EXAMPLE II.A-19B EXTENDED SINGLE-PLATE CONNECTION SUBJECT TO AXIAL AND SHEAR LOADING

Given:

Verify the available strength of an extended single-plate connection for an ASTM A992 W18×60 beam to the web of an ASTM A992 W14×90 column, as shown in Figure II.A-19B-1, to support the following beam end reactions:

LRFD	ASD
Shear, V_u = 75 kips Axial tension, N_u = 60 kips	Shear, V_a = 50 kips Axial tension, N_a = 40 kips

Use 70-ksi electrodes and ASTM A572 Grade 50 plate.

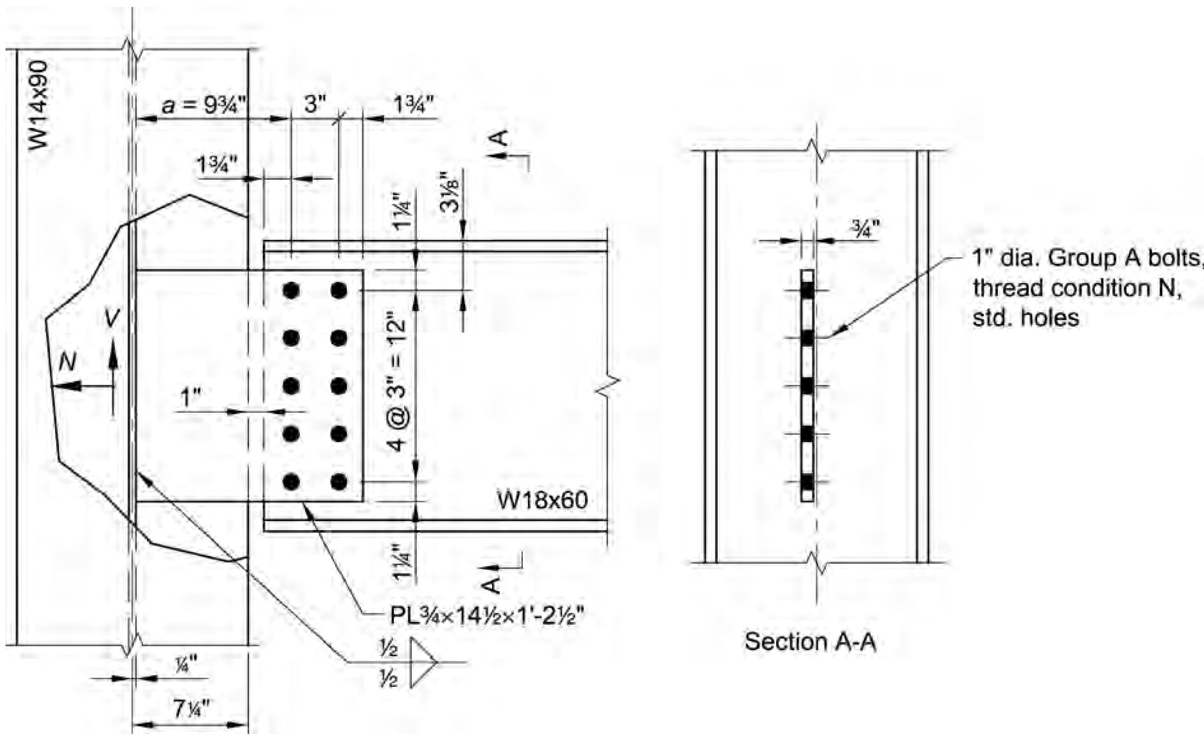


Fig. II.A-19B-1. Connection geometry for Example II.A-19B.

Solution:

From AISC *Manual* Table 2-4 and Table 2-5, the material properties are as follows:

Beam, column
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Plate
ASTM A572 Grade 50
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W18×60
$A_g = 17.6$ in.^{2}
d = 18.2 in.
t_w = 0.415 in.
b_f = 7.56 in.
t_f = 0.695 in.

Column
W14×90
d = 14.0 in.
t_w = 0.440 in.
k_{des} = 1.31 in.

From AISC *Specification* Table J3.3, for 1-in.-diameter bolts with standard holes:

d_h = 1⅛ in.

Per AISC *Specification* Section J3.2, standard holes are required for both the plate and beam web because the beam axial force acts longitudinally to the direction of a slotted hole and bolts are designed for bearing.

The resultant load is determined as follows:

LRFD	ASD
$R_u = \sqrt{V_u^2 + N_u^2}$ $= \sqrt{(75 \text{ kips})^2 + (60 \text{ kips})^2}$ $= 96.0 \text{ kips}$	$R_a = \sqrt{V_a^2 + N_a^2}$ $= \sqrt{(50 \text{ kips})^2 + (40 \text{ kips})^2}$ $= 64.0 \text{ kips}$

The resultant load angle is determined as follows:

LRFD	ASD
$\theta = \tan^{-1}\left(\dfrac{60 \text{ kips}}{75 \text{ kips}}\right)$ $= 38.7°$	$\theta = \tan^{-1}\left(\dfrac{40 \text{ kips}}{50 \text{ kips}}\right)$ $= 38.7°$

Strength of Bolted Connection—Beam Web

The strength of the bolt group is determined by interpolating AISC *Manual* Table 7-7 for Angle = 30° and n = 5. Note that 30° is used conservatively in order to employ AISC *Manual* Table 7-7. A direct analysis can be performed to obtain an accurate value using the instantaneous center of rotation method.

$$e_x = a + 0.5s$$
$$= 9\tfrac{3}{4} \text{ in.} + 0.5(3 \text{ in.})$$
$$= 11.3 \text{ in.}$$

C = 3.53 by interpolation

From AISC *Manual* Table 7-1, the available shear strength per bolt for 1-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) is:

LRFD	ASD
$\phi r_n = 31.8$ kips/bolt	$\dfrac{r_n}{\Omega} = 21.2$ kips/bolt

The available bearing strength of the beam web is determined from AISC *Specification* Equation J3-6b. This equation is applicable in lieu of Equation J3-6a, because plowing of the bolts in the beam web is desirable to provide some flexibility in the connection:

$$\begin{aligned} r_n &= 3.0 d t_w F_u \\ &= 3.0(1 \text{ in.})(0.415 \text{ in.})(65 \text{ ksi}) \\ &= 80.9 \text{ kips/bolt} \end{aligned}$$ (*Spec.* Eq. J3-6b)

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(80.9 \text{ kips/bolt})$ $= 60.7$ kips/bolt	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{80.9 \text{ kips/bolt}}{2.00}$ $= 40.5$ kips/bolt

The available tearout strength of the beam web is determined from *Specification* Equation J3-6d. Similar to the bearing strength determination, this equation is used to allow plowing of the bolts in the beam web, and thus provide some flexibility in the connection.

Because the direction of load on the bolt is unknown, the minimum bolt edge distance is used to determine a worst case available tearout strength (including a ¼-in. tolerance to account for possible beam underrun). If a computer program is available, the true l_e can be calculated based on the instantaneous center of rotation.

$$\begin{aligned} l_c &= l_{eh} - 0.5 d_h \\ &= (1\tfrac{3}{4} \text{ in.} - \tfrac{1}{4} \text{ in.}) - 0.5(1\tfrac{1}{8} \text{ in.}) \\ &= 0.938 \text{ in.} \end{aligned}$$

$$\begin{aligned} r_n &= 1.5 l_c t_w F_u \\ &= 1.5(0.938 \text{ in.})(0.415 \text{ in.})(65 \text{ ksi}) \\ &= 38.0 \text{ kips/bolt} \end{aligned}$$ (*Spec.* Eq. J3-6d)

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(38.0 \text{ kips/bolt})$ $= 28.5$ kips/bolt	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{38.0 \text{ kips/bolt}}{2.00}$ $= 19.0$ kips/bolt

The tearout strength controls for bolts in the beam web.

The available strength of the bolted connection is determined using the minimum available strength calculated for bolt shear, bearing on the beam web and tearout on the beam web. From AISC *Manual* Equation 7-16, the bolt group eccentricity is accounted for by multiplying the minimum available bolt strength by the bolt coefficient C.

LRFD	ASD
$\phi R_n = C\phi r_n$ $= 3.53(28.5\text{ kips/bolt})$ $= 101\text{ kips} > 96.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = C\dfrac{r_n}{\Omega}$ $= 3.53(19.0\text{ kips/bolt})$ $= 67.1\text{ kips} > 64.0\text{ kips}$ **o.k.**

Strength of Bolted Connection—Plate

Note that bolt bearing on the beam web controls over bearing on the plate because the beam web is thinner than the plate; therefore, this limit state will not control.

As was discussed for the beam web, the available tearout strength of the plate is determined from *Specification* Equation J3-6d. The bolt edge distance in the vertical direction controls for this design.

$$\begin{aligned} l_c &= l_{ev} - 0.5d_h \\ &= 1\tfrac{1}{4}\text{ in.} - 0.5(1\tfrac{1}{8}\text{ in.}) \\ &= 0.688\text{ in.} \end{aligned}$$

$$\begin{aligned} r_n &= 1.5 l_c t F_u \\ &= 1.5(0.688\text{ in.})(\tfrac{3}{4}\text{ in.})(65\text{ ksi}) \\ &= 50.3\text{ kips/bolt} \end{aligned}$$ (*Spec.* Eq. J3-6d)

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(50.3\text{ kips/bolt})$ $= 37.7\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{50.3\text{ kips/bolt}}{2.00}$ $= 25.2\text{ kips/bolt}$

Therefore, the available strength of the bolted connection at the beam web, as determined previously, controls.

Shear Yielding Strength of Beam

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the beam is determined as follows:

$$\begin{aligned} A_{gv} &= dt_w \\ &= (18.2\text{ in.})(0.415\text{ in.}) \\ &= 7.55\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60F_yA_{gv} \\ &= 0.60(50\text{ ksi})(7.55\text{ in.}^2) \\ &= 227\text{ kips} \end{aligned}$$ (*Spec.* Eq. J4-3)

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(227\text{ kips})$ $= 227\text{ kips} > 75\text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{227\text{ kips}}{1.50}$ $= 151\text{ kips} > 50\text{ kips}$ **o.k.**

Tensile Yielding Strength of Beam

From AISC *Specification* Section J4.1(a), the available tensile yielding strength of the beam web is determined as follows:

$$R_n = F_y A_g \qquad \text{(Spec. Eq. J4-1)}$$
$$= (50 \text{ ksi})(17.6 \text{ in.}^2)$$
$$= 880 \text{ kips}$$

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(880 \text{ kips})$ $= 792 \text{ kips} > 60 \text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{880 \text{ kips}}{1.67}$ $= 527 \text{ kips} > 40 \text{ kips}$ **o.k.**

Tensile Rupture Strength of Beam

From AISC *Specification* Section J4.1, determine the available tensile rupture strength of the beam. The effective net area is $A_e = A_n U$, where U is determined from AISC *Specification* Table D3.1, Case 2.

$$\bar{x} = \frac{2b_f^2 t_f + t_w^2 (d - 2t_f)}{8b_f t_f + 4t_w (d - 2t_f)}$$
$$= \frac{2(7.56 \text{ in.})^2 (0.695 \text{ in.}) + (0.415 \text{ in.})^2 [18.2 \text{ in.} - 2(0.695 \text{ in.})]}{8(7.56 \text{ in.})(0.695 \text{ in.}) + 4(0.415 \text{ in.})[18.2 \text{ in.} - 2(0.695 \text{ in.})]}$$
$$= 1.18 \text{ in.}$$

$$U = 1 - \frac{\bar{x}}{l}$$
$$= 1 - \frac{1.18 \text{ in.}}{3.00 \text{ in.}}$$
$$= 0.607$$

$$A_n = A_g - n(d_h + \tfrac{1}{16} \text{ in.}) t_w$$
$$= 17.6 \text{ in.}^2 - 5(1\tfrac{1}{8} \text{ in.} + \tfrac{1}{16} \text{ in.})(0.415 \text{ in.})$$
$$= 15.1 \text{ in.}^2$$

$$R_n = F_u A_e \qquad \text{(Spec. Eq. J4-2)}$$
$$= F_u A_n U$$
$$= (65 \text{ ksi})(15.1 \text{ in.}^2)(0.607)$$
$$= 596 \text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(596 \text{ kips})$ $= 447 \text{ kips} > 60 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{596 \text{ kips}}{2.00}$ $= 298 \text{ kips} > 40 \text{ kips}$ **o.k.**

Block Shear Rupture of Beam Web

Block shear rupture is only applicable in the direction of the axial load because the beam is uncoped and the limit state is not applicable for an uncoped beam subject to vertical shear. Assuming a U-shaped tearout relative to the axial load, and assuming a horizontal edge distance of l_{eh} = 1¾ in. − ¼ in. = 1½ in. to account for a possible beam underrun of ¼ in., the block shear rupture strength is:

$$R_n = 0.60 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.60 F_y A_{gv} + U_{bs} F_u A_{nt} \qquad \text{(}\textit{Spec.}\text{ Eq. J4-5)}$$

where

$$A_{gv} = (2 \text{ shear planes})(s + l_{eh})t_w$$
$$= (2 \text{ shear planes})(3 \text{ in.} + 1\tfrac{1}{2} \text{ in.})(0.415 \text{ in.})$$
$$= 3.74 \text{ in.}^2$$

$$A_{nv} = A_{gv} - (2 \text{ shear planes})(1.5)(d_h + \tfrac{1}{16} \text{ in.})t_w$$
$$= 3.74 \text{ in.}^2 - (2 \text{ shear planes})(1.5)(1\tfrac{1}{8} \text{ in.} + \tfrac{1}{16} \text{ in.})(0.415 \text{ in.})$$
$$= 2.26 \text{ in.}^2$$

$$A_{nt} = \left[12.0 \text{ in.} - (n-1)(d_h + \tfrac{1}{16} \text{ in.})\right]t_w$$
$$= \left[12.0 \text{ in.} - (5-1)(1\tfrac{1}{8} \text{ in.} + \tfrac{1}{16} \text{ in.})\right](0.415 \text{ in.})$$
$$= 3.01 \text{ in.}^2$$

$$U_{bs} = 1.0$$

and

$$R_n = 0.60(65 \text{ ksi})(2.26 \text{ in.}^2) + 1.0(65 \text{ ksi})(3.01 \text{ in.}^2) \leq 0.60(50 \text{ ksi})(3.74 \text{ in.}^2) + 1.0(65 \text{ ksi})(3.01 \text{ in.}^2)$$
$$= 284 \text{ kips} < 308 \text{ kips}$$

Therefore:

$$R_n = 284 \text{ kips}$$

From AISC *Specification* Section J4.3, the available strength for the limit state of block shear rupture of the beam web is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(284 \text{ kips})$ $= 213 \text{ kips} > 60 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{284 \text{ kips}}{2.00}$ $= 142 \text{ kips} > 40 \text{ kips}$ **o.k.**

Maximum Plate Thickness

Determine the maximum plate thickness, t_{max}, that will result in the plate yielding before the bolts shear. From AISC *Specification* Table J3.2:

$F_{nv} = 54$ ksi

From AISC *Manual* Table 7-7 for two column of bolts, Angle = 0°, s = 3 in., and n = 5:

$C' = 38.7$ in.

$$M_{max} = \frac{F_{nv}}{0.90}(A_b C') \qquad \text{(Manual Eq. 10-4)}$$
$$= \frac{54\text{ ksi}}{0.90}\left(0.785\text{ in.}^2\right)\left(38.7\text{ in.}\right)$$
$$= 1{,}820\text{ kip-in.}$$

$$t_{max} = \frac{6M_{max}}{F_y l^2} \qquad \text{(Manual Eq. 10-3)}$$
$$= \frac{6\left(1{,}820\text{ kip-in.}\right)}{\left(50\text{ ksi}\right)\left(14\tfrac{1}{2}\text{ in.}\right)^2}$$
$$= 1.04\text{ in.} > \tfrac{3}{4}\text{ in.} \quad \textbf{o.k.}$$

Flexure Strength of Plate

The required flexural strength of the plate is determined as follows:

LRFD	ASD
$M_u = V_u a$ $= (75\text{ kips})(9\tfrac{3}{4}\text{ in.})$ $= 731$ kip-in.	$M_a = V_a a$ $= (50\text{ kips})(9\tfrac{3}{4}\text{ in.})$ $= 488$ kip-in.

The plate is checked for the limit state of buckling using the double-coped beam procedure as given in AISC *Manual* Part 9, where the unbraced length for lateral-torsional buckling, L_b, is taken as the distance from the first column of bolts to the supporting column web and the top cope dimension, d_{ct}, is conservatively taken as the distance from the top of the beam to the first row of bolts.

$$C_b = \left[3 + \ln\left(\frac{L_b}{d}\right)\right]\left(1 - \frac{d_{ct}}{d}\right) \geq 1.84$$
$$= \left[3 + \ln\left(\frac{9\tfrac{3}{4}\text{ in.}}{14\tfrac{1}{2}\text{ in.}}\right)\right]\left(1 - \frac{3\tfrac{1}{8}\text{ in.}}{14\tfrac{1}{2}\text{ in.}}\right) \geq 1.84$$
$$= 2.04 > 1.84$$

Therefore:

$C_b = 2.04$

The available flexural strength of the plate is determined using AISC *Specification* Section F11 as follows:

For yielding of the plate:

$$M_n = M_p = F_y Z \le 1.6 F_y S_x \qquad (\textit{Spec}\text{. Eq. F11-1})$$

$$= (50\text{ ksi})\left[\frac{(\tfrac{3}{4}\text{ in.})(14\tfrac{1}{2}\text{ in.})^2}{4}\right] \le 1.6(50\text{ ksi})\left[\frac{(\tfrac{3}{4}\text{ in.})(14\tfrac{1}{2}\text{ in.})^2}{6}\right]$$

$$= 1{,}970\text{ kip-in.} < 2{,}100\text{ kip-in.}$$

$$= 1{,}970\text{ kip-in.}$$

For lateral-torsional buckling of the plate:

$$\frac{L_b d}{t^2} = \frac{(9\tfrac{3}{4}\text{ in.})(14\tfrac{1}{2}\text{ in.})}{(\tfrac{3}{4}\text{ in.})^2}$$

$$= 251$$

$$\frac{0.08E}{F_y} = \frac{0.08(29{,}000\text{ ksi})}{50\text{ ksi}}$$

$$= 46.4$$

$$\frac{1.9E}{F_y} = \frac{1.9(29{,}000\text{ ksi})}{50\text{ ksi}}$$

$$= 1{,}100$$

Because $\frac{0.08E}{F_y} < \frac{L_b d}{t^2} \le \frac{1.9E}{F_y}$, use AISC *Specification* Section F11.2(b):

$$M_y = F_y S_x$$

$$= (50\text{ ksi})\left[\frac{(\tfrac{3}{4}\text{ in.})(14\tfrac{1}{2}\text{ in.})^2}{6}\right]$$

$$= 1{,}310\text{ kip-in.}$$

$$M_n = C_b\left[1.52 - 0.274\left(\frac{L_b d}{t^2}\right)\frac{F_y}{E}\right]M_y \le M_p \qquad (\textit{Spec}\text{. Eq. F11-2})$$

$$= 2.04\left[1.52 - 0.274(251)\left(\frac{50\text{ ksi}}{29{,}000\text{ ksi}}\right)\right](1{,}310\text{ kip-in.}) \le 1{,}970\text{ kip-in.}$$

$$= 3{,}750\text{ kip-in.} > 1{,}970\text{ kip-in.}$$

Therefore:

$$M_n = 1{,}970\text{ kip-in.}$$

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(1{,}970 \text{ kip-in.})$ $= 1{,}770 \text{ kip-in.} > 731 \text{ kip-in.}$ **o.k.**	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{1{,}970 \text{ kip-in.}}{1.67}$ $= 1{,}180 \text{ kip-in.} > 488 \text{ kip-in.}$ **o.k.**

Shear Yielding Strength of Plate

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the plate is determined as follows:

$$\begin{aligned} A_{gv} &= lt \\ &= (14\tfrac{1}{2} \text{ in.})(\tfrac{3}{4} \text{ in.}) \\ &= 10.9 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_{nv} &= 0.60 F_y A_{gv} \\ &= 0.60(50 \text{ ksi})(10.9 \text{ in.}^2) \\ &= 327 \text{ kips} \end{aligned}$$ (*Spec.* Eq. J4-3)

LRFD	ASD
$\phi = 1.00$ $\phi R_{nv} = 1.00(327 \text{ kips})$ $= 327 \text{ kips} > 75 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_{nv}}{\Omega} = \dfrac{327 \text{ kips}}{1.50}$ $= 218 \text{ kips} > 50 \text{ kips}$ **o.k.**

Tension Yielding Strength of Plate

From AISC *Specification* Section J4.1(a), the available tensile yielding strength of the plate is determined as follows:

$$\begin{aligned} A_g &= lt \\ &= (14\tfrac{1}{2} \text{ in.})(\tfrac{3}{4} \text{ in.}) \\ &= 10.9 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_{np} &= F_y A_g \\ &= (50 \text{ ksi})(10.9 \text{ in.}) \\ &= 545 \text{ kips} \end{aligned}$$ (from *Spec.* Eq. J4-1)

LRFD	ASD
$\phi = 0.90$ $\phi R_{np} = 0.90(545 \text{ kips})$ $= 491 \text{ kips} > 60 \text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_{np}}{\Omega} = \dfrac{545 \text{ kips}}{1.67}$ $= 326 \text{ kips} > 40 \text{ kips}$ **o.k.**

Interaction of Axial, Flexure and Shear Yielding in Plate

AISC *Specification* Chapter H does not address combined flexure and shear. The method employed here is derived from Chapter H in conjunction with AISC *Manual* Equation 10-5, as follows:

LRFD	ASD
$\dfrac{N_u}{\phi R_{np}} = \dfrac{60 \text{ kips}}{491 \text{ kips}} = 0.122$ Because $\dfrac{N_u}{\phi R_{np}} < 0.2$: $\left(\dfrac{N_u}{2\phi R_{np}} + \dfrac{V_u a}{\phi M_n}\right)^2 + \left(\dfrac{V_u}{\phi R_{nv}}\right)^2 \le 1$ $= \left[\dfrac{60 \text{ kips}}{2(491 \text{ kips})} + \dfrac{(75 \text{ kips})(9\tfrac{3}{4} \text{ in.})}{1{,}770 \text{ kip-in.}}\right]^2 + \left(\dfrac{75 \text{ kips}}{327 \text{ kips}}\right)^2 \le 1$ $= 0.278 < 1$ **o.k.**	$\dfrac{\Omega N_a}{R_{np}} = \dfrac{40 \text{ kips}}{326 \text{ kips}} = 0.123$ Because $\dfrac{\Omega N_a}{R_{np}} < 0.2$: $\left(\dfrac{\Omega N_a}{2 R_{np}} + \dfrac{\Omega V_a a}{M_n}\right)^2 + \left(\dfrac{\Omega V_a}{R_{nv}}\right)^2 \le 1$ $= \left[\dfrac{40 \text{ kips}}{2(326 \text{ kips})} + \dfrac{(50 \text{ kips})(9\tfrac{3}{4} \text{ in.})}{1{,}180 \text{ kip-in.}}\right]^2 + \left(\dfrac{50 \text{ kips}}{218 \text{ kips}}\right)^2 \le 1$ $= 0.278 < 1$ **o.k.**

Tensile Rupture Strength of Plate

From AISC *Specification* Section J4.1(b), the available tensile rupture strength of the plate is determined as follows:

$$\begin{aligned} A_n &= \left[l - n\left(d_h + \tfrac{1}{16} \text{ in.}\right)\right] t \\ &= \left[14\tfrac{1}{2} \text{ in.} - (5 \text{ bolts})\left(1\tfrac{1}{8} \text{ in.} + \tfrac{1}{16} \text{ in.}\right)\right]\left(\tfrac{3}{4} \text{ in.}\right) \\ &= 6.42 \text{ in.}^2 \end{aligned}$$

AISC *Specification* Table D3.1, Case 1, applies in this case because the tension load is transmitted directly to the cross-sectional element by fasteners; therefore, $U = 1.0$.

$$\begin{aligned} A_e &= A_n U \\ &= \left(6.42 \text{ in.}^2\right)(1.0) \\ &= 6.42 \text{ in.}^2 \end{aligned}$$ (*Spec.* Eq. D3-1)

$$\begin{aligned} R_{np} &= F_u A_e \\ &= (65 \text{ ksi})\left(6.42 \text{ in.}^2\right) \\ &= 417 \text{ kips} \end{aligned}$$ (*Spec.* Eq. J4-2)

LRFD	ASD
$\phi = 0.75$ $\phi R_{np} = 0.75(417 \text{ kips}) = 313 \text{ kips} > 60 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_{np}}{\Omega} = \dfrac{417 \text{ kips}}{2.00} = 209 \text{ kips} > 40 \text{ kips}$ **o.k.**

Flexural Rupture of the Plate

The available flexural rupture strength of the plate is determined as follows:

$$Z_{net} = \frac{tl^2}{4} - \frac{t}{4}\left[\left(d_h + \tfrac{1}{16}\text{ in.}\right)(s)\left(n^2 - 1\right) + \left(d_h + \tfrac{1}{16}\text{ in.}\right)^2\right]$$

$$= \frac{\left(\tfrac{3}{4}\text{ in.}\right)\left(14\tfrac{1}{2}\text{ in.}\right)^2}{4} - \left(\frac{\tfrac{3}{4}\text{ in.}}{4}\right)\left\{\left(1\tfrac{1}{8}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)(3\text{ in.})\left[(5)^2 - 1\right] + \left(1\tfrac{1}{8}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)^2\right\}$$

$$= 23.1\text{ in.}^3$$

$$M_n = F_u Z_{net} \qquad \text{(\textit{Manual} Eq. 9-4)}$$

$$= (65\text{ ksi})\left(23.1\text{ in.}^3\right)$$

$$= 1{,}500\text{ kip-in.}$$

LRFD	ASD
$\phi = 0.75$ $\phi M_n = 0.75(1{,}500\text{ kip-in.})$ $= 1{,}130\text{ kip-in.} > 731\text{ kip-in.}$ **o.k.**	$\Omega = 2.00$ $\dfrac{M_n}{\Omega} = \dfrac{1{,}500\text{ kip-in.}}{2.00}$ $= 750\text{ kip-in.} > 488\text{ kip-in.}$ **o.k.**

Shear Rupture Strength of Plate

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the plate is determined as follows:

$$A_{nv} = \left[l - n\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t_p$$

$$= \left[14\tfrac{1}{2}\text{ in.} - 5\left(1\tfrac{1}{8}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right]\left(\tfrac{3}{4}\text{ in.}\right)$$

$$= 6.42\text{ in.}^2$$

$$R_{nv} = 0.60 F_u A_{nv} \qquad \text{(\textit{Spec.} Eq. J4-4)}$$

$$= 0.60(65\text{ ksi})\left(6.42\text{ in.}^2\right)$$

$$= 250\text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_{nv} = 0.75(250\text{ kips})$ $= 188\text{ kips} > 75\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_{nv}}{\Omega} = \dfrac{250\text{ kips}}{2.00}$ $= 125\text{ kips} > 50\text{ kips}$ **o.k.**

Interaction of Axial, Flexure and Shear Rupture in Plate

AISC *Specification* Chapter H does not address combined flexure and shear. The method employed here is derived from Chapter H in conjunction with AISC *Manual* Equation 10-5, as follows:

LRFD	ASD
$\dfrac{N_u}{\phi R_{np}} = \dfrac{60\text{ kips}}{313\text{ kips}}$ $= 0.192$	$\dfrac{\Omega N_a}{R_{np}} = \dfrac{40\text{ kips}}{209\text{ kips}}$ $= 0.191$

LRFD	ASD
Because $\dfrac{N_u}{\phi R_{np}} < 0.2$:	Because $\dfrac{\Omega N_a}{R_{np}} < 0.2$:
$\left(\dfrac{N_u}{2\phi R_{np}} + \dfrac{V_u a}{\phi M_n}\right)^2 + \left(\dfrac{V_u}{\phi R_{nv}}\right)^2 \le 1$	$\left(\dfrac{\Omega N_a}{2R_{np}} + \dfrac{\Omega V_a a}{M_n}\right)^2 + \dfrac{\Omega V_a}{R_{nv}} \le 1$
$\left[\dfrac{60\text{ kips}}{2(313\text{ kips})} + \dfrac{(75\text{ kips})(9\tfrac{3}{4}\text{ in.})}{1{,}130\text{ kip-in.}}\right]^2 + \left(\dfrac{75\text{ kips}}{188\text{ kips}}\right)^2 \le 1$	$\left[\dfrac{40\text{ kips}}{2(209\text{ kips})} + \dfrac{(50\text{ kips})(9\tfrac{3}{4}\text{ in.})}{750\text{ kip-in.}}\right]^2 + \left(\dfrac{50\text{ kips}}{125\text{ kips}}\right)^2 \le 1$
$0.711 < 1$ **o.k.**	$0.716 < 1$ **o.k.**

Block Shear Rupture Strength of Plate—Beam Shear Direction

The nominal strength for the limit state of block shear rupture of the plate, assuming an L-shaped tearout due to the shear load only as shown in Figure II.A-19B-2(a), is determined as follows:

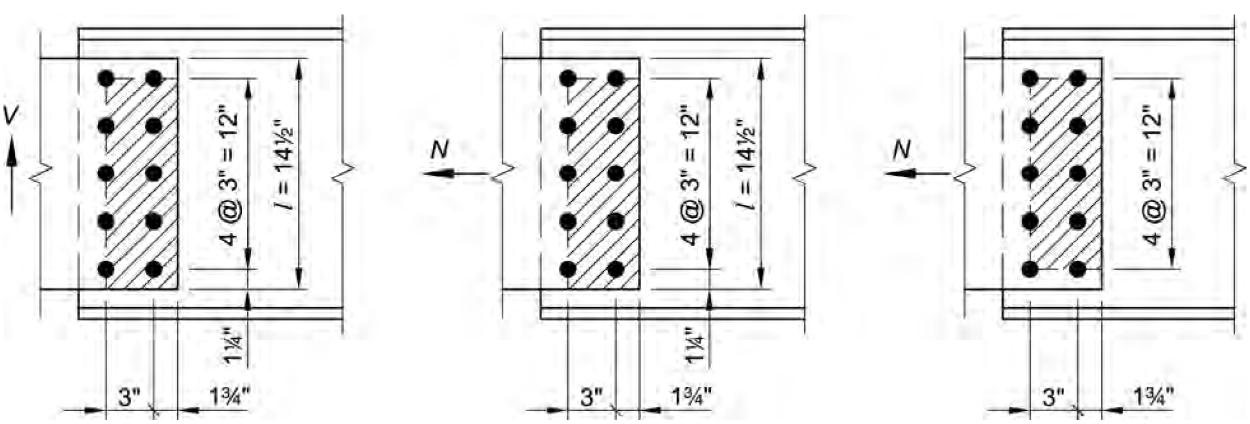
$$R_{bsv} = 0.60F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.60F_y A_{gv} + U_{bs} F_u A_{nt} \qquad (\textit{Spec.}\text{ Eq. J4-5})$$

where

$$\begin{aligned} A_{gv} &= (l - l_{ev})t \\ &= (14\tfrac{1}{2}\text{ in.} - 1\tfrac{1}{4}\text{ in.})(\tfrac{3}{4}\text{ in.}) \\ &= 9.94\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - (n_v - 0.5)(d_h + \tfrac{1}{16}\text{ in.})t \\ &= 9.94\text{ in.}^2 - (5 - 0.5)(1\tfrac{1}{8}\text{ in.} + \tfrac{1}{16}\text{ in.})(\tfrac{3}{4}\text{ in.}) \\ &= 5.93\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= \left[l_{eh} + (n_h - 1)s - (n_h - 0.5)(d_h + \tfrac{1}{16}\text{ in.})\right]t \\ &= \left[1\tfrac{3}{4}\text{ in.} + (2-1)(3\text{ in.}) - (2-0.5)(1\tfrac{1}{8}\text{ in.} + \tfrac{1}{16}\text{ in.})\right](\tfrac{3}{4}\text{ in.}) \\ &= 2.23\text{ in.}^2 \end{aligned}$$



(a) Beam shear direction *(b) Beam axial direction—L-shaped* *(c) Beam axial direction—U-shaped*

Fig. II.A-19B-2. Block shear rupture of plate.

Because stress is not uniform along the net tensile area, $U_{bs} = 0.5$.

$$R_{bsv} = 0.60(65\text{ ksi})(5.93\text{ in.}^2) + 0.5(65\text{ ksi})(2.23\text{ in.}^2) \le 0.60(50\text{ ksi})(9.94\text{ in.}^2) + 0.5(65\text{ ksi})(2.23\text{ in.}^2)$$
$$= 304\text{ kips} < 371\text{ kips}$$

Therefore:

$$R_{bsv} = 304\text{ kips}$$

From AISC *Specification* Section J4.3, the available strength for the limit state of block shear rupture on the plate is:

LRFD	ASD
$\phi = 0.75$ $\phi R_{bsv} = 0.75(304\text{ kips})$ $= 228\text{ kips} > 75\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_{bsv}}{\Omega} = \dfrac{304\text{ kips}}{2.00}$ $= 152\text{ kips} > 50\text{ kips}$ **o.k.**

Block Shear Rupture Strength of the Plate—Beam Axial Direction

The plate block shear rupture failure path due to axial load only could occur as an L- or U-shape. Assuming an L-shaped failure path due to axial load only, as shown in Figure II.A-19B-2(b), the available block shear rupture strength of the plate is:

$$R_{bsn} = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \quad \text{(Spec. Eq. J4-5)}$$

where

$$A_{gv} = \left[(n_h - 1)s + l_{eh}\right]t$$
$$= \left[(2-1)(3\text{ in.}) + 1\tfrac{3}{4}\text{ in.}\right](\tfrac{3}{4}\text{ in.})$$
$$= 3.56\text{ in.}^2$$

$$A_{nv} = A_{gv} - (n_h - 0.5)(d_h + \tfrac{1}{16}\text{ in.})t$$
$$= 3.56\text{ in.}^2 - (2-0.5)(1\tfrac{1}{8}\text{ in.} + \tfrac{1}{16}\text{ in})(\tfrac{3}{4}\text{ in.})$$
$$= 2.22\text{ in.}^2$$

$$A_{nt} = \left[l_{ev} + (n_v - 1)s - (n_v - 0.5)(d_h + \tfrac{1}{16}\text{ in.})\right]t$$
$$= \left[1\tfrac{1}{4}\text{ in.} + (5-1)(3\text{ in.}) - (5-0.5)(1\tfrac{1}{8}\text{ in.} + \tfrac{1}{16}\text{ in})\right](\tfrac{3}{4}\text{ in.})$$
$$= 5.93\text{ in.}^2$$

$$U_{bs} = 1.0$$

and

$$R_{bsn} = 0.60(65\text{ ksi})(2.22\text{ in.}^2) + 1.0(65\text{ ksi})(5.93\text{ in.}^2) \le 0.60(50\text{ ksi})(3.56\text{ in.}^2) + 1.0(65\text{ ksi})(5.93\text{ in.}^2)$$
$$= 472\text{ kips} < 492\text{ kips}$$

Therefore:

$R_{bsn} = 472 \text{ kips}$

From AISC *Specification* Section J4.3, the available strength for the limit state of block shear rupture on the plate is:

LRFD	ASD
$\phi = 0.75$ $\phi R_{bsn} = 0.75(472 \text{ kips})$ $= 354 \text{ kips} > 60 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\frac{R_{bsn}}{\Omega} = \frac{472 \text{ kips}}{2.00}$ $= 236 \text{ kips} > 40 \text{ kips}$ **o.k.**

Assuming a U-shaped failure path in the plate due to axial load, as shown in Figure II.A-19B-2(c), the available block shear rupture strength of the plate is:

$$R_{bsn} = 0.60F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.60F_y A_{gv} + U_{bs} F_u A_{nt}$$ (*Spec.* Eq. J4-5)

where

$$\begin{aligned} A_{gv} &= (2 \text{ shear planes})\left[l_{eh} + (n_h - 1)s\right]t \\ &= (2 \text{ shear planes})\left[1\tfrac{3}{4} \text{ in.} + (2-1)(3 \text{ in.})\right](\tfrac{3}{4} \text{ in.}) \\ &= 7.13 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - (2 \text{ shear planes})(n_h - 0.5)(d_h + \tfrac{1}{16} \text{ in.})t \\ &= 7.13 \text{ in.}^2 - (2 \text{ shear planes})(2 - 0.5)(1\tfrac{1}{8} \text{ in.} + \tfrac{1}{16} \text{ in.})(\tfrac{3}{4} \text{ in.}) \\ &= 4.46 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= \left[(n_v - 1)s - (n_v - 1)(d_h + \tfrac{1}{16} \text{ in.})\right]t \\ &= \left[(5-1)(3 \text{ in.}) - (5-1)(1\tfrac{1}{8} \text{ in.} + \tfrac{1}{16} \text{ in.})\right](\tfrac{3}{4} \text{ in.}) \\ &= 5.44 \text{ in.}^2 \end{aligned}$$

$$U_{bs} = 1.0$$

and

$$\begin{aligned} R_{bsn} &= 0.60(65 \text{ ksi})(4.46 \text{ in.}^2) + 1.0(65 \text{ ksi})(5.44 \text{ in.}^2) \le 0.60(50 \text{ ksi})(7.13 \text{ in.}^2) + 1.0(65 \text{ ksi})(5.44 \text{ in.}^2) \\ &= 528 \text{ kips} < 568 \text{ kips} \end{aligned}$$

Therefore:

$R_{bsn} = 528 \text{ kips}$

From AISC *Specification* Section J4.3, the available strength for the limit state of block shear rupture on the plate is:

LRFD	ASD
$\phi = 0.75$ $\phi R_{bsn} = 0.75(528 \text{ kips})$ $= 396 \text{ kips} > 60 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\frac{R_{bsn}}{\Omega} = \frac{528 \text{ kips}}{2.00}$ $= 264 \text{ kips} > 40 \text{ kips}$ **o.k.**

Block Shear Rupture Strength of Plate—Combined Shear and Axial Interaction

The same L-shaped block shear rupture failure path is loaded by forces in both the shear and axial directions. The interaction of loading in both directions is determined as follows:

LRFD	ASD
$\left(\dfrac{V_u}{\phi R_{bsv}}\right)^2+\left(\dfrac{N_u}{\phi R_{bsn}}\right)^2\le 1$ $\left(\dfrac{75\text{ kips}}{228\text{ kips}}\right)^2+\left(\dfrac{60\text{ kips}}{354\text{ kips}}\right)^2=0.137<1$ **o.k.**	$\left(\dfrac{\Omega V_a}{R_{bsv}}\right)^2+\left(\dfrac{\Omega N_a}{R_{bsn}}\right)^2\le 1$ $\left(\dfrac{50\text{ kips}}{152\text{ kips}}\right)^2+\left(\dfrac{40\text{ kips}}{236\text{ kips}}\right)^2=0.137<1$ **o.k.**

Shear Rupture Strength of Column Web at Weld

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the column web is determined as follows:

$$\begin{aligned}A_{nv}&=2lt_w\\&=2\left(14\tfrac{1}{2}\text{ in.}\right)\left(0.440\text{ in.}\right)\\&=12.8\text{ in.}^2\end{aligned}$$

$$\begin{aligned}R_n&=0.60F_uA_v\\&=0.60\left(65\text{ ksi}\right)\left(12.8\text{ in.}^2\right)\\&=499\text{ kips}\end{aligned}$$ (*Spec.* Eq. J4-4)

LRFD	ASD
$\phi=0.75$ $\phi R_n=0.75\left(499\text{ kips}\right)$ $=374\text{ kips}>75\text{ kips}$ **o.k.**	$\Omega=2.00$ $\dfrac{R_n}{\Omega}=\dfrac{499\text{ kips}}{2.00}$ $=250\text{ kips}>50\text{ kips}$ **o.k.**

Yield Line Analysis on Supporting Column Web

A yield line analysis is used to determine the strength of the column web in the direction of the axial tension load. The yield line and associated dimensions are shown in Figure II.A-19B-3 and the available strength is determined as follows:

$$\begin{aligned}T&=d-2k_{des}\\&=14.0\text{ in.}-2\left(1.31\text{ in.}\right)\\&=11.4\text{ in.}\end{aligned}$$

$$\begin{aligned}a&=\frac{d}{2}-k_{des}+\frac{t_w}{2}\\&=\frac{14.0\text{ in.}}{2}-1.31\text{ in.}+\frac{0.415\text{ in.}}{2}\\&=5.90\text{ in.}\end{aligned}$$

$$b = \frac{d}{2} - k_{des} - \frac{t_w}{2} - t_p$$
$$= \frac{14.0 \text{ in.}}{2} - 1.31 \text{ in.} - \frac{0.415 \text{ in.}}{2} - \tfrac{3}{4} \text{ in.}$$
$$= 4.73 \text{ in.}$$

$$c = t_p$$
$$= \tfrac{3}{4} \text{ in.}$$

$$R_n = \frac{t_w^{\,2} F_y}{4} \left[\frac{4\sqrt{2Tab(a+b)} + l(a+b)}{ab} \right] \qquad \text{(\textit{Manual} Eq. 9-31)}$$
$$= \frac{(0.440 \text{ in.})^2 (50 \text{ ksi})}{4} \left[\frac{4\sqrt{2(11.4 \text{ in.})(5.90 \text{ in.})(4.73 \text{ in.})(5.90 \text{ in.} + 4.73 \text{ in.})} + (14\tfrac{1}{2} \text{ in.})(5.90 \text{ in.} + 4.73 \text{ in.})}{(5.90 \text{ in.})(4.73 \text{ in.})} \right]$$
$$= 41.9 \text{ kips}$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(41.9 \text{ kips})$ $= 41.9 \text{ kips} < 60 \text{ kips}$ **n.g.**	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{41.9 \text{ kips}}{1.50}$ $= 27.9 \text{ kips} < 40 \text{ kips}$ **n.g.**

The available column web strength is not adequate to resist the axial force in the beam. The column may be increased in size for an adequate web thickness or reinforced with stiffeners or web doubler plates. For example, a W14×120 column, with t_w = 0.590 in., has adequate strength to resist the given forces.

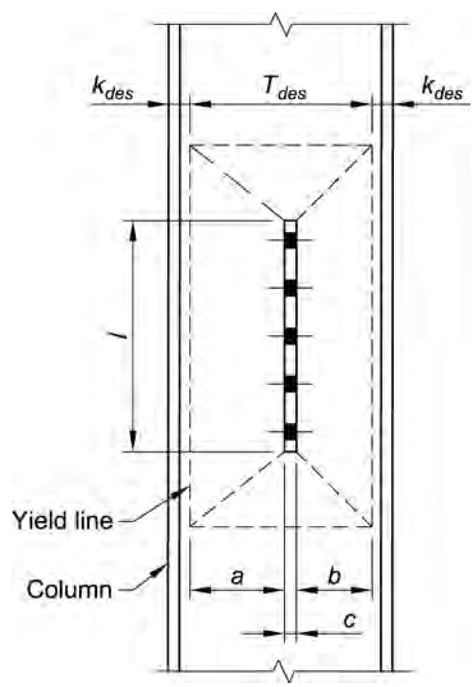


Fig II.A-19B-3. Yield line for column web.

Strength of Weld

A two-sided fillet weld with size of (⅝)t_p = 0.469 in. (use ½-in. fillet welds) is used. As discussed in AISC *Manual* Part 10, this weld size will develop the strength of the shear plate used because the moment generated by this connection is indeterminate.

The available weld strength is determined using AISC *Manual* Equation 8-2a or 8-2b, incorporating the directional strength increase from AISC *Specification* Equation J2-5, as follows:

$$\mu = 1.0 + 0.50\sin^{1.5}\theta$$
$$= 1.0 + 0.50\sin^{1.5}(38.7°)$$
$$= 1.25$$

LRFD	ASD
$R_n = (1.392\text{ kip/in.})Dl\mu(2\text{ sides})$ $= (1.392\text{ kip/in.})(8)(14½\text{ in.})(1.25)(2\text{ sides})$ $= 404\text{ kips} > 96.0\text{ kips}$ **o.k.**	$R_n = (0.928\text{ kip/in.})Dl\mu(2\text{ sides})$ $= (0.928\text{ kip/in.})(8)(14½\text{ in.})(1.25)(2\text{ sides})$ $= 269\text{ kips} > 64.0\text{ kips}$ **o.k.**

Conclusion

The configuration given does not work due to the inadequate column web. The column would need to be increased in size or reinforced as discussed previously.

Comments: If the applied axial load were in compression, the connection plate would need to be checked for compressive flexural buckling strength as follows. This is required in the case of the extended configuration of a single-plate connection and would not be required for the conventional configuration.

From AISC *Specification* Table C-A-7.1, Case c:

$$K = 1.2$$

$$\frac{L_c}{r} = \frac{KL}{r}$$
$$= \frac{1.2(9¾\text{ in.})}{¾\text{ in.}/\sqrt{12}}$$
$$= 54.0$$

As stated in AISC *Specification* Section J4.4, if L_c/r is greater than 25, Chapter E applies. The available critical stress of the plate, ϕF_{cr} or F_{cr}/Ω, is determined using AISC *Manual* Table 4-14 as follows:

LRFD	ASD
$\phi F_{cr} = 36.4\text{ ksi}$ $\phi R_n = \phi F_{cr} l t_p$ $= (36.4\text{ ksi})(14½\text{ in.})(¾\text{ in.})$ $= 396\text{ kips} > 60\text{ kips}$ **o.k.**	$\frac{F_{cr}}{\Omega} = 24.2\text{ ksi}$ $\frac{R_n}{\Omega} = \frac{F_{cr}}{\Omega} l t_p$ $= (24.2\text{ ksi})(14½\text{ in.})(¾\text{ in.})$ $= 263\text{ kips} > 40\text{ kips}$ **o.k.**

Column Reinforcement

As mentioned there are three options to correct the column web failure. These options are as follows:

1) Use a heavier column. This may not be practical because the steel may have been purchased and perhaps detailed and fabricated before the problem is found.

2) Use a web doubler plate. This plate would be fitted about the shear plate on the same side of the column web as the shear plate. This necessitates a lot of cutting, fitting and welding, and is therefore expensive.

3) Use stiffener or stabilizer plates—also called continuity plates. This is probably the most viable option, but changes the nature of the connection, because the stiffener plates will cause the column to be subjected to a moment. This cannot be avoided, but may be used advantageously.

Option 3 Solution

Because the added stiffeners cause the column to pick-up moment, the moment for which the connection is designed can be reduced.

The connection is designed as a conventional configuration shear plate with axial force for everything to the right of Section A-A as shown in Figure II.A-19B-4. The design to the left of Section A-A is performed following a procedure for Type II stabilizer plates presented in Fortney and Thornton (2016).

As shown in Figure II.A-19B-5, the moment in the shear plate to the left of Section A-A is uncoupled between the stabilizer plates.

$$V_s = \frac{Va'}{l}$$

where

$a' = 7$ in.

$l = 14\frac{1}{2}$ in.

$g = 2\frac{3}{4}$ in.

LRFD	ASD
$V_{us} = \frac{V_u a'}{l}$ $= \frac{(75 \text{ kips})(7 \text{ in.})}{14\frac{1}{2} \text{ in.}}$ $= 36.2$ kips	$V_{as} = \frac{V_a a'}{L}$ $= \frac{(50 \text{ kips})(7 \text{ in.})}{14\frac{1}{2} \text{ in.}}$ $= 24.1$ kips

The force between the shear plate and stabilizer plate is determined as follows:

LRFD	ASD
$F_{up} = V_{us} + \frac{N_u}{2}$ $= 36.2 \text{ kips} + \frac{60 \text{ kips}}{2}$ $= 66.2$ kips	$F_{ap} = V_{as} + \frac{N_a}{2}$ $= 24.1 \text{ kips} + \frac{40 \text{ kips}}{2}$ $= 44.1$ kips

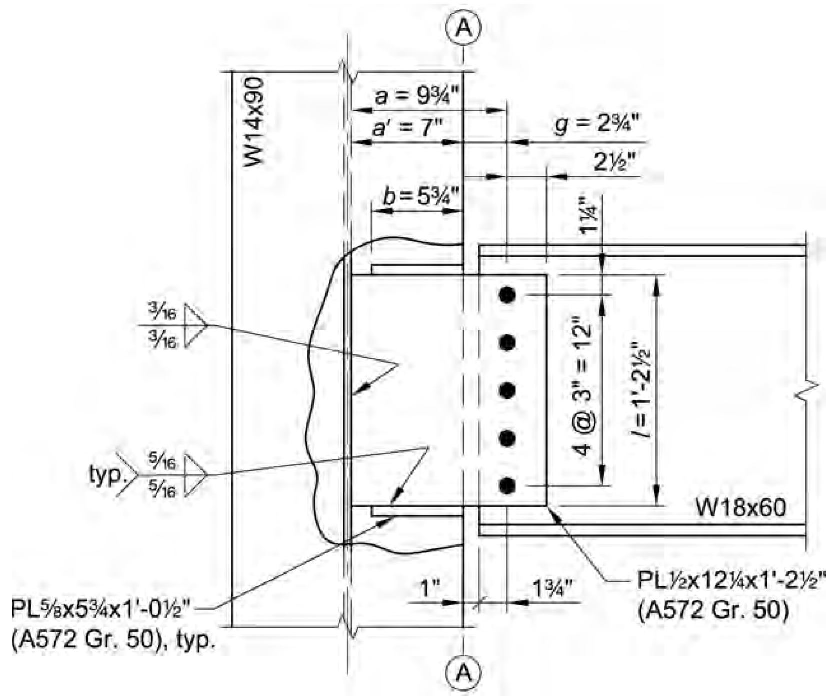


Fig. II.A-19B-4. Design of shear plate with stabilizer plates.

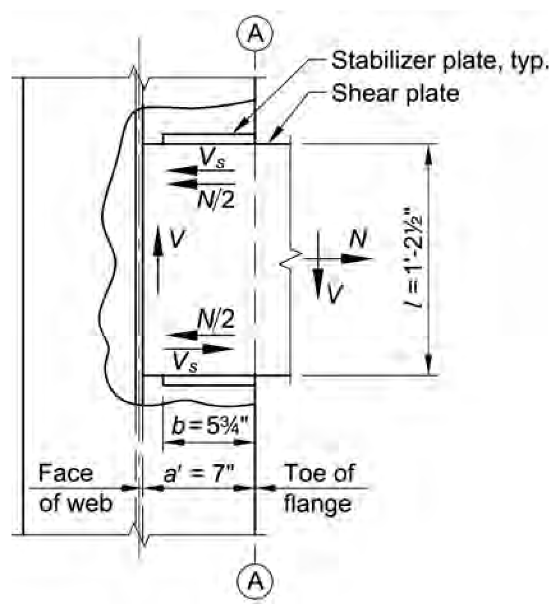


Fig. II.A-19B-5. Forces acting on shear plate.

Stabilizer Plate Design

The stabilizer plate design is shown in Figure II.A-19B-6. The forces in the stabilizer plate are calculated as follows:

LRFD	ASD
Shear: $V_u = \dfrac{F_{up}}{2}$ $= \dfrac{66.2\text{ kips}}{2}$ $= 33.1\text{ kips}$ Moment: $M_u = \dfrac{F_{up}w}{4}$ $= \dfrac{(66.2\text{ kips})(12\tfrac{1}{2}\text{in.})}{4}$ $= 207\text{ kip-in.}$	Shear: $V_a = \dfrac{F_{ap}}{2}$ $= \dfrac{44.1\text{ kips}}{2}$ $= 22.1\text{ kips}$ Moment: $M_a = \dfrac{F_{ap}w}{4}$ $= \dfrac{(44.1\text{ kips})(12\tfrac{1}{2}\text{in.})}{4}$ $= 138\text{ kip-in.}$

Try ⅝-in.-thick stabilizer plates. The available shear strength of the stabilizer plate is determined using AISC *Specification* Section J4.2 as follows:

$$A_{nv} = bt$$
$$= (5\tfrac{3}{4}\text{ in.})(\tfrac{5}{8}\text{ in.})$$
$$= 3.59\text{ in.}^2$$

$$R_n = 0.60F_uA_{nv} \qquad (\textit{Spec.}\text{ Eq. J4-4})$$
$$= 0.60(65\text{ ksi})(3.59\text{ in.}^2)$$
$$= 140\text{ kips}$$

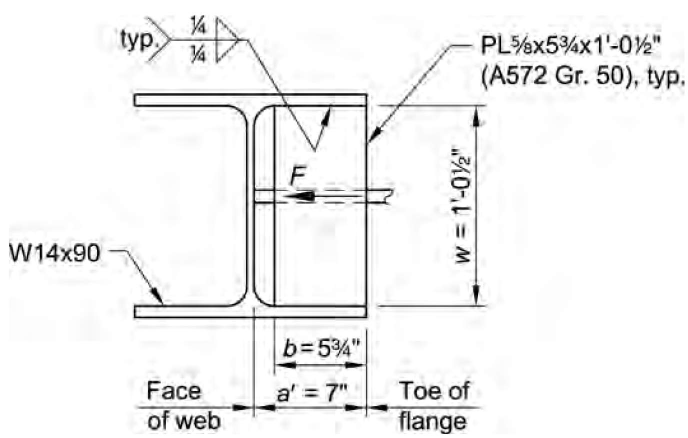


Fig. II.A-19B-6. Stabilizer plate design.

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(140 \text{ kips})$ $= 105 \text{ kips} > 33.1 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{140 \text{ kips}}{2.00}$ $= 70.0 \text{ kips} > 22.1 \text{ kips}$ **o.k.**

The available flexural strength of the stabilizer plate is determined as follows:

$$M_n = F_y Z_x$$
$$= (50 \text{ ksi})\left[\frac{(\tfrac{5}{8} \text{ in.})(5\tfrac{3}{4} \text{ in.})^2}{4}\right]$$
$$= 258 \text{ kip-in.}$$

LRFD	ASD
$\phi = 0.90$ $\phi M_n = 0.90(258 \text{ kip-in.})$ $= 232 \text{ kip-in.} > 207 \text{ kip-in.}$ **o.k.**	$\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{258 \text{ kip-in.}}{1.67}$ $= 154 \text{ kip-in.} > 138 \text{ kip-in.}$ **o.k.**

Stabilizer Plate to Column Weld Design

The required weld size between the stabilizer plate and column flanges is determined using AISC *Manual* Equations 8-2a or 8-2b as follows:

LRFD	ASD
$D_{req} = \dfrac{F_{up}/2}{(2 \text{ welds})(1.392 \text{ kip/in.})b}$ $= \dfrac{(66.2 \text{ kips}/2)}{(2 \text{ welds})(1.392 \text{ kip/in.})(5\tfrac{3}{4} \text{ in.})}$ $= 2.07 \text{ sixteenths}$	$D_{req} = \dfrac{F_{ap}/2}{(2 \text{ welds})(0.928 \text{ kip/in.})b}$ $= \dfrac{(44.1 \text{ kips}/2)}{(2 \text{ welds})(0.928 \text{ kip/in.})(5\tfrac{3}{4} \text{ in.})}$ $= 2.07 \text{ sixteenths}$

The minimum weld size per AISC *Specification* Table J2.4 controls. Use ¼-in. fillet welds.

Shear Plate to Stabilizer Plate Weld Design

The required weld size between the shear plate and stabilizer plates is determined using AISC *Manual* Equations 8-2a or 8-2b as follows:

LRFD	ASD
$D_{req} = \dfrac{F_{up}}{(2 \text{ welds})(1.392 \text{ kip/in.})l_w}$ $= \dfrac{66.2 \text{ kips}}{(2 \text{ welds})(1.392 \text{ kip/in.})(5\tfrac{3}{4} \text{ in.})}$ $= 4.14 \text{ sixteenths}$	$D_{req} = \dfrac{F_{ap}}{(2 \text{ welds})(0.928 \text{ kip/in.})l_w}$ $= \dfrac{44.1 \text{ kips}}{(2 \text{ welds})(0.928 \text{ kip/in.})(5\tfrac{3}{4} \text{ in.})}$ $= 4.13 \text{ sixteenths}$

Use 5⁄16-in. fillet welds.

Strength of Shear Plate at Stabilizer Plate Welds

The minimum shear plate thickness that will match the shear rupture strength of the weld is:

$$t_{min} = \frac{6.19D}{F_u} \qquad \text{(}\textit{Manual}\text{ Eq. 9-3)}$$

$$= \frac{6.19(4.14)}{65\text{ ksi}}$$

$$= 0.394\text{ in.} < \tfrac{1}{2}\text{ in.} \quad \textbf{o.k.}$$

Shear Plate to Column Web Weld Design

The shear plate to stabilizer plate welds act as "crack arrestors" for the shear plate to column web welds. As shown in Figure II.A-19B-7, the required shear force is *V*. The required weld size is determined using AISC *Manual* Equations 8-2a or 8-2b as follows:

LRFD	ASD
$V_u = 75$ kips $D_{req} = \dfrac{V_u}{(2\text{ welds})(1.392\text{ kip/in.})l}$ $= \dfrac{75\text{ kips}}{(2\text{ welds})(1.392\text{ kip/in.})(14\tfrac{1}{2}\text{ in.})}$ $= 1.86$ sixteenths	$V_a = 50$ kips $D_{req} = \dfrac{V_a}{(2\text{ welds})(0.928\text{ kip/in.})l}$ $= \dfrac{50\text{ kips}}{(2\text{ welds})(0.928\text{ kip/in.})(14\tfrac{1}{2}\text{ in.})}$ $= 1.86$ sixteenths

The minimum weld size per AISC *Specification* Table J2.4 controls. Use 3⁄16-in. fillet welds.

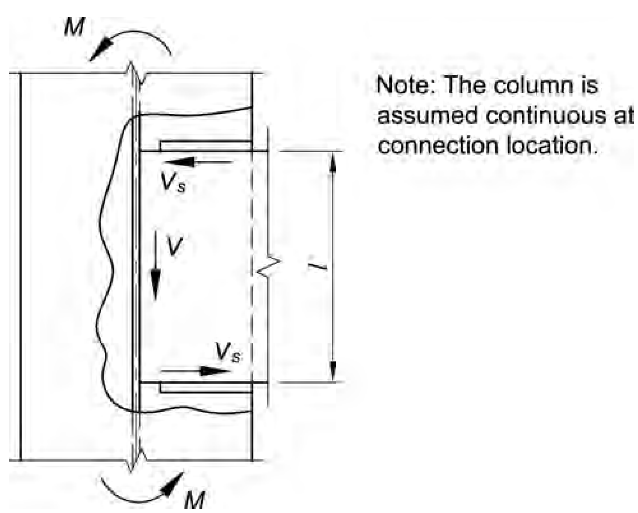


Fig. II.A-19B-7. Moment induced in column.

Strength of Shear Plate at Column Web Welds

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the shear plate is determined as follows:

$$
\begin{aligned}
A_{nv} &= lt \\
&= (14\tfrac{1}{2}\text{ in.})(\tfrac{1}{2}\text{ in.}) \\
&= 7.25\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
R_n &= 0.60F_uA_{nv} \\
&= 0.60(65\text{ ksi})(7.25\text{ in.}^2) \\
&= 283\text{ kips}
\end{aligned}
$$
(*Spec.* Eq. J4-4)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(283\text{ kips})$ $= 212\text{ kips} > 75\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{283\text{ kips}}{2.00}$ $= 142\text{ kips} > 50\text{ kips}$ **o.k.**

Moment in Column

The moment in the column is determined as follows:

LRFD	ASD
$2M_u = V_{us}l$ $= (36.2\text{ kips})(14\tfrac{1}{2}\text{ in.})$ $= 525\text{ kip-in.}$ $M_u = 263\text{ kip-in.}$	$2M_a = V_{as}l$ $= (24.1\text{ kips})(14\tfrac{1}{2}\text{ in.})$ $= 349\text{ kip-in.}$ $M_a = 175\text{ kip-in.}$

The column design needs to be reviewed to ensure that this moment does not overload the column.

Reference

Fortney, P. and Thornton, W. (2016), "Analysis and Design of Stabilizer Plates in Single-Plate Shear Connections," *Engineering Journal*, AISC, Vol. 53, No. 1, pp. 1–18.