Chapter 4
Moment Connections

4.1 W-BEAMS TO HSS COLUMNS

Contemporary information concerning moment connections to HSS columns is limited in literature, although some recommendations appear in design guides such as by Packer and Henderson (1997) and Kurobane et al. (2004). Part 12 of the AISC *Manual* discusses seven types of these connections:

- HSS through-plate flange-plated
- HSS cut-out plate flange-plated
- HSS directly welded
- HSS end plate
- HSS above and below continuous beams
- HSS welded tee flange
- HSS diaphragm plate

As with this entire Design Guide, the scope is restricted to nonseismic applications. Note that HSS-to-HSS moment connections are covered in Chapter 9 of this Design Guide.

The best configuration for a particular application depends on four factors:

1. The magnitude of the moment that must be transferred to the HSS
2. The magnitude of the moment that must be transferred through the HSS
3. The magnitude of the axial force in the HSS
4. The requirements for orthogonal framing

The detail of the connection must accommodate mill tolerances of the W-beam. These include variations in the depth and tilt of the flanges. Shimming in the field with conventional shims or finger shims is the most commonly used method. The examples in this chapter illustrate the limit states that must be considered in design.

4.2 CONTINUOUS BEAM OVER HSS COLUMN

A continuous beam over an HSS column, as shown in Figure 4-1, is a very efficient connection for single-story construction. However, there is an important stability issue that must be considered. In this type of construction, the beam is braced only at the top flange. Therefore, out-of-plane motion at the top of the HSS is restrained only by the relatively weak flexural stiffness of the beam web, greatly increasing the effective length of the column.

The restraint to this motion must be increased by adding stiffeners to the web, as in Figure 4-1(a), or by using a bottom chord extension to the joist supported by the beam at the connection as in Figure 4-1(b), or both, as illustrated in Figure 4-1(c). The stiffener in (a) can also be used as the orthogonal framing connection. See the AISC *Manual* Part 2 for additional information and details for beams framing continuously over columns.

AISC *Manual* CD Companion Example K.11 (AISC, 2005c) is a continuous beam over an HSS column, but only the limit states associated with a concentrated load on the end of an HSS are evaluated. Example 4.1 in Section 4.5 examines all the limit states.

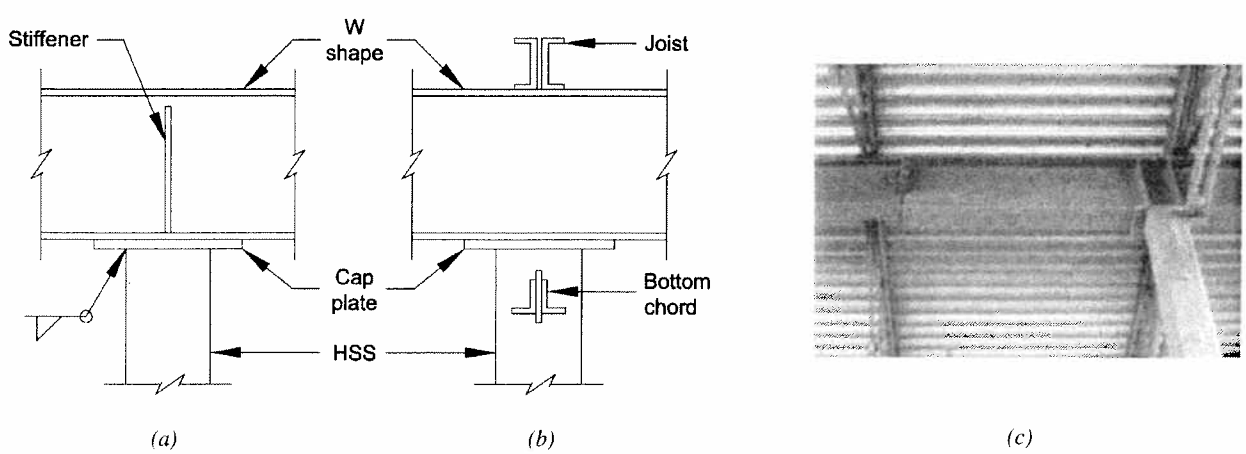


Fig. 4-1. Beam over HSS column connection.

4.3 THROUGH-PLATE CONNECTIONS

If the required moment transfer to the column is larger than can be provided by a bolted base plate or cap plate, or if the HSS width is larger than that of the W-shape beam, a through-plate fully-restrained (FR) connection can be used. Example 4.2 shows a calculation for a through-plate connection.

4.4 DIRECTLY WELDED CONNECTIONS

Some moment transfer can be achieved by welding the flanges of the W-shape directly to the wall of the HSS column as shown in Figure 4-2. The single-plate connection transfers all of the shear force and facilitates erection of the beam. These connections may be capable of developing the full flexural strength of the HSS (depending primarily on the beam width relative to the HSS column width). The full flexural strength of the W-shape, however, is seldom achievable.

The strength of the flanges in tension and compression are calculated by considering them as transverse plates acting on the face of the HSS column. To obtain the maximum efficiency, the HSS should be thick, and the beam flange width should be close to the flat width of the HSS, estimated as $B - 3t$.

The limit states that must be considered in AISC *Specification* Section K1.3b are local yielding of the beam flange due to uneven load distribution, HSS shear yielding (punching), sidewall tension, and sidewall compression failure. However, shear yielding only needs to be checked for certain flange widths and the sidewall limit states only apply when the flange and HSS widths are the same. This is also tabulated in Table 7-2 of this Design Guide. Example 4.3 shows a calculation for a directly welded connection.

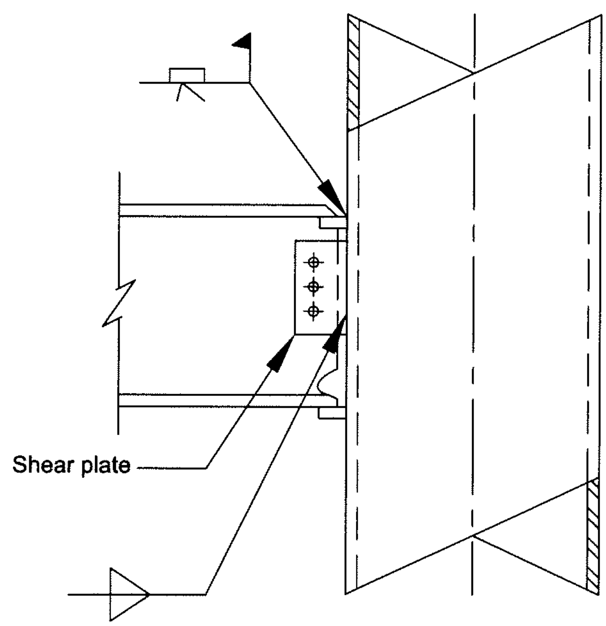


Fig. 4-2. Directly welded connection.

4.5 CONNECTION DESIGN EXAMPLES

Example 4.1—Beam over HSS Column Connection

Q-1) Can we use this connection for MWFRS (?)

Given:

Verify the adequacy of the connection shown in Figure 4-3, between a two-span W18×40 beam supported by an HSS8×8×¼ column. The column axial dead and live loads are $P_D = 7.50$ kips and $P_L = 22.5$ kips, respectively. The moment transferred across the beam-to-column interface consists of dead and live load moments of $M_D = 4.50$ kip-ft and $M_L = 13.5$ kip-ft, respectively.

From AISC *Manual* Tables 2-3 and 2-4, the material properties are as follows:

HSS8×8×¼ HSS 6 x 6 x 1/2
ASTM A500 Grade B
F_y = 46 ksi
F_u = 58 ksi

W18×40 W 14 X 26
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Cap plate
ASTM A36
F_{yp} = 36 ksi
F_{up} = 58 ksi

From AISC *Manual* Table 1-12, the HSS geometric properties are as follows:

HSS8×8×¼
H = 8.00 in.
B = 8.00 in.
t = 0.233 in.

Can we use 4 bolts on each side of column to reduce the tensile Load on bolt (?)

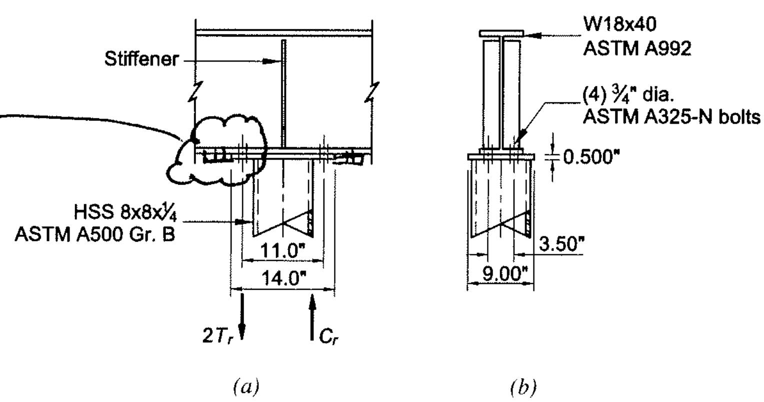


Fig. 4-3. W-beam over HSS column connection.

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

W18×40
d = 17.9 in.
t_w = 0.315 in.
t_f = 0.525 in.
k = 0.927 in.
k_1 = 13⁄16 in.

The gage, g, is 3.50 in. for the W18×40.

The geometric properties of the cap plate and bolts are as follows:

Cap plate
length = 14.0 in.
width = 9.00 in.
t_p = 0.500 in.

ASTM A325 bolts
d_b = ¾ in.

Solution:

From Chapter 2 of ASCE 7, the required strength is:

LRFD	ASD
$P_u = 1.2(7.50\text{ kips}) + 1.6(22.5\text{ kips})$ $= 45.0\text{ kips}$ $M_u = 1.2(4.50\text{ kip-ft}) + 1.6(13.5\text{ kip-ft})$ $= 27.0\text{ kip-ft}$	$P_a = 7.50\text{ kips} + 22.5\text{ kips}$ $= 30.0\text{ kips}$ — 20 k $M_a = 4.50\text{ kip-ft} + 13.5\text{ kip-ft}$ $= 18.0\text{ kip-ft}$ — 40 k

Tensile load in bolt

Calculate the bolt tension, T_r, assuming that the compression force, C_r, acts at the face of the column as shown in Figure 4-3(a).

From the summation of forces and moments with $C_r = C_u$ and $T_r = T_u$ for LRFD, and $C_r = C_a$ and $T_r = T_a$ for ASD, the required bolt tension is determined as follows:

LRFD	ASD
$C_r = C_u;\ T_r = T_u$ $P_u = 45.0\text{ kips}$ $= C_u - 2T_u$ $M_u = 27.0\text{ kip-ft}(12\text{ in./ft})$ $= 324\text{ kip-in.}$ $M_u = \dfrac{8.00\text{ in.}}{2}C_u + \dfrac{11.0\text{ in.}}{2}(2T_u)$ $324\text{ kip-in.} = 4.00\text{ in.}(45.0\text{ kips} + 2T_u) + 5.50\text{ in.}(2T_u)$ $2T_u = 15.2\text{ kips on 2 bolts}$ $T_u = \dfrac{15.2\text{ kips}}{2}$ $= 7.60\text{ kips/bolt}$	$C_r = C_a;\ T_r = T_a$ $P_a = 30.0\text{ kips}$ $= C_a - 2T_a$ $M_a = 18.0\text{ kip-ft}(12\text{ in./ft})$ $= 216\text{ kip-in.}$ $M_a = \dfrac{8.00\text{ in.}}{2}C_a + \dfrac{11.0\text{ in.}}{2}(2T_a)$ $216\text{ kip-in.} = 4.00\text{ in.}(30.0\text{ kips} + 2T_a) + 5.50\text{ in.}(2T_a)$ $2T_a = 10.1\text{ kips on 2 bolts}$ $T_a = \dfrac{10.1\text{ kips}}{2}$ $= 5.05\text{ kips/bolt}$

T_a = 12 kips/bolt

Effect of prying action—W-shape flange

Check the flange thickness for prying action per Part 9 of the AISC *Manual.*

Using Figure 4-3(b):

b = distance from bolt centerline to face of beam web

$$= (g - t_w)/2$$
$$= (3.50 \text{ in.} - 0.315 \text{ in.})/2$$
$$= 1.59 \text{ in.}$$

b' = distance from face of bolt to face of web

$$= b - \frac{d_b}{2}$$
$$= 1.59 \text{ in.} - \frac{3/4 \text{ in.}}{2}$$
$$= 1.22 \text{ in.}$$

p = tributary length per pair of bolts in the perpendicular direction, which should preferably not exceed the gage between the pair of bolts, g

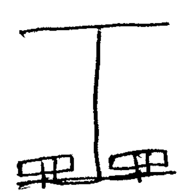
$$= g$$
$$= 3.50 \text{ in.}$$

The following is a simplified check provided in Part 9 of the AISC *Manual* based upon the "no prying action" equation.

LRFD	ASD
$t_{min} = \sqrt{\dfrac{4.44T_u b'}{pF_u}}$	$t_{min} = \sqrt{\dfrac{6.66T_a b'}{pF_u}}$
$= \sqrt{\dfrac{4.44(7.60 \text{ kips/bolt})(1.22 \text{ in.})}{3.50 \text{ in.}(65 \text{ ksi})}}$	$= \sqrt{\dfrac{6.66(5.05 \text{ kips/bolt})(1.22 \text{ in.})}{3.50 \text{ in.}(65 \text{ ksi})}}$
$= 0.425$ in.	$= 0.425$ in.
0.425 in. < 0.525 in. **o.k.**	0.425 in. < 0.525 in. **o.k.**

Because $t_{min} < t_f$, there is no prying action in the beam flange.

$t_{min} > t_p$; Can we use steel plate to increase the bottom flange thickness & steel plate will be welded to beam bottom flange.



Effect of prying action—cap plate

Check the cap plate thickness for prying action per Part 9 of the AISC *Manual.*

Using Figure 4-3(a):

b = distance from bolt centerline to face of HSS

$$= (11.0 \text{ in.} - 8.00 \text{ in.})/2$$
$$= 1.50 \text{ in.}$$

b' = distance from face of bolt to face of HSS

$$= b - \frac{d_b}{2}$$
$$= 1.50 \text{ in.} - \frac{3/4 \text{ in.}}{2}$$
$$= 1.13 \text{ in.}$$

p = tributary length per pair of bolts in the perpendicular direction, which should preferably not exceed the gage between the pair of bolts, g

= half the cap plate width

$= \dfrac{9.00 \text{ in.}}{2}$

= 4.50 in.

Use p = 4.50 in.

The gage limitation given on page 9-11 of the AISC *Manual* does not apply to this configuration because with bending checked at the face of the HSS, and the "gage" being interrupted by the HSS, an artificially large value of g results.

The following is a simplified check provided in Part 9 of the AISC *Manual* based upon the "no prying action" equation.

LRFD	ASD
$t_{min} = \sqrt{\dfrac{4.44 T_u b'}{pF_{up}}}$ $= \sqrt{\dfrac{4.44(7.58 \text{ kips/bolt})(1.13 \text{ in.})}{4.50 \text{ in.}(58 \text{ ksi})}}$ $= 0.382$ in. 0.382 in. < 0.500 in. **o.k.**	$t_{min} = \sqrt{\dfrac{6.66 T_a b'}{pF_{up}}}$ $= \sqrt{\dfrac{6.66(5.05 \text{ kips/bolt})(1.13 \text{ in.})}{4.50 \text{ in.}(58 \text{ ksi})}}$ $= 0.382$ in. 0.382 in. < 0.500 in. **o.k.**

Because $t_{min} < t_p$, there is no prying action in the cap plate.

Bolt available tensile strength

Because there is no prying action, the required strength of the bolt is T_r (T_u for LRFD and T_a for ASD). The available tensile strength, ϕr_n and r_n/Ω, of a ¾-in.-diameter ASTM A325 bolt is given in AISC *Manual* Table 7-2.

LRFD	ASD
$\phi r_n \geq T_u = 7.60$ kips/bolt $\phi r_n = 29.8$ kips/bolt 29.8 kips > 7.60 kips **o.k.**	$r_n/\Omega \geq T_a = 5.05$ kips/bolt $r_n/\Omega = 19.9$ kips/bolt 19.9 kips > 5.05 kips **o.k.**

Beam web local yielding

The limit state of beam web local yielding is checked as follows.
Determine the compression force, C_r, where $C_r = C_u$ for LRFD and $C_r = C_a$ for ASD.

LRFD	ASD
$C_u = P_u + 2T_u$ $= 45.0 \text{ kips} + 2(7.60 \text{ kips/bolt})$ $= 60.2$ kips	$C_a = P_a + 2T_a$ $= 30.0 \text{ kips} + 2(5.05 \text{ kips/bolt})$ $= 40.1$ kips

Assume a 2.5:1 dispersion slope or 21.8° dispersion angle of the load from the HSS wall through the cap plate. From the AISC *Specification Commentary* Section K1.6:

$$N = t + 5t_p$$
$$= 0.233 \text{ in.} + 5(0.500 \text{ in.})$$
$$= 2.73 \text{ in.}$$

$$\begin{aligned}F_{yw} &= F_y \\ &= 50\text{ ksi}\end{aligned}$$

From AISC *Specification* Section J10.2,

$$\begin{aligned}R_n &= \left[5k + N\right]F_{yw}t_w \\ &= \left[5(0.927\text{ in.}) + 2.73\text{ in.}\right](50\text{ ksi})(0.315\text{ in.}) \\ &= 116\text{ kips}\end{aligned} \qquad (\textit{Spec.}\text{ Eq. J10-2})$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(116\text{ kips}) = 116\text{ kips}$ $116\text{ kips} > 60.2\text{ kips}$ **o.k.**	$\Omega = 1.50$ $R_n/\Omega = \dfrac{116\text{ kips}}{1.50} = 77.3\text{ kips}$ $77.3\text{ kips} > 40.1\text{ kips}$ **o.k.**

Beam web crippling

Check the limit state of beam web crippling using AISC *Specification* Section J10.3 and the following equation.

$$R_n = 0.80t_w^2\left[1 + \frac{3N}{d}\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \qquad (\textit{Spec.}\text{ Eq. J10-4})$$

Using AISC *Manual* Table 9-4, this equation can be simplified as follows:

LRFD	ASD
For $N/d = 2.73\text{ in.}/17.9\text{ in.} = 0.153 \le 0.2$: $\phi R_n = 2\left[\phi R_3 + N\phi R_4\right]$ $\phi R_3 = 46.3\text{ kips}$ $\phi R_4 = 3.60\text{ kips/in.}$ $\phi R_n = 2\left[46.3\text{ kips} + 2.73\text{ in.}(3.60\text{ kips/in.})\right]$ $= 112\text{ kips}$ $112\text{ kips} > 60.2\text{ kips}$ **o.k.**	For $N/d = 2.73\text{ in.}/17.9\text{ in.} = 0.153 \le 0.2$: $R_n/\Omega = 2\left[R_3/\Omega + N\,R_4/\Omega\right]$ $R_3/\Omega = 30.9\text{ kips}$ $R_4/\Omega = 2.40\text{ kips/in.}$ $R_n/\Omega = 2\left[30.9\text{ kips} + 2.73\text{ in.}(2.40\text{ kips/in.})\right]$ $= 74.9\text{ kips}$ $74.9\text{ kips} > 40.1\text{ kips}$ **o.k.**

Note that if the total axial force were applied at the beam top flange over the connection, web compression buckling (AISC *Specification* Section J10.5) would also be checked to determine if a stiffener is required over the HSS wall.

HSS wall strength

The compression force in the HSS wall may not be uniformly distributed over the entire HSS wall width due to shear lag. This phenomenon is also discussed in Section 7.4 of this Design Guide. Assume that the force in the beam web disperses at a slope of 2.5:1 (or 21.8°). From Carden et al. (2008):

$$\begin{aligned}N &= 2k_1 + 5t_f \\ &= 2(\tfrac{13}{16}\text{ in.}) + 5(0.525\text{ in.}) \\ &= 4.25\text{ in.}\end{aligned}$$

From AISC *Specification* Section K1.6:

$$\begin{aligned}5t_p + N &= 5(0.500\text{ in.}) + 4.25\text{ in.} \\ &= 6.75\text{ in.} < B = 8.00\text{ in.}\end{aligned}$$

Because $(5t_p + N) < B$, the available strength of the HSS is normally computed by summing the contributions of the two walls into which the load is distributed. However, in this example, only one wall is subjected to the compression load, C_r.

HSS wall local yielding

Check the limit state of HSS wall local yielding using the following equation from AISC *Specification* Section K1.6(i). (Table 7-2 of this Design Guide provides the equation for two walls.) For one wall:

$$R_n = F_y t\left[5t_p + N\right] \le BF_y t \qquad \text{(\textit{Spec.} Eq. K1-11)}$$

$$= 46 \text{ ksi}\left(0.233 \text{ in.}\right)\left[5\left(0.500 \text{ in.}\right) + 4.25 \text{ in.}\right] \le 8.00 \text{ in.}\left(46 \text{ ksi}\right)\left(0.233 \text{ in.}\right)$$

$$= 72.3 \text{ kips} < 85.7 \text{ kips}$$

The available strength is,

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00\left(72.3 \text{ kips}\right)$ $= 72.3 \text{ kips}$ $72.3 \text{ kips} > 60.2 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $R_n/\Omega = \dfrac{72.3 \text{ kips}}{1.50}$ $= 48.2 \text{ kips}$ $48.2 \text{ kips} > 40.1 \text{ kips}$ **o.k.**

HSS wall local crippling

Check the limit state of HSS wall local crippling using the following equation from AISC *Specification* Section K1.6(ii) and Table 7-2.

$$R_n = 0.80t^2\left[1 + \frac{6N}{B}\left(\frac{t}{t_p}\right)^{1.5}\right]\sqrt{\frac{EF_y t_p}{t}} \qquad \text{(\textit{Spec.} Eq. K1-12)}$$

$$= 0.80\left(0.233 \text{ in.}\right)^2\left[1 + \frac{6\left(4.25 \text{ in.}\right)}{8.00 \text{ in.}}\left(\frac{0.233 \text{ in.}}{0.500 \text{ in.}}\right)^{1.5}\right]\sqrt{\frac{(29{,}000 \text{ ksi})\left(46 \text{ ksi}\right)\left(0.500 \text{ in.}\right)}{0.233 \text{ in.}}}$$

$$= 148 \text{ kips}$$

The available strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(148 \text{ kips}\right)$ $= 111 \text{ kips}$ $111 \text{ kips} > 60.2 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $R_n/\Omega = \dfrac{148 \text{ kips}}{2.00}$ $= 74.0 \text{ kips}$ $74.0 \text{ kips} > 40.1 \text{ kips}$ **o.k.**

Weld size between the HSS and cap plate

Assuming a 45° maximum load dispersion angle, the weld of the cap plate to the HSS can be checked by evaluating a load per inch of the lesser of T_r/g or $T_r/(d_b + 2b)$, where b is the distance from the bolt center to the face of the HSS. If $2g$ or $2(d_b + 2b)$ exceeds the width of the HSS, B, the weld should be checked for $2T/B$.

Check if $2g \le B$ and $2(d_b + 2b) \le B$:

$$2g \le B$$

$$2\left(3.50 \text{ in.}\right) = 7.00 \text{ in.} < 8.00 \text{ in.} \qquad \textbf{o.k.}$$

$$2(d_b + 2b) \le B$$

$$2\left[\tfrac{3}{4} \text{ in.} + 2\left(11.0 \text{ in.} - 8.00 \text{ in.}\right)/2\right] = 7.50 \text{ in.} < 8.00 \text{ in.} \qquad \textbf{o.k.}$$

Determine the greater of T_r/g or $T_r/(d_b + 2b)$:

LRFD	ASD
With $T_r = T_u$ $\frac{T_u}{g} = \frac{7.60 \text{ kips}}{3.50 \text{ in.}}$ $= 2.17 \text{ kips/in.}$ $\frac{T_u}{d_b + 2b} = \frac{7.60 \text{ kips}}{\frac{3}{4} \text{ in.} + 2(11.0 \text{ in.} - 8.00 \text{ in.})}$ $= 1.13 \text{ kips/in.}$ Use T_u/g in the weld size determination.	With $T_r = T_a$ $\frac{T_a}{g} = \frac{5.05 \text{ kips}}{3.50 \text{ in.}}$ $= 1.44 \text{ kips/in.}$ $\frac{T_a}{d_b + 2b} = \frac{5.05 \text{ kips}}{\frac{3}{4} \text{ in.} + 2(11.0 \text{ in.} - 8.00 \text{ in.})}$ $= 0.748 \text{ kips/in.}$ Use $T_a/(d_b + 2b)$ in the weld size determination.

Using the procedure given in the AISC *Manual* Part 8, determine the weld size required.

LRFD	ASD
$\phi R_n = 1.392Dl$ $\frac{\phi R_n}{l} \geq \frac{T_u}{g}$ $1.392D \geq \frac{7.60 \text{ kips}}{3.50 \text{ in.}}$ $D \geq 1.56$ sixteenths-of-an-inch Use ⅛-in. minimum weld size with 70-ksi filler metal.	$\frac{R_n}{\Omega} = 0.928Dl$ $\frac{R_n}{\Omega l} \geq \frac{T_a}{g}$ $0.928D \geq \frac{5.05 \text{ kips}}{3.50 \text{ in.}}$ $D \geq 1.55$ sixteenths-of-an-inch Use ⅛-in. minimum weld size with 70-ksi filler metal.

Example 4.2—Through-Plate Connection

Given:

Design a through-plate connection as shown in Figure 4-4. Use 1-in.-diameter A325-N bolts in standard holes and 70-ksi filler metal. The applied dead and live loads are shown in Figure 4-4. The loads on the two beams can be reversed; therefore, the connection should be symmetric.

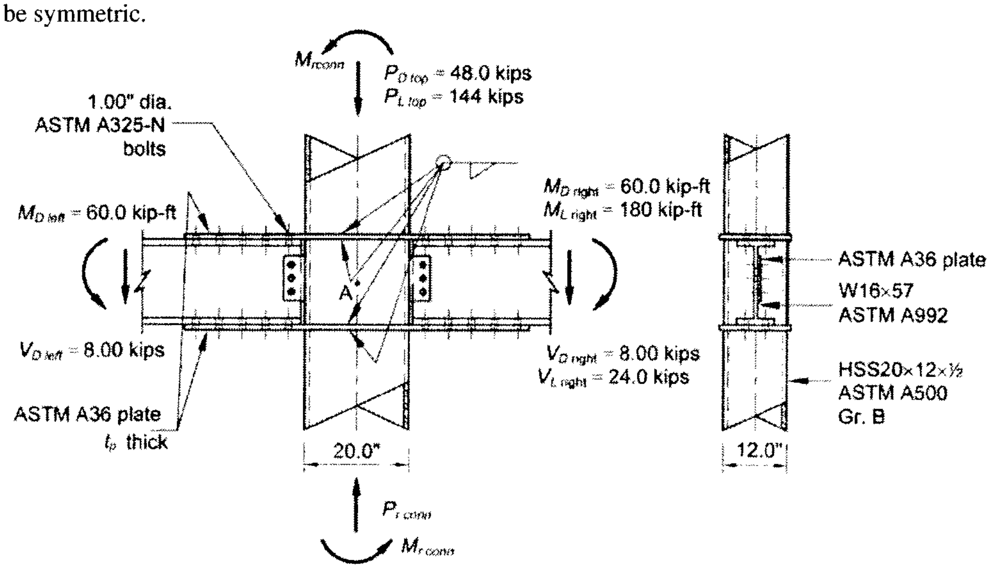


Fig. 4-4. Through-plate moment connection.