual* for the moment-only case (instantaneous center of rotation at the centroid of the bolt group), in.

F_y = plate specified yield stress, ksi

d = plate depth, in.

The foregoing check is made at the nominal strength level, since the check is to ensure ductility, not strength.

Exceptions:

a. For a single vertical row of bolts only, the foregoing criterion need not be satisfied if either the beam web or the plate satisfies $t \le d_b/2 + 1/16$ and both satisfy $L_{eh} \ge 2d_b$.

b. For a double vertical row of bolts only, the foregoing criterion need not be satisfied if both the beam web and the plate satisfy $t \le d_b/2 + 1/16$ and $L_{eh} \ge 2d_b$.

3. Check the plate for shear yielding, shear rupture and block shear rupture.

4. Check the plate for flexure with the von Mises shear reduction. That is, check the available flexural yielding strength of the plate, ϕM_n or M_n/Ω, based upon a critical stress F_{cr}:

$$F_{cr} = \sqrt{F_y^2 - 3f_v^2}$$

$$M_n = F_{cr} Z$$

$$\phi = 0.90 \qquad \Omega = 1.67$$

These equations can be rearranged to be directly solvable in terms of the connection available strength as ϕR_n or R_n/Ω such that

$$R_n = \frac{F_y d t_p}{\sqrt{2.25 + 16\left(\frac{a}{d}\right)^2}}$$

$$\phi = 0.90 \qquad \Omega = 1.67$$

5. Check the plate for buckling.

6. Size weld as $w = 5/8 t_p$

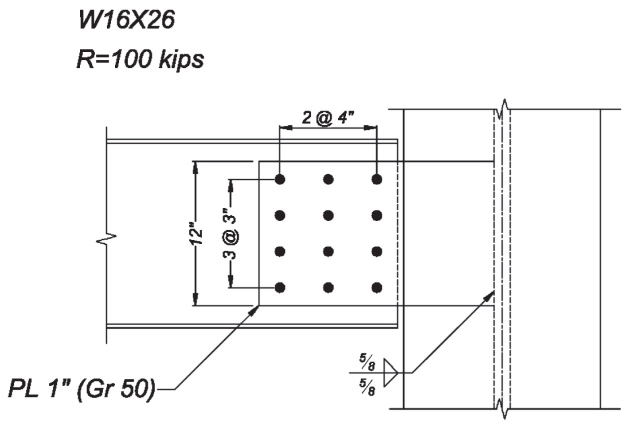
7. Ensure that the supported beam is braced at points of support.

8. Check serviceability criteria.

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 support, for example, 5% of the beam fixed-end moment, provided that this moment is also considered in the design of the supporting member.

To provide for stability during erection and lateral support of the beam, it is recommended that the minimum plate length be one-half the *T*-dimension of the beam to be supported. The maximum length of the plate must be compatible with the *T*-dimension of an uncoped beam and the remaining web depth, exclusive of fillets, of a coped beam. Note that the plate may encroach upon the fillet(s) as shown in Figure 10-3 of the AISC *Steel Manual*.

DESIGN EXAMPLE



Consider a W16×26 with a 100 kip (LRFD) end reaction (see figure). Find an extended single-plate shear connection to support the end reaction.

Assume 1-in. diameter ASTM A490-N bolts (ϕR_v = 35.3 kips/bolt), a = 9.5 in., and all edge distances equal to 1.5 in.

Determine the equivalent bolt strength of the bolt group.

$C = 3.44$

Maximum strength of bolt group due to bearing

$$R = \phi 2.4 F_u t_w d_b$$

$$= (0.75)(2.4)(65 \text{ ksi})(0.25 \text{ in.})(1.0 \text{ in.})$$

$$= 29.3 \text{ kips/bolt}$$

Determine required C-value

$$C_{req'd} = \frac{R_u}{\phi R_{n/bolt}} = \frac{100 \text{ kips}}{29.3 \text{ kips}} = 3.41 \leq 3.44 \ \ \textbf{o.k.}$$

The check, however, will be performed to demonstrate the calculations involved. Find maximum plate thickness such that plate will yield before bolts rupture by accounting for the 20% reduction in bolt strength generally applied to account for the case of end-loaded connections.

$$R_n = \frac{\phi R_v}{(1-0.20)\phi} = \frac{35.3 \text{ kips}}{0.8(0.75)} = 58.8 \text{ kips/bolt}$$

Find the "pure moment" strength of the bolt group (Many conditions are tabulated in the AISC *Steel Manual* Part 7, in Tables 7-7 through 7-14)

$$\begin{aligned} C' = {} & 4(6.02)\left(1-e^{-10(0.34)}\right)^{0.55} \\ & + 2(4.5)\left(1-e^{-\left(\frac{10(4.5)(0.34)}{6.02}\right)}\right)^{0.55} \\ & + 4(4.27)\left(1-e^{-\left(\frac{10(4.27)(0.34)}{6.02}\right)}\right)^{0.55} \\ & + 2(1.5)\left(1-e^{-\left(\frac{10(1.5)(0.34)}{6.02}\right)}\right)^{0.55} \end{aligned}$$

$C' = 50.7$ in.

The moment strength of the bolt group is therefore,

$$M = C' R_n = 50.7 \text{ in.}(58.8 \text{ kips}) = 2{,}980 \text{ kip-in.}$$

Determine the maximum plate thickness. Note that the elastic section modulus is used because the plate will begin to redistribute stress after first yielding.

$$t_{max} = \frac{6M}{F_y d^2} = \frac{6(2{,}980 \text{ kip-in.})}{50 \text{ ksi}(12 \text{ in.})^2} = 2.48 \text{ in.}$$

Try a 1-in.-thick plate with F_y = 50 ksi.

Check shear and bending interaction on gross plate section (von Mises criteria)

$$\phi R_n = \frac{\phi F_y d t_p}{\sqrt{2.25 + 16\left(\frac{a}{d}\right)^2}}$$

$$= \frac{0.90(50 \text{ ksi})(12 \text{ in.})(1 \text{ in.})}{\sqrt{2.25 + 16\left(\frac{9.5 \text{ in.}}{12 \text{ in.}}\right)^2}} = 154 \text{ kips}$$

Check net shear rupture on the plate

$R_n = 0.6 F_u A_{nv} = 0.75(0.6)(65 \text{ ksi})(7.5 \text{ in.}^2)$

$= 219$ kips > 100 kips **o.k.**

Check block shear rupture on the plate

$\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6 F_y A_{gv}, \phi 0.6 F_u A_{nv})$

Tension rupture component

$\phi F_u A_{nt} U_{bs} = 0.75(65 \text{ ksi})(6.69 \text{ in.})(1 \text{ in.})(0.5) = 163 \text{ kips}$

Shear yielding component

$\phi 0.6 F_y A_{gv} = 0.75(0.6)(50 \text{ ksi})(10.5 \text{ in.})(1 \text{ in.}) = 236 \text{ kips}$

Shear rupture component

$\phi 0.6 F_u A_{nv} = 0.75(0.6)(65 \text{ ksi})(6.56 \text{ in.})(1 \text{ in.}) = 192 \text{ kips}$

$R_n = (163 \text{ kips} + 192 \text{ kips}) = 355 \text{ kips} \quad 100 \text{ kips}$ **o.k.**

Check plate buckling

$$\lambda = \frac{d\sqrt{F_y}}{t_p\sqrt{47,500 + 112,000\left(\frac{d}{2e'}\right)^2}}$$

$$= \frac{12\sqrt{50}}{1\sqrt{47,500 + 112,000\left(\frac{12}{2(9.5)}\right)^2}} = 0.279 < 0.7$$

Buckling does not control.

Size the weld to ensure that the plate will yield before the weld ruptures:

$$w = 0.625 t_p = 0.625(1) = \tfrac{5}{8} \text{ in.}$$

SYMBOLS

a = the distance from the face of the support to the first vertical line of bolts

c = number of column (vertical rows) of bolts

d = plate depth

d_b = bolt diameter

e = the distance from the face of the support to the center of the bolt group

f_a = the applied bending stress (net or gross)

f_v = the applied shear stress (net or gross)

n = the number of rows

s = the spacing between rows of bolts

t_p = plate thickness, in.

w = weld leg size, in.

A_b = area of bolt, in.^{2}

C = coefficient for eccentrically loaded bolt groups, from AISC *Steel Manual* Part 7.

C' = coefficient used in determining the equivalent pure-moment bolt group strength

D = live load

E = modulus of elasticity, ksi

E_T = tangent modulus of elasticity, ksi

L = live load

F_{EXX} = electrode classification number, ksi

F_{nv} = the shear strength of an individual bolt from AISC *Specification* Table J3.2, ksi

F_y = the minimum specified yield stress, ksi

F_u = the minimum specified tensile stress, ksi

L_i = the distance of the ith bolt from the center of gravity of the bolt group

L_{max} = the distance of the bolt furthest from the center of gravity of the bolt group to the center

L_s = the length of the slot

L_w = length of weld, in.

R = the simple beam end reaction

ϕR_{sc} = the slip resistance provided by a single bolt as a serviceability limit state

ϕR_{brg} = the shear capacity of a single bolt

Δ = vertical deflection caused by the rotation of bolts in short slotted holes

Δ_{max} = the maximum deformation on the bolt furthest from the center of gravity, 0.34 in.

λ = slenderness parameter, dimensionless (Young's modulus is incorporated in formula)

θ = rotation allowed by movement in short slotted holes

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