Analysis and Design of Stabilizer Plates in Single-Plate Shear Connections

PATRICK J. FORTNEY and WILLIAM A. THORNTON

ABSTRACT

Single-plate shear connections experience some magnitude of torsional moment, either due to the lateral torsional buckling phenomena or due to the effects of lap eccentricity. When the required torsional strength of the connection exceeds the available torsional strength of the connection, the designer has two options: alter the geometry of the connection to increase the torsional resistance of the connecting plate or provide stabilizer plates. Thornton and Fortney (2011) provide analysis techniques for accounting for the effects of lap eccentricity and lateral torsional buckling strength. Part 10 of the *Manual* (*Steel Construction Manual,* 14th ed., 2011) presents a summary of the equations used for such an evaluation. However, no discussion was provided by Thornton and Fortney with regard to the size and detailing of a stabilizer plate when such a plate is required. This paper presents recommendations for the analysis with regard to appropriate stabilizer plate cross-sectional dimensions and the attachment of the stabilizer plate to the connecting material and support. Three different types of stabilizer plates are presented along with recommendations for the design and detailing of the stabilizer plates; the impact that each type has on the design of the single-plate shear connection and the supporting column is presented as well.

Keywords: nodal bracing, single-plate shear connections, stabilizer plates, stiffener plates.

INTRODUCTION

Stabilizer plates can be used to counteract the effects of lap eccentricity and to prevent lateral torsional buckling in single-plate shear connections (Thornton and Fortney, 2011). Part 10 of the *Steel Construction Manual* (AISC, 2011) presents equations for evaluating the need for stabilizer plates based on these two considerations. For either limit state, the role of the stabilizer plate in the connection is simply to provide lateral support to the connecting material. In the ideal case, the stabilizer plate is not part of the load path for transferring the beam end shear to the support. In order to minimize the participation of the stabilizer plate in transferring the force to the support, the stiffness that the stabilizer plate provides to the overall connection must be minimized. However, it may not be desirable to provide such a "flexible" stabilizer plate. When this is the case, the role that the stabilizer plate plays in the load path of the beam-to-column connection needs to be considered. When developing analysis and design procedures for stabilizer plates, the following questions should be considered:

Patrick J. Fortney, Ph.D., P.E., S.E., P.Eng., President and Chief Engineer, Cives Engineering Corporation, Roswell, GA (corresponding). Email: pfortney@cives.com

William A. Thornton, Ph.D., P.E., NAE, Corporate Consultant, Cives Engineering Corporation, Roswell, GA. Email: bthornton@cives.com

Paper No. 2013-15

1. What is the role of the stabilizer plate?
2. What types of stabilizer plates need to be considered?
3. What types of analysis and design procedures should be used?
    a. To what degree does the stabilizer plate act as part of the load path in the beam-to-column connection?
    b. What is the impact on the beam-to-column connection design?
    c. What impact does the stabilizer plate have on the supporting member?

These three primary questions are addressed in this paper.

ROLE OF THE STABILIZER PLATE

The purpose of the stabilizer plate is to provide a lateral brace to the single-plate shear connection when the required shear force in the connection exceeds the shear that initiates lateral torsional buckling or to offset the effects of lap eccentricity. A stabilizer plate should only be used when all attempts to work with connection geometry and hardware cannot provide adequate torsional strength. Stabilizer plates not only add unnecessary costs, they potentially change the behavior of the connection and induce rotational demands not accounted for during the frame analyses as well.

The lateral brace can be thought of as a nodal brace, in the sense of AISC *Specification* Appendix 6 (AISC, 2010), between the connecting plate and the column flange (see Figure 1), thus providing lateral stability to the connecting plate. To provide adequate bracing, both the axial strength and axial stiffness of the stabilizer plate must be considered. The required brace force in the stabilizer plate, P_s, would need to be transferred from the connection plate to the stabilizer plate. The bracing load acts at approximately mid-length of the stabilizer plate. If the ends of the stabilizer plate are connected to the column flanges, one-half of the stabilizer plate is in compression, while the other half is in tension (see Figure 1). The axial force can be resisted either through bearing at the plate-column flange interface on the compression side or through a weld at the plate-column flange interface on the tension side.

TYPES OF STABILIZER PLATES

In order to assess the number of different types of stabilizer plates that might be used, not only the connected beam and column must be considered, but also the members framing into the joint from the perpendicular direction. The role the stabilizer plate plays in transporting the beam end shear to the support also needs to be considered.

For the purposes of the discussion presented in this paper, the authors assume that there are two different types of connections provided for the perpendicular members: simple shear connections and moment connections. When simple shear connections are used to connect the perpendicular members, it is assumed that the joint is clean (i.e., no continuity plates or web doublers). When moment connections are used to connect the perpendicular members, it is assumed that continuity plates are required and the shear connection plate is fitted to the continuity plates. Note that if the perpendicular members are moment-connected and continuity plates are not required, the joint is clean and, for the purpose of a discussion regarding stabilizer plates, can be considered to be no different than when the perpendicular members were shear-connected. Additionally, it is not uncommon to see the connection plate in a single-plate shear connection fitted to the flanges of a spandrel beam in a beam-to-spandrel beam connection or a beam-to-beam connection with a supported beam on only one side of the support beam. This condition is similar to a connection plate fitted to the continuity plates in a joint with moment connections framing in from the perpendicular direction.

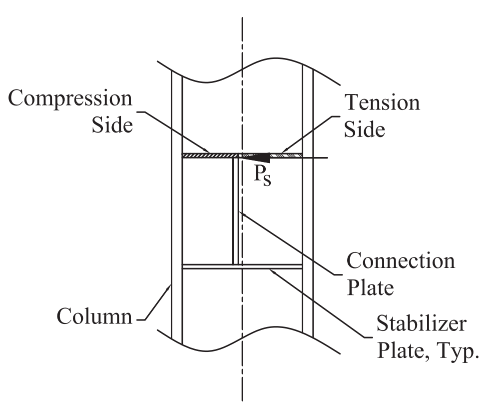

Fig. 1. Brace force, P_s, *provided by stabilizer plate.*

Figure 2 shows sketches of three different types of stabilizer plates: types I, II and III. Type I and type II stabilizer plates can be used at joints where the perpendicular members are shear connected (a clean joint). Where the perpendicular member is moment-connected and continuity plates are required, the type III stabilizer plate would be required. Type Ia and type Ib stabilizer plates are assumed to play no role in transporting the required beam end shear. Types II (if not permitted to float) and III will play a role in the transport of the beam end shear.

The type Ia stabilizer plate is not practical. This type of stabilizer plate would certainly raise concerns in regard to surviving transportation and erection. It is presented only as a tool to facilitate a discussion regarding the analysis and design of the plate. The type Ib stabilizer plate will provide a more practical approach to providing lateral stability of the single-plate shear connection while not participating in the load path for the connection.

Analysis and Design of Stabilizer Plates

A stabilizer plate may increase the amount of rotational restraint provided by the connection. One simple way of eliminating any concern in this regard is to make no attachment of the stabilizer plate to the column flanges. Note that this approach can be considered only for type I and II plates; it would not be an option for a type III stabilizer plate, which also acts as a continuity plate for the moment-connected beam framing into the joint from the perpendicular direction. If the stabilizer plate is not attached to the column flanges, it would be attached to the connection plate only, and given a length, l_s (see Figure 3)—such that the stabilizer plate fits within the column flanges—but is allowed to translate relative to the column flanges as the beam undergoes simple beam end rotation under the presence of load. In essence, the stabilizer plate is permitted to "float" between the column flanges.

Where a welded connection of the stabilizer plate to the column flange is neither required nor desired, the required axial force, P_s, in the stabilizer plate would be transferred to the column flange through bearing. Under this condition, because no weld is provided on the "tension" side of the stabilizer plate (refer to Figures 1 and 3), buckling of the stabilizer plate would need to be considered when determining the cross-sectional dimensions of the stabilizer plate. Where a welded connection is required or desired at the stabilizer plate-column flange interface, buckling of the stabilizer plate would not be a concern because the axial force in the stabilizer plate would be resisted on the tension side of the plate.

Type Ia Stabilizer Plates

Figure 4 shows a sketch representative of a type Ia stabilizer plate. This type of stabilizer plate is relatively flexible and can be reasonably assumed to provide no additional stiffness or rotational resistance to the single-plate shear connection. Furthermore, it is reasonable to assume that the stabilizer plate provides no appreciable redundancy to the connection. That is, the stabilizer plate is not part of the load path for transporting the required beam end shear to the support. Therefore, the design of the single-plate shear connection would follow established design procedures (Part 10 of the AISC *Manual*). For this type of stabilizer plate, the required cross-sectional dimensions can be established as follows.

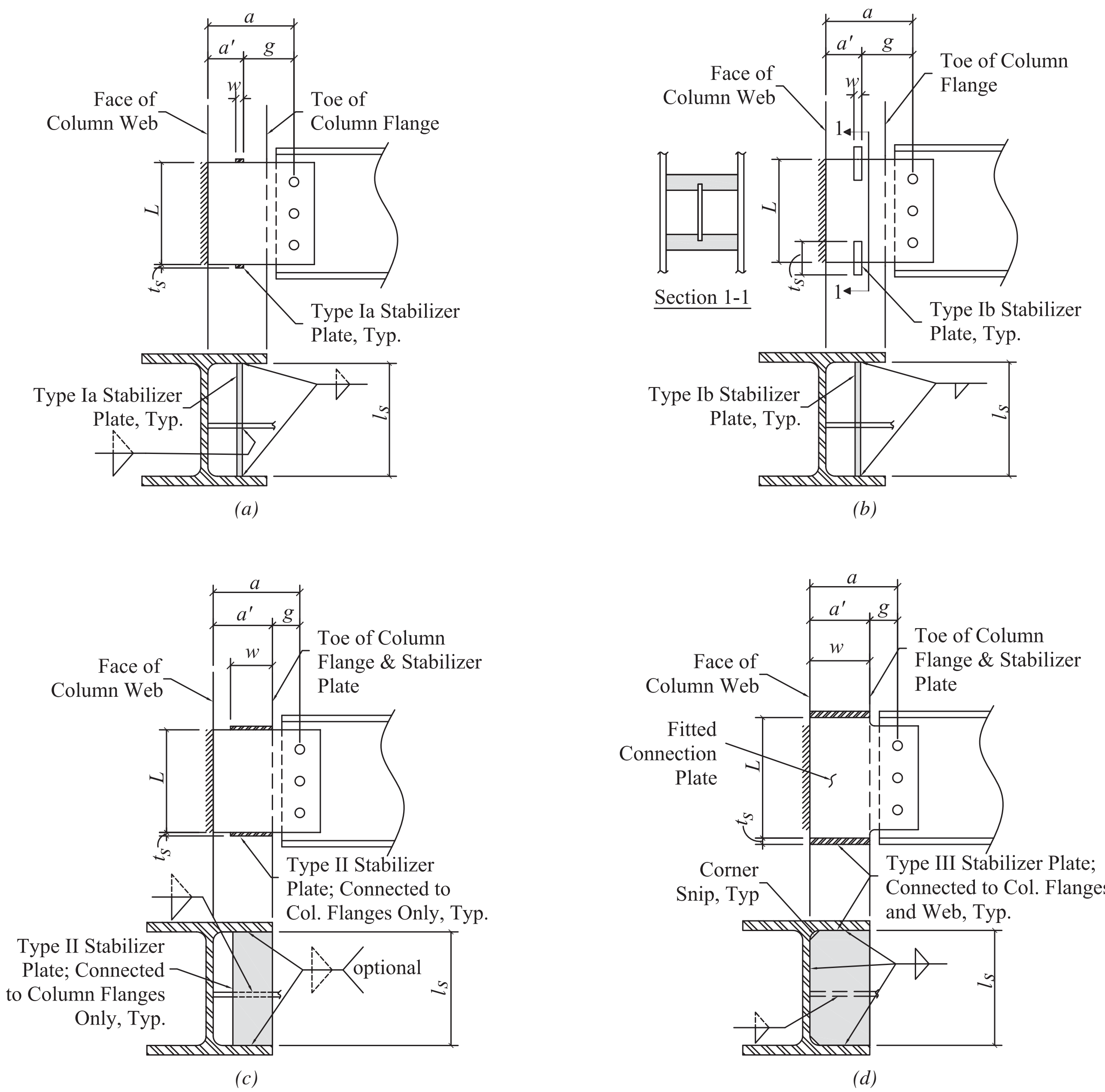

Fig. 2. Options for stabilizer plates: (a) type Ia; (b) type Ib; (c) type II; (d) type III.

Assuming nodal bracing, the required axial strength of the stabilizer plate can be taken as given in *Specification* Equation A-6-7 (AISC, 2010). A form of that equation is shown here in Equation 1.

$$P_S = \frac{0.02 M_b C_d}{L} \tag{1}$$

where
P_s = the required axial strength of the stabilizer plate
$M_b = M_{ba} = R_a a$ (ASD), $M_{bu} = R_u a$ (LRFD)
C_d = 1.0
L = connection length (see Figure 2)
R = R_a (ASD), R_u (LRFD)

Taking the axial tensile strength of the stabilizer plate as

$$\phi P_n = \phi F_y A_s \qquad \text{(LRFD)} \tag{2a}$$

$$\frac{P_n}{\Omega} = \frac{F_y A_s}{\Omega} \qquad \text{(ASD)} \tag{2b}$$

where A_s is the cross-sectional area of the stabilizer plate, the cross-section of the stabilizer plate can be determined by setting Equation 1 equal to Equation 2 and rearranging to solve for A_s, as shown in Equation 3.

$$\begin{aligned} \phi F_y A_s &= \frac{0.02 M_{bu} C_d}{L} \\ \phi F_y A_s L &= 0.02 M_{bu} C_d \\ \phi F_y A_s L &= 0.02 R_u a (1.0) \end{aligned}$$

$$A_s = \frac{0.02 R_u a}{\phi F_y L} \qquad \text{(LRFD)} \tag{3a}$$

$$\begin{aligned} \frac{F_y A_s}{\Omega} &= \frac{0.02 M_{ba} C_d}{L} \\ \frac{F_y A_s L}{\Omega} &= 0.02 M_{ba} C_d \\ \frac{F_y A_s L}{\Omega} &= 0.02 R_a a (1.0) \end{aligned}$$

$$A_s = \frac{0.02 \Omega R_a a}{F_y L} \qquad \text{(ASD)} \tag{3b}$$

In Equations 3a and 3b, $\phi = 0.75$ and $\Omega = 2.00$.

Similarly, the required axial stiffness of the stabilizer plate can be taken as given in *Specification* Equation A-6-8, and shown in Equation 4.

$$\beta_S = \frac{1}{\phi}\left(\frac{10 M_{bu} C_d}{aL}\right) \qquad \text{(LRFD)} \tag{4a}$$

$$\beta_s = \Omega\left(\frac{10 M_{ba} C_d}{aL}\right) \qquad \text{(ASD)} \tag{4b}$$

where
β_s = required axial stiffness of the stabilizer plate
$M_{bu} = R_u a$, $M_{ba} = R_a a$
C_d = 1.0
L = connection depth
a = the unbraced length of the connection plate

The axial stiffness of the stabilizer plate is taken as

$$\beta_s = \frac{A_s E}{\frac{l_s}{2}} = \frac{2 A_s E}{l_s} \tag{5}$$

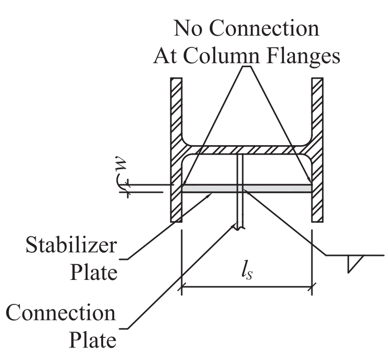

Fig. 3. Type I floating stabilizer plate (no connection to column flanges).

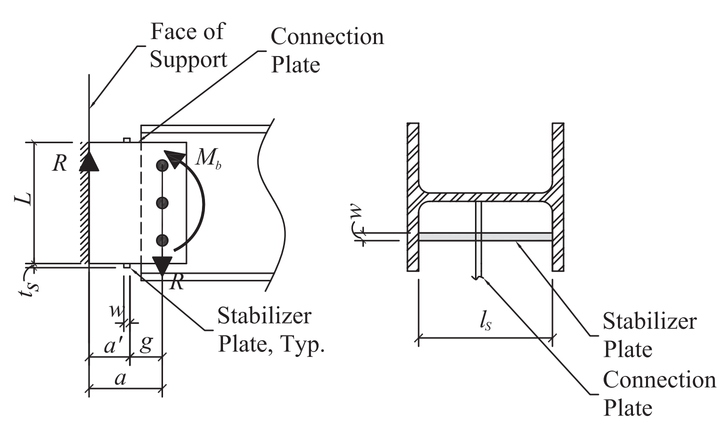

Fig. 4. Type Ia stabilizer plate (flexible).

where l_s is the length of the stabilizer plate. Note that in Equation 5, the stiffness of the stabilizer plate is based on the length of the plate between the connection plate and the column flange ($l_s/2$). The cross-section of the stabilizer can be determined by setting Equation 4 equal to Equation 5 and rearranging to solve for A_s, as shown in Equation 6.

$$\frac{1}{\phi}\left(\frac{10M_{bu}C_d}{aL}\right)=\frac{2A_sE}{l_s}$$

$$A_s=\frac{10M_{bu}C_d l_s}{2\phi aLE}=\frac{10R_u a(1.0)l_s}{2\phi aLE}$$

$$A_s=\frac{5R_u l_s}{\phi LE} \qquad \text{(LRFD)} \qquad (6a)$$

$$\Omega\left(\frac{10M_{ba}C_d}{aL}\right)=\frac{2A_sE}{l_s}$$

$$A_s=\frac{10\Omega M_{ba}C_d l_s}{2aLE}=\frac{10\Omega R_a a(1.0)l_s}{2aLE}$$

$$A_s=\frac{5\Omega R_a l_s}{LE} \qquad \text{(ASD)} \qquad (6b)$$

In Equations 6a and 6b, $\phi = 0.75$, $\Omega = 2.00$.

When using a flexible stabilizer plate, it can be assumed that little or no additional connection stiffness is provided and that no additional rotational restraint is provided by the connection. For these given assumptions, the single-plate shear connection would be designed using the same procedures used if the stabilizer plate were not present. The stabilizer plate cross-section would be determined using Equations 3 and 6 to check against the required axial strength and stiffness.

The attachment of the stabilizer plate to the connection plate can be made with a fillet weld sized to transfer the required axial strength as determined by Equation 1. Note that the weld lengths will need to be at least as long as four times the fillet weld size (Section J2b of AISC 360-10). Therefore, the stabilizer plate width, w, will need to be at least ¾ in. (assuming a 3⁄16-in. fillet weld is used). Because it is assumed that the type Ia plate does not participate in the load path for transporting the beam end reaction, only the axial force, P_s, need be considered in the determination of the size of the stabilizer plate or its attachment.

Where the stabilizer plate axial force is transferred to the column flange through bearing and no welded connection is made at the column flanges, the cross-sectional dimensions of the stabilizer plate will be controlled by bearing or buckling. Equations 7 and 8 give design equations for checking bearing strength at the column flange–stabilizer plate interface and buckling of the stabilizer plate, respectively, where P_s is determined by Equation 1.

$$P_{su} \le \phi 1.8 F_y A_s \qquad \text{(LRFD)} \qquad (7a)$$

$$P_{sa} \le \frac{1.8F_yA_s}{\Omega} \qquad \text{(ASD)} \qquad (7b)$$

In Equations 7a and 7b, $\phi = 0.75$ and $\Omega = 2.00$.

$$P_{su} \le \phi F_{cr} A_s \qquad \text{(LRFD)} \qquad (8a)$$

$$P_{sa} \le \frac{F_{cr}A_s}{\Omega} \qquad \text{(ASD)} \qquad (8b)$$

In Equations 8a and 8b, $\phi = 0.90$ and $\Omega = 1.67$.

Example Problem 1—Flexible Stabilizer Plate, Type Ia

Problem Statement

Figure 5 shows a single-plate shear connection used as a beam end connection for a W12×35 framing to the web of a W12×30 column. The required shear force for the design of the connection is $R_u = 22$ kips.

1. Determine if a stabilizer plate is required based on
   a. Lateral torsional buckling of the plate.
   b. The effect of lap eccentricity.
2. Whether a stabilizer plate is required or not, determine the required cross-sectional dimensions for a type Ia A36 stabilizer plate. The stabilizer plate will be welded to the column flanges. Specify the welds at the column flanges and connection plate. Assume a one-sided 3⁄16-in. fillet weld is used for attaching the stabilizer plate.

Solution

Part 1a

The lateral torsional buckling strength of the plate is (Thornton and Fortney, 2011)

$$\begin{aligned}
\phi R_n &= \phi \frac{1{,}500\pi L t^3}{a^2} \\
&= (0.9)\left(\frac{(1{,}500 \text{ ksi})\pi(9 \text{ in.})(0.25 \text{ in.})^3}{(5.0 \text{ in.})^2}\right) \\
&= 23.8 \text{ kips} > 22.0 \text{ kips} \quad \rightarrow \quad \text{stabilizer plate not required}
\end{aligned}$$

Part 1b

The required torsional moment strength due to lap eccentricity is (Thornton and Fortney, 2011)

$$\begin{aligned}
M_{tu} &= R\left(\frac{t_w + t_p}{2}\right) \\
&= (22 \text{ kips})\left(\frac{0.30 \text{ in.} + 0.25 \text{ in.}}{2}\right) \\
&= 6.05 \text{ kip-in.}
\end{aligned}$$

The available torsional moment strength is (Thornton and Fortney, 2011)

$$\begin{aligned}
\phi M_t &= \left[\phi_v \left(0.6F_{yp}\right) - \frac{R_u}{Lt_p}\right]\left(\frac{Lt_p^2}{2}\right) + \frac{2R_u^2\left(t_w + t_p\right)b_f}{\phi_b F_{yb} L_s t_w^2} \\
&= \left[(1.0)(0.6)(36 \text{ ksi}) - \frac{22 \text{ kips}}{(9 \text{ in.})(0.25 \text{ in.})}\right]\left(\frac{(9 \text{ in.})(0.25 \text{ in.})^2}{2}\right) + \frac{(2)(22 \text{ kips})^2 (0.30 \text{ in.} + 0.25 \text{ in.})(6.56 \text{ in.})}{(0.90)(50 \text{ ksi})(28 \text{ ft})(12 \text{ in./ft})(0.30 \text{ in.})^2} \\
&= 3.33 \text{ kip-in.} + 2.57 \text{ kip-in.} \\
&= 5.90 \text{ kip-in.} < 6.05 \text{ kip-in.} \quad \rightarrow \quad \text{stabilizer plate is required}
\end{aligned}$$

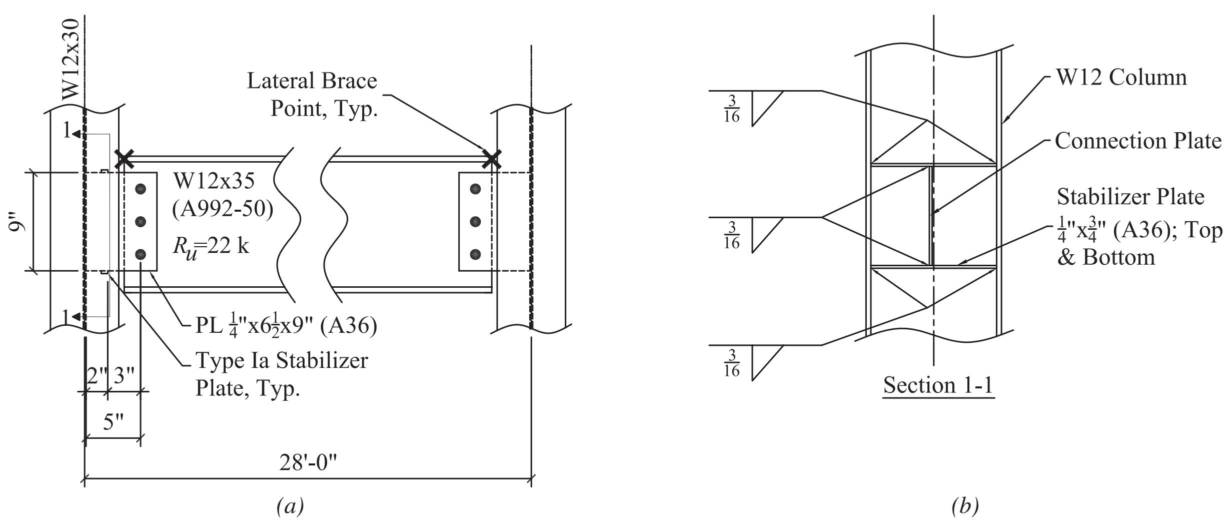

Fig. 5. Single-plate shear connection referenced in Example Problem 1: (a) elevation; (b) final stabilizer plate details.

Part 2

Because a 3⁄16-in. fillet weld is used to attached the stabilizer plate, the width of the plate must be at least (4)(0.1875 in.) = 0.750 in.

Stabilizer plate size based on required axial strength using Equation 3a:

$$\begin{aligned} A_s &= \frac{0.02R_u a}{\phi F_y L} \\ &= \frac{(0.02)(22\text{ kips})(5\text{ in.})}{(0.75)(36\text{ ksi})(9\text{ in.})} \\ &= 0.0091\text{ in.}^2 \end{aligned}$$

Assuming the width of the plate is 0.750 in., the required plate thickness is:

$$\begin{aligned} A_s &= wt_s \\ 0.0091\text{ in.}^2 &= (0.750\text{ in.})t_s \\ t_s &= \frac{0.0091\text{ in.}^2}{0.750\text{ in.}} \\ &= 0.012\text{ in.} \end{aligned}$$

Good detailing practice would suggest using at least a ¼-in. plate, which is greater than $t_s = 0.012$ in. Therefore,

Use a ¼-in. × ¾-in. plate.

Stabilizer plate size based on required axial stiffness using Equation 6a:

$$\begin{aligned} l_s &= \left(d - 2t_f\right)_{column} \\ &= 12.3\text{ in.} - (2)(0.44\text{ in.}) \\ &= 11.4\text{ in.} \end{aligned}$$

$$\begin{aligned} A_s &= \frac{5R_u l_s}{\phi LE} \\ &= \frac{(5)(22\text{ kips})(11.4\text{ in.})}{(0.75)(9\text{ in.})(29{,}000\text{ ksi})} \\ &= 0.006\text{ in.}^2 < 0.0091\text{ in.}^2 \quad \rightarrow \quad \text{strength controls over stiffness} \end{aligned}$$

Use a ¼-in. × ¾-in. stabilizer plate.

Attachment of Stabilizer Plate to Single-Plate Shear Connection and Column Flange

The required axial force in the stabilizer plate is determined using Equation 1.

$$\begin{aligned} P_{Su} &= \frac{0.02M_{bu}C_d}{L} \\ &= \frac{(0.02)(22\text{ kips})(5\text{ in.})(1.0)}{9.0\text{ in.}} \\ &= 0.244\text{ kips} \end{aligned}$$

The width of the stabilizer plate is 0.750 in. Check that a 3⁄16-in. single-sided fillet weld at the connection plate and column flange is sufficient.

$$\phi R_w = 1.392Dnl\left(1+0.5\sin^{1.5}\theta\right)$$
$$= (1.392)(3 \text{ sixteenths})(1)(0.75 \text{ in.})\left(1+0.5\sin^{1.5} 90°\right)$$
$$= 4.70 \text{ kips} > 0.244 \text{ kips} \quad \textbf{OK}$$

Use a $\frac{3}{16}$-in. fillet weld at the stabilizer plate-to-connection plate and stabilizer plate-to-column flange interfaces.

Note that if one were to choose not to attach the stabilizer plate to the column flanges, the stabilizer plate would need to be checked for (1) bearing against the column flanges and (2) buckling on the compression side of the stabilizer plate.

See Figure 5b for a sketch of the details of the stabilizer plate and its attachments.

Type Ib Stabilizer Plates

Type Ib Plate Geometry

The type Ib stabilizer plate is similar to the type Ia plate in that it is assumed that the stabilizer plate does not play a role in transferring the beam end reaction to the support. Therefore, as with the type Ia configuration, the single-plate shear connection design procedure of *Manual* Part 10 applies. With the type Ia stabilizer plate, a welded connection is required at the stabilizer-to-connection plate interface. An attachment at this location is not required with the type Ib plate. As can be seen in Figure 6, the type Ib plate is notched around the connection plate, and the required lateral bracing of the connection plate is achieved through bearing of the connection plate on the notched portion of the stabilizer plate. A weld is used to transfer the axial load in the stabilizer plate to the column flange. Both ends of the stabilizer plate are welded to the column flange so, regardless of the direction of the P_s load, the tension side of the stabilizer plate will provide the entire brace force. Thus, stabilizer buckling does not need to be checked.

The height of the notch, h_n, should be sufficient to transfer the P_s force in bearing and also provide vertical clearance such that the connection plate does not contact the stabilizer during simple beam end rotation. The height of the contact area between the connection plate and the stabilizer, h_p, will be driven by what is required for bearing. The authors recommend a vertical clearance of 0.5 times the thickness of the connection plate, t_p.

The width of the notch, w_n, should be wide enough to provide horizontal clearance of the connection plate but not so wide as to allow undesirable lateral movement of the connection plate. The authors recommend a notch width of $t_p + \frac{1}{16}$ in.

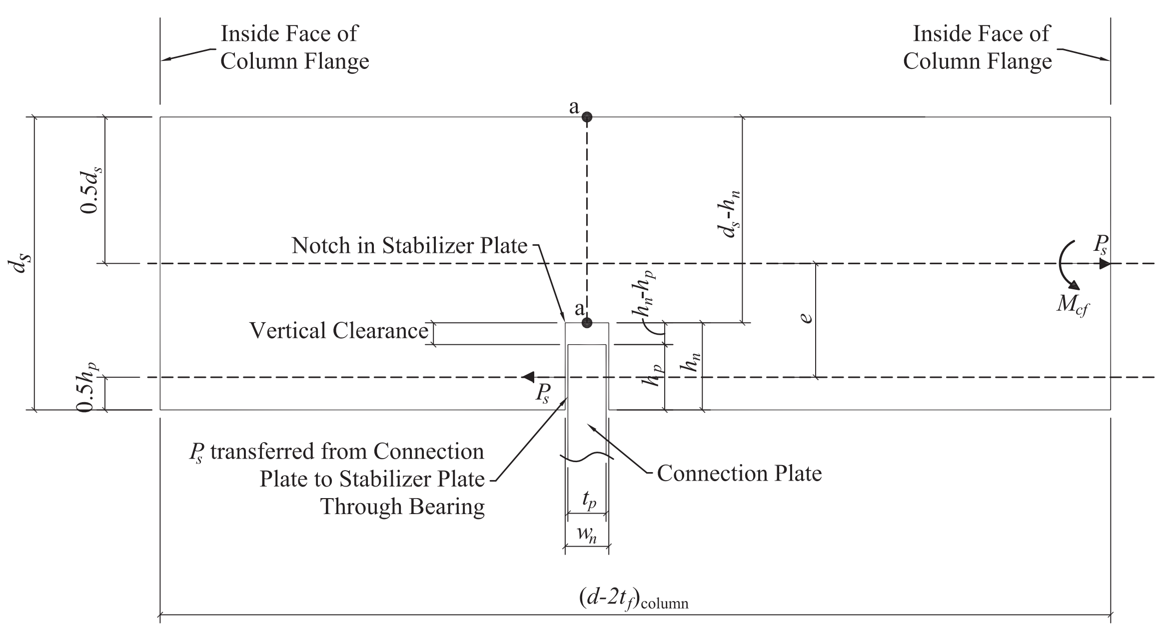

Fig. 6. Type Ib stabilizer plate.

The total height of the stabilizer, d_s, should be sufficient to transfer the axial force, P_s, and the moment, M_{notch}, present at section a-a (see Figure 6 and 7). Section a-a is the critical section for bending and axial load (discussed later). The authors recommend a stabilizer height, d_s, equal to or greater than 2 times the height of the notch, h_n.

For design, trial geometry of the notch can be assumed to have height h_n = 2 times the thickness of the connection plate ($h_n = 2t_p$) and the height of the contact area of the connection plate with the stabilizer plate to be h_p = 0.75 times the height of the notch ($h_p = 1.5t_p$).

Type Ib Plate Analysis and Design

At the connection plate–stabilizer plate interface, the brace force is transferred at the center of the contact area of the two plates. At the stabilizer–column flange interface, the load is transferred at mid-depth of the stabilizer plate. As can be seen in Figure 6, the two forces act along two different lines of action. Thus, an eccentricity, e, exists. This eccentricity is resolved by splitting the stabilizer vertically at section a-a located at mid-width ($w_n/2$) of the notch. Figures 7 and 8 show the free-body diagrams of the compression side and tension side of section a-a, respectively.

As can be seen in Figure 7, the stabilizer plate is required to transfer the axial load, P_s, and a moment, M_{notch}, at the notch (section a-a). P_s acts at one-half the height of the vertical dimension of the contact surface of the connection plate with the stabilizer (h_p). The P_s force acts on section a-a at the centroid of section a-a. The distance between the two lines of action is $0.5(d_s + h_n - h_p)$. The moment at the notch is

$$\sum M_i = 0 = P_s\left[0.5(d_s + h_n - h_p)\right] - M_{notch}$$

$$M_{notch} = P_s\left[0.5(d_s + h_n - h_p)\right]$$

Referring to Figure 8, it can be seen that the stabilizer-to-column flange interface must transfer the required axial force, P_s, and a moment, M_{cf}. The distance between these two lines of action is $0.5h_n$. The moment at the face of the column is

$$\sum M_{cf} = 0 = -P_s(0.5h_n) + M_{notch} - M_{cf}$$

$$M_{cf} = M_{notch} - 0.5h_n P_s$$

Comparing the moments, M_{notch} and M_{cf}, it can be deduced that the moment M_{notch} will always be larger than M_{cf} ($M_{cf} = M_{notch} - 0.5h_nP_s$). The height of the stabilizer at section a-a is smaller than at the face of the column. Thus, section a-a is the critical section for bending. The axial force, P_s, acts at both section a-a and at the face of the column. Because section a-a has a smaller cross-sectional area, section a-a is the critical section for axial load. Thus, the combination of bending and tension yield will be checked at section a-a. This check will be done by checking tension yield for a force, N_{Tot}, equal to the sum of P_s plus the equivalent normal force of the moment, M_{notch}. The weld at the face of the column will be sized for the combination of axial load, P_s, and moment, M_{cf}.

Checks at Section a-a

The total equivalent axial force acting on section a-a is

$$N_{Tot} = P_s + N_{eq}$$

where

$$N_{eq} = \frac{4M_{notch}}{d_s - h_n}$$

$$N_{Tot} = P_s + \frac{4M_{notch}}{d_s - h_n} \tag{9}$$

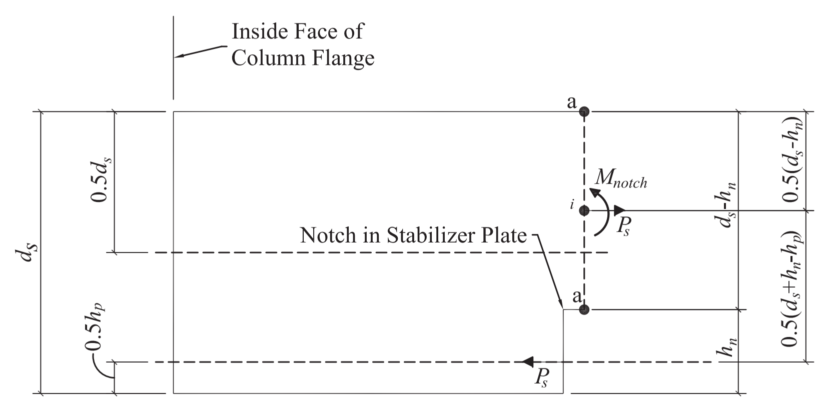

Fig. 7. Type Ib stabilizer plate—free-body diagram: compression side.

The required plate thickness, t_s, for tension yield is

$$\phi T_n = \phi F_y \left(d_s - h_n\right) t_s = N_{Tot}$$

$$t_s \geq \frac{N_{Tot,u}}{\phi F_y \left(d_s - h_n\right)} \qquad \text{(LRFD)} \qquad (10a)$$

$$\frac{T_n}{\Omega} = \frac{F_y \left(d_s - h_n\right)}{\Omega} = N_{Tot,a}$$

$$t_s \geq \frac{\Omega N_{Tot,a}}{F_y \left(d_s - h_n\right)} \qquad \text{(ASD)} \qquad (10b)$$

In Equations 10a and 10b, $\phi = 0.90$, $\Omega = 1.67$.

Bearing Check at the Notch

The plate provides lateral bracing through bearing of the stabilizer-connection plate bearing. Thus, the required contact area of the plate is given by Equation J7-1 of the *Specification*.

$$R_n = 1.8 F_y A_{pb}$$

Taking A_{pb} as the contact height, h_p, times the thickness of the stabilizer plate, t_s, the nominal bearing strength is

$$R_n = 1.8 F_y h_p t_s$$

Thus, the required plate thickness based on bearing is

$$\phi R_n = 1.8 F_y h_p t_s \geq P_{su}$$

$$t_s \geq \frac{P_{su}}{\phi 1.8 F_y h_p} \qquad \text{(LRFD)} \qquad (11a)$$

$$\frac{R_n}{\Omega} = \frac{1.8 F_y h_p t_s}{\Omega} \geq P_{sa}$$

$$t_s \geq \frac{\Omega P_{sa}}{1.8 F_y h_p} \qquad \text{(ASD)} \qquad (11b)$$

In Equations 11a and 11b, $\phi = 0.75$, $\Omega = 2.00$, and F_y is the smaller of the yield strengths of the stabilizer and connection plates.

Weld at the Column Flange

The weld is designed for the combination of axial load, P_s, and moment, M_{cf}. As discussed in other parts of the paper, the weld is sized for a total axial load equal to the sum of P_s plus N_{eq}, where N_{eq} is equal to $4M_{cf}/d_s$.

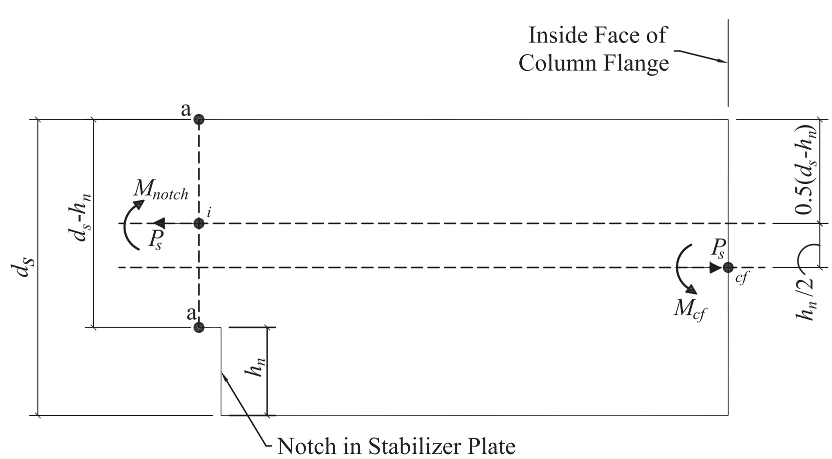

Fig. 8. Type Ib stabilizer plate—free-body diagram: tension side.

Example Problem 2—Type Ib Stabilizer Plate

Figure 9a shows a W16×26 framing to the web of a W14×90 column. The required beam end reaction is R_u = 50 kips. Checks for connection plate lateral torsional buckling and lap eccentricity show that the connection plate is sufficiently strong for these checks without the need of stabilizer plates. However, a type Ib plate is used with the connection. Assume the height of the notch is ¾ in.

1. Generate trial stabilizer and notch dimensions.

2. Determine the required thickness of the type Ib plate.

3. Determine the weld size required at the stabilizer-to-column flange interfaces.

All material is Grade 50.

Solution

Part 1

The length of the stabilizer plate, l_s, is

$$\begin{aligned} l_s &= (d - 2t_f)_{column} \\ &= [14.0 \text{ in.} - (2)(0.71 \text{ in.})] \\ &= 12.6 \text{ in.} \end{aligned}$$

The connection plate thickness, t_p, is 0.375 in. Assume the height of the notch, h_n, is

$$\begin{aligned} h_n &= 2t_p \\ &= (2)(0.375 \text{ in.}) \\ &= 0.75 \text{ in.} \end{aligned}$$

The width of the notch, w_n, is

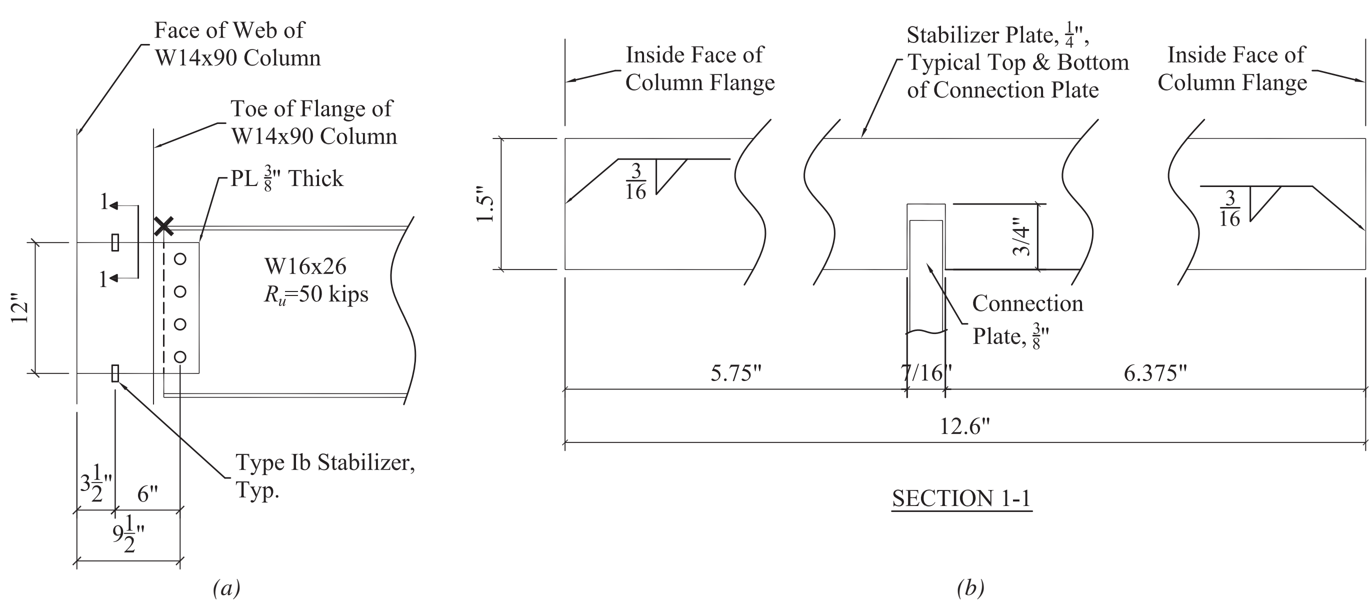

Fig. 9. Connection and details for Example Problem 2: (a) elevation; (b) final details of stabilizer plate.

$$w_n = t_p + 1/16 \text{ in.}$$
$$= 0.375 \text{ in.} + 0.0625 \text{ in.}$$
$$= 0.4375 \text{ in.}$$

Assume the height the contact area of the stabilizer with the connection plate, h_p, is

$$h_p = h_n - 0.5t_p$$
$$= 0.75 \text{ in.} - (0.5)(0.375 \text{ in.})$$
$$= 0.563 \text{ in.}$$

The depth of the stabilizer is estimated to be 2 times the notch depth, h_n. The depth, d_s, is estimated as

$$d_s = 2h_n$$
$$= (2)(0.75 \text{ in.})$$
$$= 1.5 \text{ in.}$$

Try a 1.5-in.-deep stabilizer with a ¾-in.-high × 7⁄16-in.-wide notch.

Part 2

The required axial force in the stabilizer is given by Equation 1 as

$$P_{su} = \frac{0.02 M_{bu} C_d}{L}$$
$$= \frac{(0.02)(50 \text{ kips})(9.50 \text{ in.})(1.0)}{12.0 \text{ in.}}$$
$$= 0.792 \text{ kips}$$

Checks on Section a-a

The stabilizer thickness required for tension yield is (see Equation 10a)

$$M_{notch,u} = P_s\left[0.5(d_s + h_n - h_p)\right]$$
$$= (0.792 \text{ kips})\left[(0.5)(1.5 \text{ in.} + 0.75 \text{ in.} - 0.563 \text{ in.})\right]$$
$$= 0.666 \text{ kip-in.}$$

$$N_{eq} = \frac{4M_{notch}}{d_s - h_n}$$
$$= \frac{(4)(0.666 \text{ kip-in.})}{1.5 \text{ in.} - 0.75 \text{ in.}}$$
$$= 3.55 \text{ kips}$$

$$t_s \geq \frac{N_{Tot,u}}{\phi F_y (d_s - h_n)}$$
$$\geq \frac{0.792 \text{ kips} + 3.55 \text{ kips}}{(0.9)(50 \text{ ksi})(1.5 \text{ in.} - 0.75 \text{ in.})}$$
$$\geq 0.129 \text{ in.}$$

Check at Notch

The stabilizer thickness required for bearing is

$$t_s \geq \frac{P_{su}}{\phi 1.8 F_y h_p}$$
$$\geq \frac{0.792 \text{ kips}}{(0.75)(1.8)(50 \text{ ksi})(0.75 \text{ in.})}$$
$$\geq 0.0156 \text{ in.}$$

Tension yield on section a-a controls; required t_s is greater than or equal to 0.129 in. Use a minimum plate thickness of 0.25 in.

Provide a ¼-in. × 1½-in. × 12⅝-in. stabilizer with a 7⁄16-in. × ¾-in. notch.

Part 3

The weld at the column flange must transfer the required axial force, $P_{su} = 0.792$ kips, and the required moment, M_{cfu}, equal to

$$M_{cfu} = M_{notch,u} - 0.5 h_n P_{su}$$
$$= 0.666 \text{ kip-in.} - (0.5)(0.75 \text{ in.})(0.792 \text{ kips})$$
$$= 0.369 \text{ kip-in.}$$

The total required force for the design of the weld is

$$N_{Tot} = P_{su} + \frac{4 M_{cfu}}{d_s}$$
$$= 0.792 \text{ kips} + \frac{(4)(0.369 \text{ kip-in.})}{1.5 \text{ in.}}$$
$$= 1.78 \text{ kips}$$

The weld size required is

$$D_{req} = \frac{N_{Tot}}{1.392 l \mu n}$$
$$= \frac{1.78 \text{ kips}}{(1.392)(1.5 \text{ in.})(1.5)(1)}$$
$$= 0.568 \text{ sixteenths of an inch}$$

Provide a minimum 3⁄16-in. fillet weld at the column flanges.

The final details of the stabilizer and the welds are shown in Figure 9b.

Type II Stabilizer Plates

Figure 2c shows a type II stabilizer plate. Type II is a variation of the type I with three primary differences: (1) the type II plate has a significantly larger cross-sectional area than what would be required based on the stabilizer plate's required strength and stiffness; (2) the type II plate will qualitatively provide more stiffness to the beam connection than will type I; and (3) the length of the weld at the stabilizer plate to the connection plate is longer, potentially producing a couple that should be considered in the design of the connection and, therefore, acting as part of the load path for transporting beam end shear to the support.

If the type II plate is permitted to float, the design procedure of the single-plate shear connection should be in accordance with the procedure shown in Part 10 of the *Manual*. If not permitted to float and the ends of the stabilizer are welded to the column flanges, the stabilizer will participate in the transfer of the beam end connection to the support. In this case, the *Manual* Part 10 procedure can be adjusted to use "*g*" in place of "*a*." Additionally, the minimum plate thickness requirement for extended shear plate connections as well as the (⅝)t_p weld requirement for both conventional and extended shear plate connections can be ignored.

For the following methodology, refer to Figures 10, 11, and 12. Figure 10 shows a generalized free-body diagram of single-plate shear connection in the presence of a stabilizer plate along with the distribution of shear and moment over the span of the connection plate. Figure 11 shows a specific solution when the bolt group is designed for a moment equal to $M_b = Ra$, while Figure 12 shows a specific solution when the bolt group is designed for a moment $M_b = Rg$.

A Generalized Solution

Refer to Figure 10 for the following discussion.

Shear and Moment at the Bolt Group

Referring to the shear and moment acting on the bolt group,

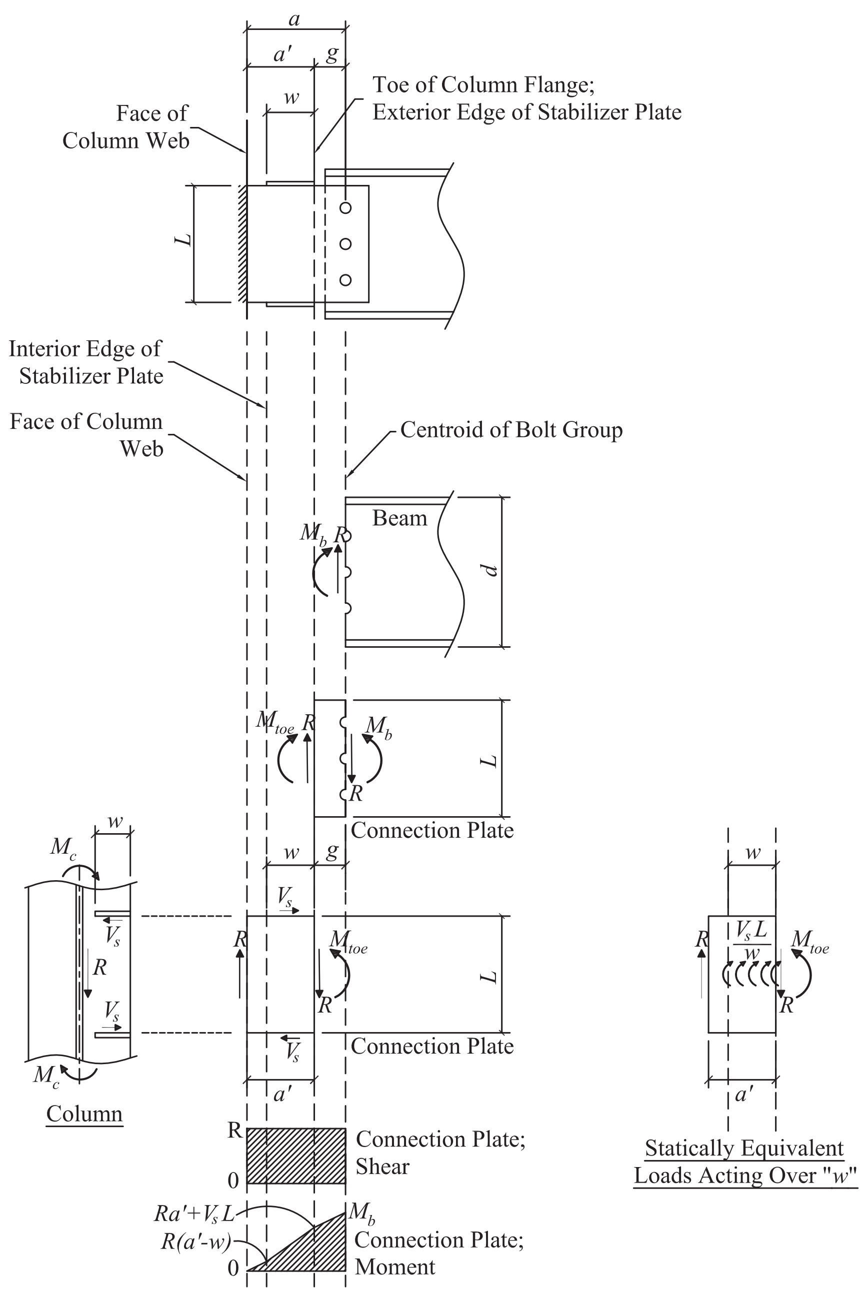

Fig. 10. Generalized free-body diagram of the connection plate in the presence of a stabilizer plate.

the shear in the bolt group, R, is considered to be the beam end reaction. The moment acting on the bolt group is given as M_b.

Shear and Moment on Connection Plate Over the "g" Distance

The shear and moment acting on the bolt group location is R and M_b, respectively. The shear acting on the left edge of the plate (at the toe of the column flange) by inspection is R. The moment, M_{toe}, at the left edge of the plate is

$$\Sigma M_{toe} = 0 = Rg - M_b + M_{toe}$$
$$M_{toe} = M_b - Rg \tag{12}$$

Shear and Moment on Connection Plate Over the "a′" Distance

The shear acting on the left edge of the plate (at the face of the web) by inspection is R. Because it is assumed that only shear is delivered to the support, the moment at this location is considered to be zero. The horizontal shear acting at the stabilizer plate-to-connection plate interface, V_s, is

$$\sum M_{web} = 0 = -M_{toe} + Ra' + \frac{2V_sL}{2}$$
$$V_s = \frac{M_{toe} - Ra'}{L} \tag{13}$$

Loads Acting on Column

The loads acting at the connection plate-to-column web and stabilizer plate-to-connection plate interfaces are transferred to the column. As can be seen in Figure 8, the column web is subjected to a vertical shear equal to R, and the column is subjected to a weak-axis bending moment equal to V_sL.

Shear and Moment Distribution in Connection Plate

Figure 8 shows the resulting shear and moment distribution along the span of the connection plate. The shear is constant with a magnitude of R over the span of the connection from the face of the web to the bolt group. The moment at the face of the web is zero. The moment at the interior side of the stabilizer plate is

$$M_{s,i} = R(a' - w) \tag{14}$$

and is distributed linear at a slope equal to R.

The moment at the interior side of the stabilizer plate is $R(a' - w)$ and increases over the interval of w to the exterior edge of the stabilizer plate to a magnitude, M_{toe}, equal to

$$M_{toe} = R(a' - w) + V_sL + Rw$$
$$M_{toe} = Ra' - Rw + V_sL + Rw$$
$$M_{toe} = Ra' + V_sL \tag{15}$$

and is distributed linearly at a slope equal to $V_sL/w + R$.

The moment acting on the plate at the bolt group is equal to

$$M_b = Ra' + V_sL + Rg \tag{16}$$

and is distributed linearly at a slope equal to R.

***Specific Solution for Bolt Group Moment* $\mathbf{M_b = Ra}$**

Refer to Figure 11 for the following discussion.

In conventional, extended, single-plate shear connections, and in the absence of a stabilizer plate, the bolt group is designed assuming the bolt group carries a moment equal to R times the full eccentricity—that is, a moment $M_b = Ra$. If the bolt group is designed for that "full" moment (i.e., $M_b = Ra$), then substituting this into Equation 16 gives

$$M_b = Ra = Ra' + V_sL + Rg \tag{17}$$

Recognizing that $a' + g = a$, it can be deduced that the relationship shown in Equation 17 can only be true when $V_s = 0$ (recognizing that, practically, R, a', g, and L cannot be taken as zero). If $V_s = 0$, it can be deduced that the presence of the stabilizer plate has no load at the stabilizer-to-connection plate interface parallel to the connection plate, is not part of the load path, and plays no role in the transfer of the beam end reaction to the support; thus, the force distribution is no different than that of a connection with no stabilizer plate. Equally important, it also suggests that the presence of the stabilizer plate does cause the connection to impart rotational demand to the column. Figure 11 shows the force distribution through the connection when the bolt group is designed for a moment equal to $M_b = Ra$.

***Specific Solution for Bolt Group Moment* $\mathbf{M_b = Rg}$**

Refer to Figure 12 for the following discussion.

Suppose the bolt group is designed to transfer the beam end reaction from the bolt group to the toe of the stabilizer plate (i.e., a moment equal to $M_b = Rg$). Setting $M_b = Rg$ and substituting into Equation 16 gives

$$M_b = Rg = Ra' + V_sL + Rg \tag{18}$$

Upon inspection of Equation 18, it can be seen that V_s is a non-zero quantity. In other words, there is a shear force acting parallel to the plate at the stabilizer-to-connection plate interface. The shear force, V_s, can be described by rearranging Equation 18 and solving for V_s given by

$$V_s = \frac{Rg - Ra' - Rg}{L} = -\frac{Ra'}{L} \tag{19}$$

Multiplying both sides of Equation 19 by L gives

$$V_s L = -Ra' \tag{20}$$

It can be seen that, in this case, the magnitude of the moment at the toe of the stabilizer plate is zero by substituting $M_b = Rg$ into Equation 9 giving

$$M_{toe} = M_b - Rg = Rg - Rg = 0 \tag{21}$$

The change in moment from the face of the column web to the interior edge of the stabilizer plate is not affected by

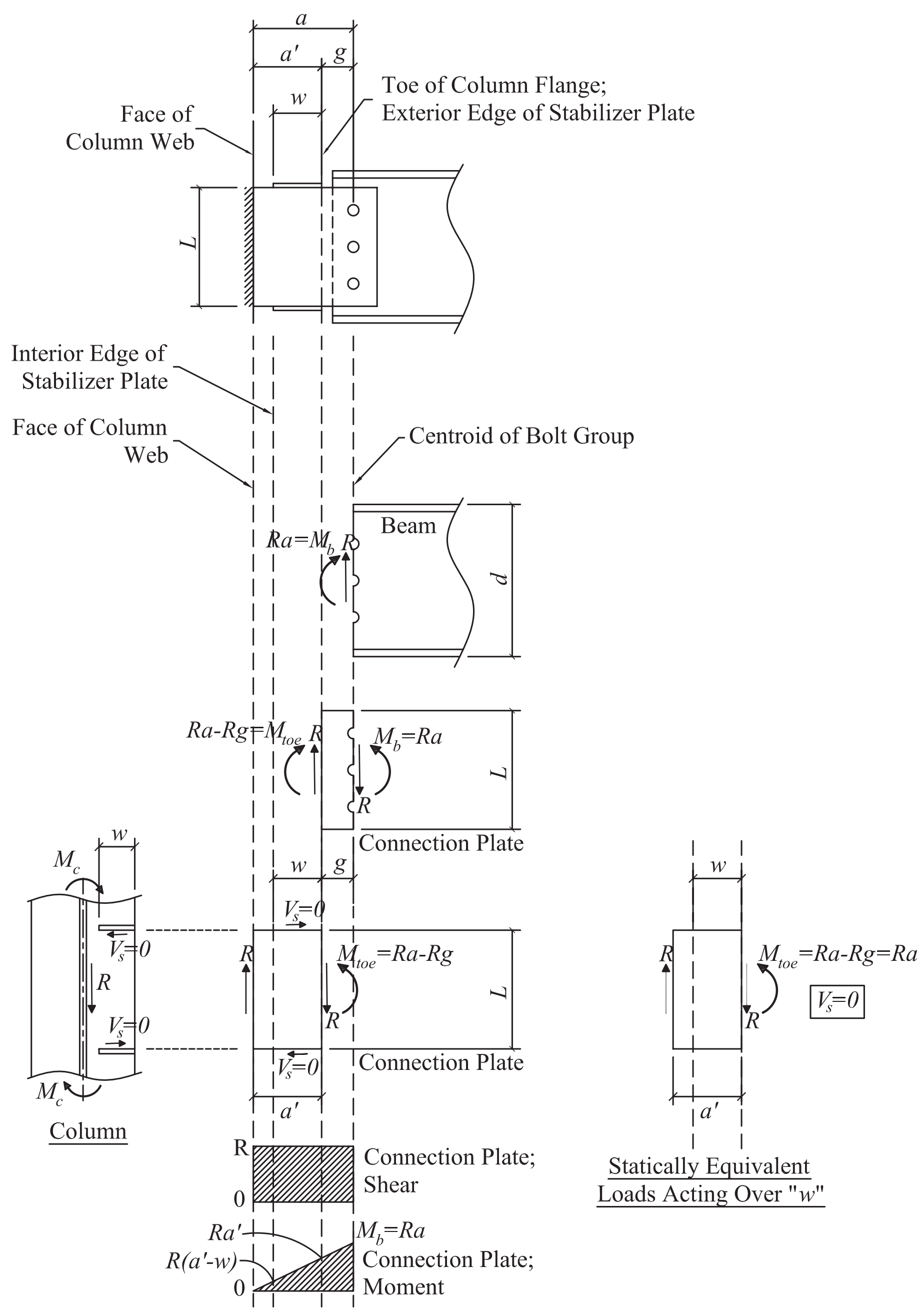

Fig. 11. Theoretical free-body diagram of connection plate with or without presence of a stabilizer plate; bolt group designed for full eccentricity, $M_b = Ra$.

the stabilizer force and is $R(a' - w)$. The change in moment from the interior edge of the stabilizer to the toe of column flange is the sum of the moments caused by the V_s force plus the area under the shear distribution. Thus, the moment at the toe of the column is

$$M_{toe} = R(a' - w) + Rw + \frac{V_s L}{w} w$$

$$M_{toe} = Ra' - Rw + Rw + V_s L \tag{22}$$

Substituting Equation 20 into Equation 22 gives

$$M_{toe} = Ra' - Rw + Rw + \frac{-Ra'}{L} L$$

$$M_{toe} = Ra' - Rw + Rw - Ra' = 0 \tag{23}$$

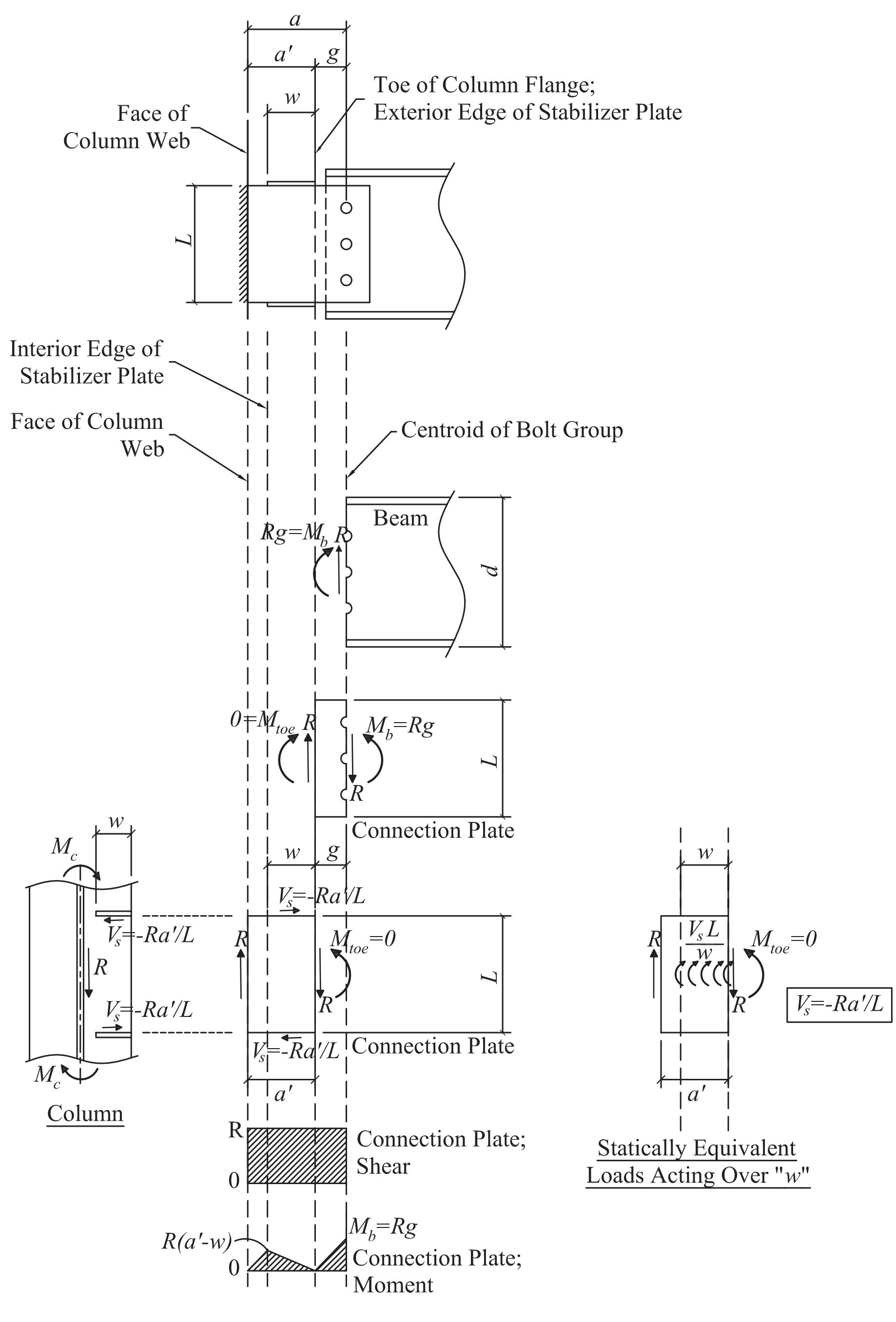

Fig. 12. Free-body diagram of connection plate in presence of a stabilizer plate; bolt group designed for eccentricity equal to g, $M_b = Rg$.

The force distribution, load path, and distribution of shear and moment when $M_b = Rg$ is shown in Figure 12. Several observations can be made when referring to Figure 12: (1) The shear demand on the connection plate is the same regardless of whether or not a stabilizer plate is present and regardless of the bolt group being designed for $M_b = Ra$ or the reduced $M_b = Rg$; (2) the moment demand on the plate is reduced when $M_b = Rg$ relative to $M_b = Ra$; and (3) because the magnitude of V_s is non-zero, a rotational demand is imparted to the column.

$M_b = Ra$ versus $M_b = Rg$

In the preceding discussion, it has been shown with statics that the presence of a stabilizer plate does not impart a rotational demand to the column if the bolt group is designed for a moment equal to $M_b = Ra$; when the bolt group is designed for a moment equal to $M_b = Rg$, a rotational demand is imparted to the column. On face value, it might be surmised that it is advantageous to design the bolt group for the full eccentricity from the bolt group to the weld line. However, we must recognize that stiffness attracts load.

Statically it seems that $V_s = 0$ when $M_b = Ra$. However, the attachment of the stabilizer plate to the connection plate—and, ultimately, connecting the stabilizer plate to the flanges of the columns—is going to produce a condition in which the stabilizer plate becomes part of the load path for transferring the beam end reaction to the column support. The weld at the stabilizer-to-connection plate will carry load (V_s not equal to zero), which will indeed impart a rotational demand to the column, M_c. The authors caution designers to carefully consider the impact on the behavior of the connection and the rotational demands on the support in the presence of a type II or type III stabilizer plate. Ultimately, the stabilizer plate will participate in the load transfer and impart a rotational demand to the column. Consequently, the authors suggest the following:

1. Only use a stabilizer plate when all other alternatives, such as changing the geometry and proportioning of the connection, have been exhausted to no avail. Stabilizer plates are rarely required in this type of connection (see Thornton and Fortney, 2011).
2. In the unlikely event that a stabilizer plate is required, recognize that the plate will play a role in load transfer. As such, the bolt group only need be designed for a moment $M_b = Rg$, and the weak axis rotational demand imparted to the column, M_c, should be accounted for when sizing the column. For typical connections, the rotational demand on the produced by the V_s force will be relatively small; however, because it is a weak axis rotational demand, it may require consideration.

Design of Type II Stabilizer Plates

In addition to the required axial strength and stiffness of the stabilizer plate, a type II stabilizer plate would have to be checked for shear and bending as shown in the shear and bending distribution presented in Figure 13. The authors recognize that the V_s force is not located directly at the center of the stabilizer plate, offset from the center by $(t_w + t_p)/2$, but, for simplicity, it is assumed to be. Shear rupture of the stabilizer plate beneath the weld of the stabilizer plate to the connection plate also needs to be checked.

The cross-sectional dimensions of a type II stabilizer plate, based on the required shear, V_s, are given in Equation 24.

$$\phi V_n = V_u = \frac{V_{su}}{2}$$
$$\phi 2\left(0.6F_y\right)A_s = V_{su}$$

$$A_s = \frac{V_{su}}{\phi 1.2F_y} \qquad \text{(LRFD)} \tag{24a}$$

$$\frac{V_n}{\Omega} = \frac{V_{sa}}{2}$$
$$\frac{0.6F_yA_s}{\Omega} = \frac{V_{sa}}{2}$$

$$A_s = \frac{\Omega V_{sa}}{1.2F_y} \qquad \text{(ASD)} \tag{24b}$$

In Equations 24a and 24b, $\phi = 1.00$ and $\Omega = 1.50$.

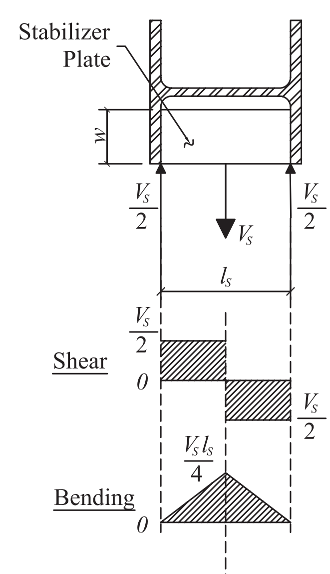

Fig. 13. Shear and bending in stabilizer plate.

The required thickness of a type II stabilizer plate, based on the required bending, $V_s l_s/4$, are given in Equation 25.

$$\phi M_n = \frac{V_{su} l_s}{4}$$

$$\phi F_y Z_s = \frac{\phi F_y t_s w^2}{4} = \frac{V_{su} l_s}{4}$$

$$\phi F_y t_s w^2 = V_{su} l_s$$

$$t_s \geq \frac{V_{su} l_s}{\phi F_y w^2} \qquad \text{(LRFD)} \qquad (25a)$$

$$\frac{M_n}{\Omega} = \frac{V_{sa} l_s}{4}$$

$$\frac{F_y Z_s}{\Omega} = \frac{F_y t_s w^2}{4\Omega} = \frac{V_{sa} l_s}{4}$$

$$\frac{F_y t_s w^2}{\Omega} = V_{sa} l_s$$

$$t_s = \frac{\Omega V_{sa} l_s}{F_y w^2} \qquad \text{(ASD)} \qquad (25b)$$

In Equations 25a and 25b, $\phi = 0.90$ and $\Omega = 1.67$.

The thickness, t_s, of the stabilizer plate based on shear rupture beneath the weld of the stabilizer plate to the connection plate is determined as follows.

$$\phi R_n = \phi(0.6F_u) w t_s = \frac{V_{su}}{2}$$

$$t_s = \frac{V_{su}}{\phi 2(0.6F_u) w} \qquad \text{(LRFD)} \qquad (26a)$$

$$\frac{R_n}{\Omega} = \frac{(0.6F_u) w t_s}{\Omega} = \frac{V_{sa}}{2}$$

$$t_s = \frac{V_{sa}\Omega_s}{2(0.6F_u) w} \qquad \text{(ASD)} \qquad (26b)$$

In Equation 26a and 26b, $\phi = 0.75$, $\Omega = 2.00$.

The weld at the stabilizer plate to the connection plate is sized for the force, V_s, and the weld at the stabilizer plate to the column flanges is sized for $V_s/2$.

Example Problem 3—Type II Stabilizer Plate

Problem Statement

Figure 14a shows a single-plate shear connection used as a beam end connection for a W30×90 framing to the web of a W12×152 column. The required shear force for the design of the connection is $R_u = 150$ kips. For the given geometry, an evaluation of the lateral torsional buckling capacity of the single-plate connection and the effect of lap eccentricity indicate that a stabilizer plate is not required. However, the engineer of record has implemented a design parameter requiring all single-sided connections to be supported with a stabilizer plate with a width approximately equal to $b_f/2 - k_1$. Additionally, the bolt group in the single-plate shear connections must be designed for a moment equal to $M_b = Ra$.

1. Demonstrate that a stabilizer plate is not required based on:

    a. Lateral torsional buckling of the plate.

    b. The effect of lap eccentricity.

2. Determine the required cross-sectional dimensions of the type II stabilizer plate, assuming $M_b = Ra$.

3. Determine fillet weld sizes required for attaching the stabilizer plate assuming $M_b = Ra$.

Assume all material is GR 50.

Solution

Part 1a

The lateral torsional buckling strength of the plate is

$$\phi R_n = \phi \frac{1{,}500\pi L t^3}{a^2}$$
$$= (0.9)\frac{(1{,}500 \text{ ksi})\pi(24 \text{ in.})(0.50 \text{ in.})^3}{(9 \text{ in.})^2}$$
$$= 157 \text{ kips} > 150 \text{ kips} \quad \rightarrow \quad \text{stabilizer plate not required}$$

Part 1b

The required torsional moment strength due to lap eccentricity is

$$M_{tu} = R\left(\frac{t_w + t_p}{2}\right)$$
$$= (150 \text{ kips})\left(\frac{0.47 \text{ in.} + 0.50 \text{ in.}}{2}\right)$$
$$= 72.8 \text{ kip-in.}$$

The available torsional moment strength is

$$\phi M_t = \left[\phi_v\left(0.6F_{yp}\right) - \frac{R_u}{Lt_p}\right]\left(\frac{Lt_p^2}{2}\right) + \frac{2R_u^2\left(t_w + t_p\right)b_f}{\phi_b F_{yb} L_s t_w^2}$$
$$= \left[(1.0)(0.6)(50 \text{ ksi}) - \frac{150 \text{ kips}}{(24 \text{ in.})(0.50 \text{ in.})}\right]\left(\frac{(24 \text{ in.})(0.50 \text{ in.})^2}{2}\right) + \frac{(2)(150 \text{ kips})^2\left(0.47 \text{ in.} + 0.50 \text{ in.}\right)(10.4 \text{ in.})}{(0.90)(50 \text{ ksi})(28 \text{ ft})(12 \text{ in./ft})(0.47 \text{ in.})^2}$$
$$= 52.5 \text{ kip-in.} + 136 \text{ kip-in.}$$
$$= 188 \text{ kip-in.} > 72.8 \text{ kip-in.} \quad \rightarrow \quad \text{stabilizer plate not required}$$

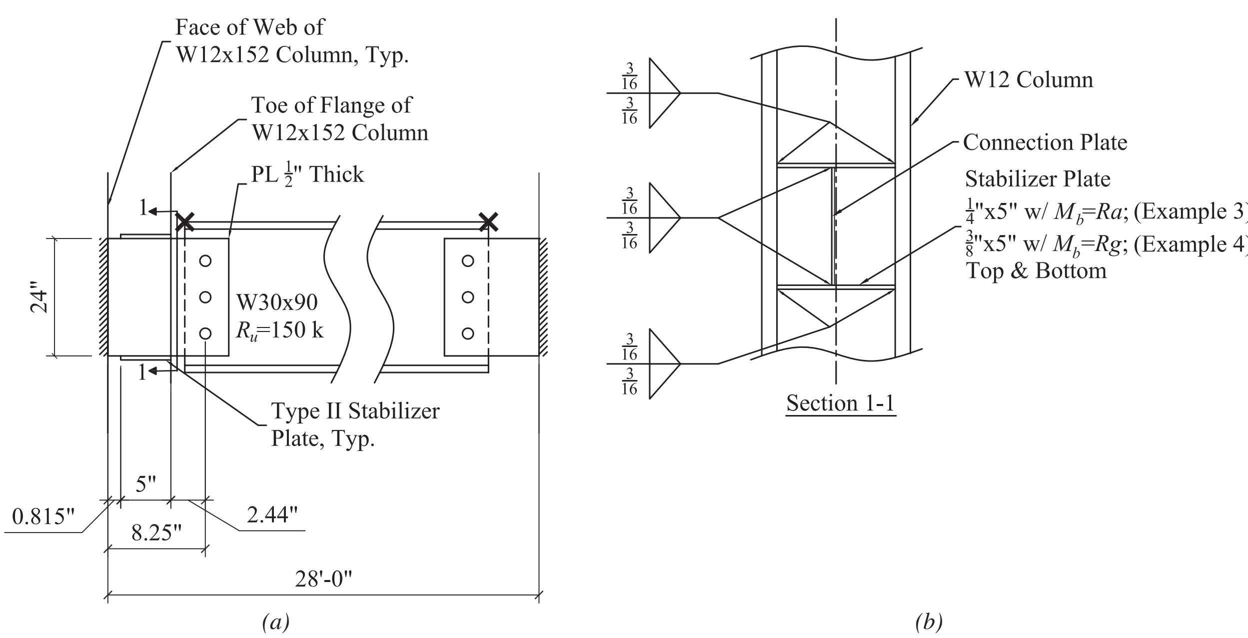

Fig. 14. Single-plate shear connection—examples 3 and 4: (a) elevation; (b) final stabilizer plate details.

Part 2

In Part 1 of this solution, it is demonstrated that a stabilizer plate is not required. However, the problem statement requires a stabilizer plate with a width of $w = (b_f/2) - k_1$ be provided regardless of the outcome of Part 1.

The width, w, of the stabilizer plate is half the flange width of the column minus k_1.

$$\begin{aligned} w &= \frac{b_f}{2} - k_1 \\ &= \frac{12.5 \text{ in.}}{2} - 1.25 \text{ in.} \\ &= 5.00 \text{ in.} \end{aligned}$$

Take $w = 5.00$ in.

Stabilizer plate size based on required axial strength using Equation 3a.

$$\begin{aligned} A_s &= \frac{0.02R_u a}{\phi F_y L} \\ &= \frac{(0.02)(150 \text{ kips})(8.25 \text{ in.})}{(0.75)(50 \text{ ksi})(24.0 \text{ in.})} \\ &= 0.0275 \text{ in.}^2 \end{aligned}$$

Given the width of the plate is 5.0 in., the required plate thickness is

$$\begin{aligned} t_s &= \frac{0.0275 \text{ in.}^2}{5.0 \text{ in.}} \\ &= 0.006 \text{ in.} \end{aligned}$$

Stabilizer plate size based on required axial stiffness using Equation 6a.

$$\begin{aligned} l_s &= \left(d - 2t_f\right)_{column} \\ &= 13.7 \text{ in.} - (2)(1.40 \text{ in.}) \\ &= 10.9 \text{ in.} \end{aligned}$$

$$\begin{aligned} A_s &= \frac{5R_u l_s}{\phi LE} \\ &= \frac{(5)(150 \text{ kips})(10.9 \text{ in.})}{(0.75)(24.0 \text{ in.})(29{,}000 \text{ ksi})} \\ &= 0.016 \text{ in.}^2 < 0.0275 \text{ in.}^2 \quad \rightarrow \quad \text{strength controls over stiffness} \end{aligned}$$

The required axial force in the stabilizer plate is determined using Equation 1.

$$\begin{aligned} P_{su} &= \frac{0.02M_{bu}C_d}{L} \\ &= \frac{(0.02)(150 \text{ kips})(8.25 \text{ in.})(1.0)}{24.0 \text{ in.}} \\ &= 1.03 \text{ kips} \end{aligned}$$

To determine the transverse shear and bending on the stabilizer plate, the force V_s is required. From the previous discussion, when the bolt group is designed for a moment $M_b = Ra$, the force $V_s = 0$. To demonstrate this, the moment at the exterior edge of the stabilizer (equal to the moment at the toe of the column flange because the stabilizer is flush with the toe, i.e., $a' + g = a$) is given by Equation 12:

$$M_{toe} = M_b - Rg = Ra - Rg = Ra'$$

$$\begin{aligned} M_{toe} &= (150 \text{ kips})(8.25 \text{ in.} - 2.44 \text{ in.}) \\ &= 872 \text{ kip-in.} \end{aligned}$$

Substituting the value for M_{toe} into Equation 13 shows that $V_s = 0$ when $M_b = Ra$.

$$\begin{aligned} V_{su} &= \frac{M_{toe} - Ra'}{L} \\ &= \frac{872 \text{ kip-in.} - (150 \text{ kips})(5.815 \text{ in.})}{24.0 \text{ in.}} \\ &= 0 \end{aligned}$$

Because $V_s = 0$, there are no transverse loads on the stabilizer plate. Thus, transverse shear and bending of the stabilizer plate do not apply. The stabilizer and its attachments only need be based on the axial force, P_{su}.

Axial strength controls (t_s greater than 0.006 in.).

Use a ¼-in. × 5.0-in. stabilizer with 3⁄16-in. fillets welds.

Part 3

Weld at Connection Plate to Stabilizer Plate

The force to be transferred by the weld is the axial force $P_s = 1.03$ kips. Assuming a 5-in.-long × 3⁄16-in. fillet weld is provided on both sides of the connection plate, and on both sides of the stabilizer at the column flanges, the weld strength provided is

$$\theta = 90°$$

$$\mu = 1.0 + 0.5\sin^{1.5}(90°) = 1.50$$

$$\begin{aligned} \phi R_w &= 1.392 Dln\mu \\ &= (1.392)(3 \text{ sixteenths})(5.0 \text{ in.})(2)(1.50) \\ &= 62.6 \text{ kips} > 1.03 \text{ kips} \quad \textbf{OK} \end{aligned}$$

Provide a 5-in.-long × 3⁄16-in. fillet weld on both sides of the connection plate. The final details of the stabilizer plate and welds are shown in Figure 14b.

Example Problem 4—Type II Stabilizer Plate

Problem Statement

For the connection given in Example 3, it was required that a stabilizer plate be provided and that the bolt group be designed for a moment equal to $M_b = Ra$. As discussed previously, in the presence of a stabilizer plate, attached with welds at the connection plate and the column flanges, there will be a moment imparted to the column regardless of the statics discussion supporting Equation 17. Considering that imparting a moment to the column is inevitable in the presence of a stabilizer, the cost of providing the stabilizer plates can be offset, somewhat, by reducing the moment for which the bolt group is designed ($M_b = Rg$). A force V_s not equal to zero will be present.

For the connection shown in Example 3, size the stabilizer plate and welds assuming the bolt group is designed for a moment equal to $M_b = Rg$.

1. Describe the how the approach to the design would differ from the solution given in Example 2.

2. Size the stabilizer plate and welds assuming the bolt group is designed for a moment equal to $M_b = Rg$.

3. Draw the shear and moment distribution along the span of the connection plate (from the face of the support to the centroid of the bolt group). This is a special case of Figure 12.

Solution

Part 1

The column moment, M_c, will be $V_sL/2$ or V_sL (see Figure 12), depending on whether the column is continuous or discontinuous at the connection location, respectively.

When the bolt group is designed for a moment $M_b = Ra$, statics dictates that $V_s = 0$, but the presence of the stabilizer plates will induce a non-zero statically indeterminate V_s force due to simple beam end rotation. The authors believe that it would be better to consider the V_s force and its effect on the column. A statically determinate force V_s can be calculated when the bolt group can be designed for the shear, R, and the moment, $M_b = Rg$. Because $g < a$, a more economical bolt group will result, and the effect of the stabilizer plate on the column can be easily evaluated.

Part 2

When the bolt group is designed for a moment $M_b = Rg$, the transverse force on the stabilizer is not zero. From Equation 23, when $M_b = Rg$, the moment at the exterior edge of the stabilizer (toe of the column in this example) is $M_{toe} = 0$.

From Equation 20, when $M_b = Rg$, the transverse force, V_s, is given as

$$\begin{aligned} V_s &= -\frac{Ra'}{L} \\ &= -\frac{(150 \text{ kips})(5.815 \text{ in.})}{24.0 \text{ in.}} \\ &= -36.3 \text{ kips} \end{aligned}$$

Stabilizer plate area based on shear yielding strength is determined using Equation 24a.

$$\begin{aligned} A_s &= \frac{V_{su}}{\phi 1.2 F_y} \\ &= \frac{36.3 \text{ kips}}{(1.00)(1.2)(50 \text{ ksi})} \\ &= 0.605 \text{ in.}^2 \end{aligned}$$

The required thickness for shear yield of the 5.0-in.-wide plate is

$$\begin{aligned} t_s &= \frac{0.605 \text{ in.}^2}{5.0 \text{ in.}} \\ &= 0.121 \text{ in.} \end{aligned}$$

Stabilizer plate thickness based on bending strength is determined using Equation 25a.

$$\begin{aligned} t_s &= \frac{V_{su} l_s}{\phi F_y w^2} \\ &= \frac{(36.3 \text{ kips})(10.9 \text{ in.})}{(0.90)(50 \text{ ksi})(5.0 \text{ in.})^2} \\ &= 0.352 \text{ in.} \end{aligned}$$

Stabilizer plate thickness based on shear rupture is determined using Equation 26a.

$$t_s = \frac{V_{su}}{\phi 2(0.6F_u)w}$$
$$= \frac{36.3\text{ kips}}{(0.75)(2)(0.6)(65\text{ ksi})(5.0\text{ in.})}$$
$$= 0.124\text{ in.}$$

Bending controls the required plate size.

Provide a ⅜-in. × 5-in. stabilizer plate.

Weld at Connection Plate to Stabilizer Plate

The force to be transferred by the weld is the shear force $V_s = 36.3$ kips and $P_s = 1.03$ kips. Assuming a 5-in.-long × 3⁄16-in. fillet weld is provided on both sides of the connection plate, the weld strength provided is

$$R = \sqrt{(1.03\text{ kips})^2 + (36.4\text{ kips})^2}$$
$$= 36.4\text{ kips}$$
$$\theta = \tan^{-1}\left(\frac{1.03\text{ kips}}{36.4\text{ kips}}\right)$$
$$= 1.62^\circ$$
$$\mu = 1.0 + 0.5\sin^{1.5}(1.62^\circ)$$
$$= 1.00$$
$$\phi R_w = 1.392 Dl n\mu$$
$$= (1.392)(3\text{ sixteenths})(5\text{ in.})(2)(1.00)$$
$$= 41.8\text{ kips} > 36.3\text{ kips} \qquad \textbf{OK}$$

Provide a 5-in.-long × 3⁄16-in. fillet weld on both sides of the connection plate.

Weld at Stabilizer Plate to the Column Flange

The required force to be transferred by the weld is one-half of the shear force, $V_s/2 = 36.3\text{ kips}/2 = 18.2$ kips, and $P_s = 1.03$ kips. Assuming a 5-in.-long × 3⁄16-in. fillet weld is provided on both sides of the stabilizer plate, the weld strength provided is

$$R = \sqrt{(1.03\text{ kips})^2 + (18.2\text{ kips})^2}$$
$$= 18.2\text{ kips}$$
$$\theta = \tan^{-1}\left(\frac{1.03\text{ kips}}{18.2\text{ kips}}\right)$$
$$= 3.24^\circ$$
$$\mu = 1.0 + 0.5\sin^{1.5}(3.24^\circ)$$
$$= 1.01$$
$$\phi R_w = 1.392 Dl n\mu$$
$$= (1.392)(3\text{ sixteenths})(5\text{ in.})(2)(1.01)$$
$$= 42.2\text{ kips} > 18.2\text{ kips} \qquad \textbf{OK}$$

Provide 5-in.-long × 3⁄16-in. fillet weld on both sides of the stabilizer plate.

Figure 12b shows the final details of the stabilizer plate when the bolt group is designed for a moment $M_b = Rg$.

Part 3

In this case, the bolt group is designed for a moment, M_{bu}, equal to

$$\begin{aligned} M_{bu} &= R_u g \\ &= (150 \text{ kips})(2.44 \text{ in.}) \\ &= 366 \text{ kip-in.} \end{aligned}$$

The moment at the interior edge of the stabilizer plate, $M_{s,i}$, is

$$\begin{aligned} M_{s,iu} &= R_u (a' - w) \\ &= (150 \text{ kips})(8.815 \text{ in.} - 0.815 \text{ in.}) \\ &= 122 \text{ kip-in.} \end{aligned}$$

The moment at the exterior edge of the stabilizer plate ($M_{toeu} = M_{s,eu}$) is $M_{s,iu}$ plus the area under the shear distribution over the interval "w" (5.0 in.) plus the moment associated with the stabilizer shear force, V_{su}, and is

$$\begin{aligned} M_{toe} &= 122 \text{ kip-in.} + (150 \text{ kips})(5.0 \text{ in.}) - (36.3 \text{ kips})(24 \text{ in.}) \\ &\approx 0 \end{aligned}$$

The moment at the bolt group is

$$\begin{aligned} M_{bu} &= M_{toe,u} + R_u g \\ &= 0 + (150 \text{ kips})(2.44 \text{ in.}) \\ &= 366 \text{ kip-in.} \end{aligned}$$

The free-body diagram and distribution along the span of the connection plate is shown in Figure 15.

Type III Stabilizer Plates

Type III stabilizer plates are not only connected to the column flanges, but also to the column web. The discussion presented for type II stabilizer plates are applicable to type III stabilizer plates as well, with the exception that the flexure in the plate can be neglected. The stabilizer plate would need to be checked for shear rupture at the connection plate-stabilizer plate and column flange-stabilizer plate interfaces for a shear equal to $V_s/2$.

The design approach given in the *Manual* Part 10 for conventional and extended single-plate shear connections does not apply in the presence of this type of stabilizer plate. Therefore, the maximum plate thickness criterion and the $(5/8)t_p$ weld criterion do not apply.

Example Problem 5—Type III Stabilizer Plate

Problem Statement

The connection given in Example 4 is used. In that example, the bolt group is designed for the combination of the required shear, R_u, and a moment equal to $M_b = Rg$. In lieu of the type II plate used in Example 4, a type III plate is required.

Describe how the approach to the design would differ from the solution given in Example 4.

Solution

For this type of stabilizer plate, the bolt group is designed for a moment equal to $M_b = Rg$. Because the type III plate is attached to the column web as well as the column flanges, shear yielding and bending due to the V_s force are not applicable. The following applicable limits states and considerations apply. Also, because the V_s force acts over the entire distance from the face of the web to the exterior edge of the stabilizer, the moment in the connection plate over that interval is zero (i.e., the connection plate is in pure shear over the interval).

1. Shear rupture of the stabilizer at the stabilizer-to-connection plate weld.

2. Axial tension due to the required P_s bracing force.

3. Presence of the type III stabilizer plate as it plays the role of a continuity plate for the moment connection(s) framing into the column flanges. The welds of the continuity plate to the column are sized based on the requirements for the moment connection(s). The load from the P_s force must be included with the moment connection requirements when determining the required continuity plate-to-column web/flanges welds.

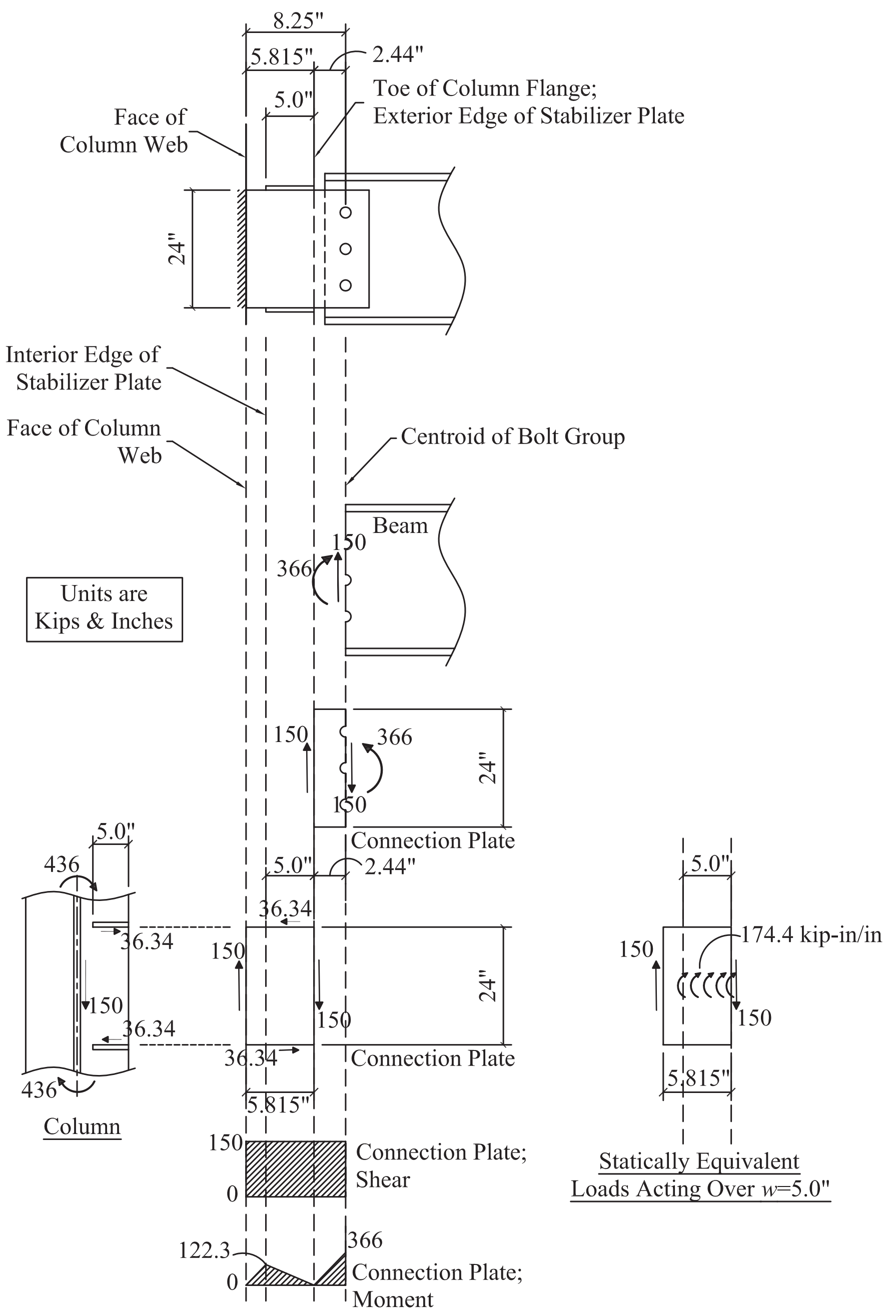

Fig. 15. Free-body diagram of force distribution for Example 4.

SUMMARY

Only after all attempts to proportion the geometry and hardware of a single-plate shear connection to have adequate lateral torsional buckling and lap eccentricity strength, without the use of a stabilizer plate, have been exhausted, can a stabilizer plate be considered as an option.

Three types of stabilizer plates are discussed in this paper: types I, II, and III. Types I and II may be used when members framing in from the perpendicular direction require only simple shear connections or moment connections with no continuity plates. The type III connection is applicable when the perpendicular member framing into the joint is a moment connection that requires continuity plates.

For each type of stabilizer plate, the analysis and design of the beam end connection, and the loads imparted on the support, need to be considered. However, one option for eliminating a need for reevaluation of the connection design and the impact on the support is to allow the stabilizer plate to "float" inside the column flanges. If this option is used, the stabilizer plate cross-section need only satisfy axial strength and stiffness requirements, in addition to bearing at the column flange and buckling on the compression half of the stabilizer plate.

The analysis of the stabilizer plate involves an evaluation of the required axial strength and stiffness assuming the stabilizer plate provides nodal bracing to the connecting plate. When the stabilizer plate is welded to the column flanges, the weld needs to be designed for the required axial force, P_s, imposed on the tension half of the stabilizer plate and the shear force, V_s, if present.

For type I stabilizer plates, it is assumed that the stabilizer plate is flexible enough such that the plate goes along for the ride during beam end rotation and that no shear or bending is imposed on the stabilizer plate. The type Ia plate is presented only for the purpose of generating a generalized analysis and design procedure. If a type I plate is desired, the type Ib plate would be more practical with regard to surviving transportation and erection. Recommendations have been made on the dimensions of the notch and attachment of the stabilizer plate to column flanges. It is recommended that the connection plate not be attached to the stabilizer plate, but rather allow simple beam end rotation by permitting the connection plate to float within the notch. This also ensures that the type Ib stabilizer plate does not become part of the load path for transferring the beam end reaction.

For type II stabilizer plates, it is assumed that the plate provides stiffness that would require an evaluation of the shear and bending imposed on the stabilizer plate. Type III stabilizer plates need only be evaluated for required axial stiffness and strength assuming the stabilizer plate has two pinned ends and two free sides. Shear and bending in a type III stabilizer plate are not applicable. However, shear rupture beneath the connected interfaces should be evaluated.

The design of the bolt group is also an important consideration. For type II stabilizer plates, if the bolt group in the beam end connection is evaluated as if the stabilizer plate is not present (i.e., design bolt group for $M_b = Ra$), statics suggest that no moment is imparted to the support. However, the stiffness provided by the weld at the stabilizer plate-to-column flange will attract load, forcing the stabilizer plate to become part of the load path and ultimately imparting rotational demand on the support. It is recommended that if a type II plate is used, and not permitted to float, the bolt group should be designed for a moment $M_b = Rg$. This reduces the moment demand on the bolt group and reduces the moment demand on the connecting plate. A moment generated by the V_s force will be imparted to the column in this condition.

When the bolt group is designed for a moment $M_b = Rg$, the moment would need to be considered when evaluating the strength of the column. Regardless of the moment considered in the bolt group, the presence of the stabilizer plate, if not permitted to float between the column flanges, alters the behavior of the connection and reduces the eccentricity on the bolt group. In this condition, the weld of the connection plate to the support can be sized based on the required shear while neglecting the ductility checks that require a minimum weld size equal to ⅝ times the thickness of the connection plate and a maximum plate thickness.

SYMBOLS

A_s Gross cross-sectional area of stabilizer plate

E Young's modulus (29,000 ksi)

F_y Nominal specified yield strength

F_u Nominal specified tensile strength

L Length of single-plate connection (vertical dimension)

M_b Moment in bolt group

M_b Moment in single-plate shear connection

M_c Moment imparted to column by stabilizer plate force, V_s

M_n Nominal moment strength

$M_{s,i}$ Moment in connection plate at the interior edge of the stabilizer plate

M_{toe} Moment in connection plate at toe of column flange when exterior edge of stabilizer plate is aligned with toe of column flange ($M_{s,e} = M_{toe}$ in this condition)

P_s Nominal required axial force in stabilizer plate

R Required shear strength; R_a for ASD; R_u for LRFD

R_n Nominal shear strength

R_w Nominal weld strength

a Distance from face of support to centroid of the bolt group

a' Distance from face of support to exterior edge of stabilizer plate

d Depth of beam

g Distance from centroid of bolt group to exterior edge of stabilizer plate

h_n Height of notch in a type Ib stabilizer plate

h_p Vertical dimension on contact area between connection plate and stabilizer in a type Ib stabilizer plate

l_s Length of stabilizer plate

t_f Thickness of beam or column flange

t_p Thickness of connection plate

t_s Thickness of stabilizer plate

w Width of stabilizer plate

w_n Width of notch in a type Ib stabilizer plate

Ω ASD strength reduction factor

β_s Nominal required stabilizer plate stiffness

ϕ LRFD strength reduction factor

REFERENCES

AISC (2010), *Specification for Structural Steel Buildings*, ANSI/AISC 360-10, American Institute of Steel Construction, Chicago, IL.

AISC (2011), *Steel Construction Manual*, 14th ed., American Institute of Steel Construction, Chicago, IL.

Thornton, W.A., and Fortney, P.J. (2011), “On the Need for Stiffeners for and the Effect of Lap Eccentricity on Extended Shear Tabs,” *Engineering Journal*, AISC Vol. 48, No. 2, pp. 117–125.