

Example 3.14. EBF Link Design

Given: Refer to Beam BM-1 in Figure 3-27. Determine the adequacy of a W16x77 ASTM A992 wide-flange section ($F_y = 50$ ksi, $F_u = 65$ ksi) as the link segment for the following loading. The Applicable Building Code specifies the use of ASCE 7 for calculation of loads.

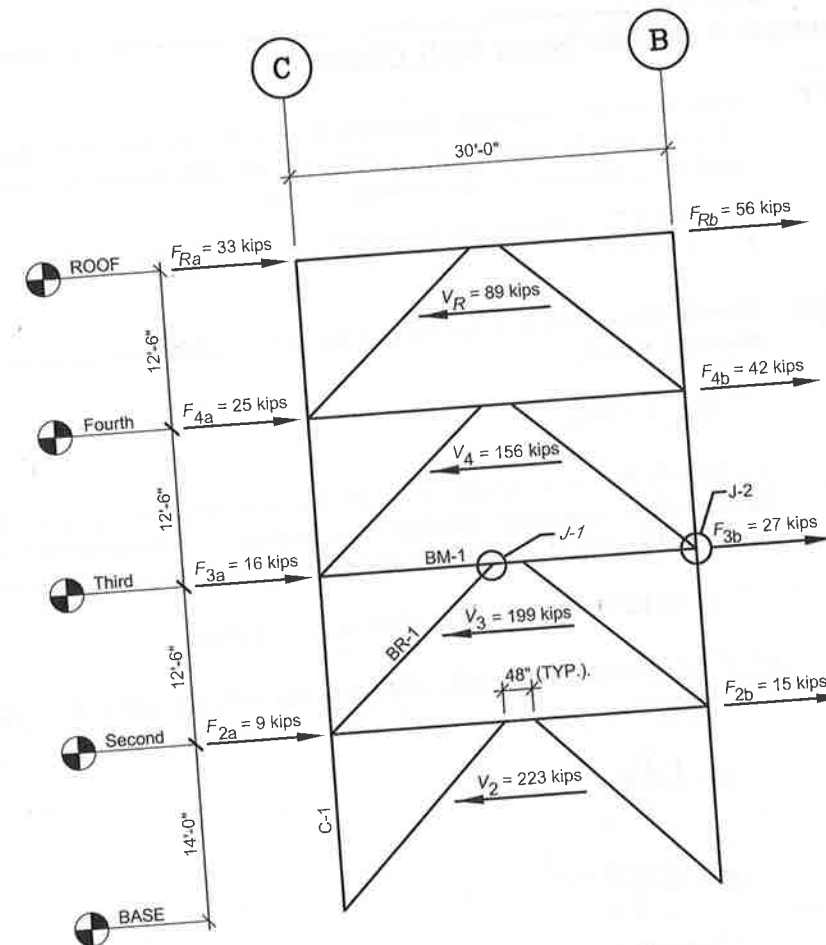


Figure 3-27. EBF elevation for Examples 3.13, 3.14, 3.15, 3.16, and 3.17.

$P_D = 7.4$ kips	$P_L = 5.3$ kips	$P_L = 5.3$ kips
$V_D = 1.8$ kips	$V_L = 1.3$ kips	$V_{Q_E} = 84$ kips
$M_D = 14.4$ kip-ft	$M_L = 9.6$ kip-ft	$M_{Q_E} = 168$ kip-ft

W16x77	$d = 16.5$ in.	$t_w = 0.455$ in.	$A_g = 22.6$ in. ²
	$t_f = 0.760$ in.	$b_f = 10.3$ in.	$Z_x = 150$ in. ³

From ASCE 7, Seismic Design Category is D, $\rho = 1.3$ and $S_{DS} = 1.0$.

$$0.2S_{DS} = 0.2(1.0) = 0.2$$

Assume the brace-to-beam connection will be that shown in Seismic Provisions Figure C-I-15.5. The brace will be detailed as fixed to the link in order to attract some link moment, decreasing the flexural demand on the beam outside of the link. This will reduce, if not eliminate, the need for additional flange bracing of the beam outside of the link. Assume the brace will be a W10 wide-flange section.

Solution: Determine the factored loads on the link

Considering the load combinations given in ASCE 7, it was determined that the governing load combination for the link is,

$$1.2D + 1.0E + 0.5L + 0.2S$$

$$P_u = (1.2 + 0.2S_{DS})P_D + \rho P_{Q_E} + 0.5P_L + 0.2P_S$$

$$P_u = 1.4(7.4 \text{ kips}) + 1.3(5.5 \text{ kips}) + 0.5(5.3 \text{ kips}) + 0.2(0) = 20.2 \text{ kips}$$

$$V_u = (1.2 + 0.2S_{DS})V_D + \rho V_{Q_E} + 0.5V_L + 0.2V_S$$

$$V_u = 1.4(1.8 \text{ kips}) + 1.3(84 \text{ kips}) + 0.5(1.3 \text{ kips}) + 0.2(0) = 112 \text{ kips}$$

$$M_u = (1.2 + 0.2S_{DS})M_D + \rho M_{Q_E} + 0.5M_L + 0.2M_S$$

$$M_u = 1.4(14.4 \text{ kip-ft}) + 1.3(168 \text{ kip-ft}) + 0.5(9.6 \text{ kip-ft}) + 0.2(0) = 243 \text{ kip-ft}$$

Check geometry

Seismic Provisions Section 15.5a prohibits the extension of any portion of the brace-to-beam connection within the link segment. This section also requires that the intersection of the centerlines of the beam and brace occurs at the end of the link or within the link segment. Assuming the intersection will occur at the end of the link for the connection used and the geometry shown in Figure 3-27, the minimum depth of the link beam necessary to preclude the W10 brace from extending inside the link segment is approximately 16 in.

$$d_b > 16 \text{ in.} \quad \text{o.k.}$$

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link beam flange width must be greater
th. Assuming that the maximum flange

, the stiffened and unstiffened elements of
visions Table I-8-1.

anges is,

(Specification B4.1)

3-1, for flange compactness,

$$\frac{100 \text{ ksi}}{\text{ksi}} = 7.22$$

the local buckling requirements.

web is,

(Specification B4.2)

$$\frac{20.2 \text{ kips}}{0.9(50 \text{ ksi})(22.6 \text{ in.}^2)} = 0.0199$$

Provisions Table I-8-1, for web compactness,

$$1 - 1.54(0.0199)$$

ets the local buckling requirements.

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Alternatively, using Table 1-2, it can be seen that a W16×77 will satisfy the local buckling requirements for an EBF column as long as $P_u < 1259$ kips.

$$P_u < 1259 \text{ kips} \quad \text{o.k.}$$

Determine the shear strength of the link

$$0.15P_y = 0.15F_yA_g = 0.15(50 \text{ ksi})(22.6 \text{ in.}^2) = 170 \text{ kips}$$

Alternatively, from Table 3-1 for a W16×77,

$$0.15P_y = 170 \text{ kips}$$

With $P_u < 0.15P_y$, Seismic Provisions Section 15.2b allows the effect of the axial force on the link shear strength to be ignored. For this case the nominal shear strength of the link is defined as the lesser of V_p or $2M_p/e$. Seismic Provisions Section 15.2b defines V_p as,

$$V_p = 0.6F_yA_w$$

where,

$$A_w = (d_b - 2t_f)t_w = [16.5 \text{ in.} - 2(0.760 \text{ in.})]0.455 \text{ in.} = 6.82 \text{ in.}^2$$

$$V_p = 0.6(50 \text{ ksi})(6.82 \text{ in.}^2) = 205 \text{ kips}$$

$$M_p = Z_xF_y = 150 \text{ in.}^3(50 \text{ ksi}) = 7500 \text{ kip-in.}$$

Alternatively, from Table 3-1 for a W16×77,

$$V_p = 205 \text{ kips}$$

$$M_p = 7500 \text{ kip-in.}$$

$$\frac{2M_p}{e} = \frac{2(7500 \text{ in.-kips})}{48 \text{ in.}} = 313 \text{ kips}$$

The Commentary to Seismic Provisions Section 15.2 suggests that the plastic story drift can be conservatively assumed to equal the design story drift. Using the design story drift determined in Example 3.13,

$$\theta_p = \frac{0.700 \text{ in.}}{12.5 \text{ ft}(12 \text{ in./ft})} = 0.00467 \text{ rad}$$

$$\gamma_p = \frac{30 \text{ ft}(12 \text{ in./ft})}{48 \text{ in.}}(0.00467 \text{ rad}) = 0.0350 \text{ rad} < 0.08 \text{ rad}$$

$$\gamma_p < 0.08 \text{ rad} \quad \text{o.k.}$$

specifies a maximum link rotation angle
the link. The expected link behavior is deter-
h to multiples of M_p/V_p .

can be rearranged, yielding,

$$= 1.31 < 1.6$$

than 1.6 indicates that the link behavior will be

for a W16x77,

link behavior is dominated by shear yielding.

the link rotation angle for this type of expected link
mic Provisions Figure C-I-15.3 defines the link

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The W16x77 is adequate to resist the loads given for the link segment of Beam BM-1.

Check lateral bracing requirements

Seismic Provisions Section 15.5 requires that the beam flanges at each end of the link be braced to resist the following required force:

$$\begin{aligned} R_u &= 0.06 R_y Z F_y / h_o \\ &= 0.06(1.1) \left(150 \text{ in.}^3 \right) (50 \text{ ksi}) / (15.7 \text{ in.}) \\ &= 31.5 \text{ kips} \end{aligned}$$

Top and bottom flange bracing with a design strength greater than 31.5 kips will be provided at each end of the link segment.

Check stiffener requirements

Seismic Provisions Section 15.3 requires double-sided, full-depth stiffeners at each end of the link. The minimum required combined width of the stiffeners is,

$$w_{min} = \frac{b_f - 2t_w}{2} = \frac{10.3 \text{ in.} - 2(0.455 \text{ in.})}{2} = 4.70 \text{ in.}$$

The minimum required thickness is,

$$\begin{aligned} t_{min} &= 0.75t_w \geq \frac{3}{8} \text{ in.} \\ &= 0.75(0.455 \text{ in.}) \geq \frac{3}{8} \text{ in.} \\ &= 0.341 \text{ in.} \geq \frac{3}{8} \text{ in.} \end{aligned}$$

W16×77,

ers will be provided on both sides of the
ent.

o requires full-depth intermediate web stiff-
e the length of the link is less than $1.6M_p/V_p$,
mediate web stiffeners are as follows:

.08 radian,

$$\left(\frac{16.5 \text{ in.}}{5}\right) = 10.3 \text{ in.}$$

0.02 radian or less,

$$-\left(\frac{16.5 \text{ in.}}{5}\right) = 20.4 \text{ in.}$$

or a W16×77,

mits using the calculated link rotation angle, the
web stiffeners is 17.9 in.

in., the intermediate stiffeners are required on one
minimum required thickness of the intermediate web

in.

intermediate stiffeners is,

$$\frac{3 \text{ in.}}{2} - 0.455 \text{ in.} = 4.70 \text{ in.}$$

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Alternatively, from Table 3-1 for a W16×77,

$$w_{min} = 4.69 \text{ in.}$$

$$t_{min} = \frac{1}{2} \text{ in.}$$

Full-depth, 1/2-in. × 4³/₄-in. intermediate web stiffeners will be provided within the link segment, on one side of the web only and at a maximum spacing of 17.9 in.

Note that it may be beneficial to also use 1/2-in.-thick material for the link end stiffeners in order to simplify the detailing and fabrication of the link.

Seismic Provisions Section 15.3 also specifies that the required strength of the welds connecting the link stiffeners to the link web is $A_{st}F_y$, and of the welds connecting the link stiffeners to the link flanges is $A_{st}F_y/4$. For the 1/2-in.-thick stiffener, the area of the stiffener is,

$$A_{st} = 0.5 \text{ in.}(4.75 \text{ in.}) = 2.38 \text{ in.}^2$$

Assuming 3/4-in. clips to allow the stiffeners to clear the fillets, the minimum double-sided fillet weld size connecting the link stiffeners to the link web is,

$$\begin{aligned} D &= \frac{F_y A_{st}}{2(1.392 \text{ kips/in.}) \left[d - 2t_f - 2\left(\frac{3}{4} \text{ in.}\right) \right]} \\ &= \frac{36 \text{ ksi}(2.38 \text{ in.}^2)}{2(1.392 \text{ kips/in.}) \left[16.5 \text{ in.} - 2(0.760 \text{ in.}) - 1.50 \text{ in.} \right]} \\ &= 2.28 \text{ sixteenths} \end{aligned}$$

Checking Specification Table J2.4, with the 1/2-in. stiffener plate thickness, the minimum fillet weld size is 3/16 in.

Use double-sided, 3/16-in. fillet welds to connect the link stiffeners to the link web.

The minimum double-sided fillet weld size connecting the link stiffeners to the link flanges is,

$$\begin{aligned} D &= \frac{F_y A_{st}}{4(2)(1.392 \text{ kips/in.})(4.75 \text{ in.} - \frac{3}{4} \text{ in.})} \\ &= \frac{36 \text{ ksi}(2.38 \text{ in.}^2)}{4(2)(1.392 \text{ kips/in.})(4.00 \text{ in.})} \\ &= 1.92 \text{ sixteenths} \end{aligned}$$

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with the $3/4$ -in. link flange thickness, the
to connect the link stiffeners to the link

o use double-sided, $1/4$ -in. fillet welds to
web in order to simplify the detailing and

Design of the Link

27. Determine the adequacy of the ASTM
($F_y = 65$ ksi) link segment selected in Example
for the following loading. The Applicable
ASCE 7 for calculation of loads.

$P_{Q_E} = 105$ kips
 $V_{Q_E} = 8.7$ kips
 $M_{Q_E} = 113$ kip-ft

$I_x = 1110$ in.⁴ $R_y = 1.1$

Category is D, $\rho = 1.3$, and $S_{DS} = 1.0$.

the columns will be W14 wide-flange sections,
are braced at the columns.

15.6(b), the required strength of the beam outside
the factored gravity forces plus the forces gener-
shear strength of the link, $R_y V_n$. From Example
h of the link was determined to be 205 kips.

(kips) = 248 kips

axial force in the beam outside of the link based
h of the link is,

$$\frac{248 \text{ kips}(30 \text{ ft})}{2(12.5 \text{ ft})} = 298 \text{ kips}$$

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As shown in Figure 3-28, the resulting link end moment based on the expected
shear strength of the link is,

$$M = \frac{1.1 R_y V_n e}{2} = \frac{248 \text{ kips}(48 \text{ in.})}{2} = 5,950 \text{ kip-in.}$$

From Example 3.14, the brace-to-beam connection will be detailed as a fixed
connection; therefore, the moment at the end of the link will be distributed
between the brace and the beam outside of the link. One way to determine the
portion of this moment resisted by the beam outside of the link is based on
relative member stiffness. Since the modulus of elasticity is the same for both
members, it can be neglected in the stiffness calculation. Using relative mem-
ber stiffness to distribute the link end moment, the portion of the moment taken
by the beam outside of the link is,

$$\frac{I_{bol}}{L_{bol}} \div \frac{I_{bol}}{L_{bol}} + \frac{I_{br}}{L_{br}}$$

$$L_{br} = \sqrt{(12.5 \text{ ft})^2 + (13 \text{ ft})^2} = 18.0 \text{ ft}$$

$$\frac{I_{bol}}{L_{bol}} = \frac{1110 \text{ in.}^4}{13 \text{ ft}} = 85.4$$

$$\frac{I_{br}}{L_{br}} = \frac{716 \text{ in.}^4}{18.0 \text{ ft}} = 39.8$$

$$\frac{I_{bol}}{L_{bol}} \div \frac{I_{bol}}{L_{bol}} + \frac{I_{br}}{L_{br}} = \frac{85.4}{85.4 + 39.8} = 0.682$$

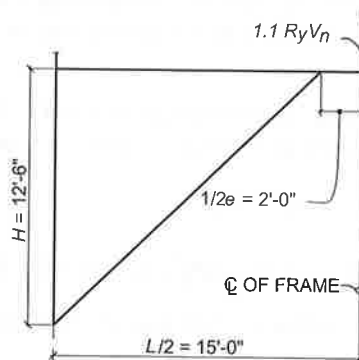


Figure 3-28. Diagram for Example 3.15.

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of the link is assumed to take 68.2 percent
t in the beam outside of the link is then,

$$4,060 \text{ kip-in.} = 338 \text{ kip-ft}$$

the calculation of a link shear overstrength
the strain hardened expected yield strength
shear generated by the code specified earth-
quake strength factor is used to amplify the remaining
the analysis using the code specified earth-
quake strength factor obtained from a computer analysis using
is,

S,

of the link due to the link mechanism is,

$$6(113 \text{ kip-ft}) = 333 \text{ kip-ft}$$

side of the link due to the link mechanism is,

$$(105 \text{ kips}) = 310 \text{ kips}$$

of the link due to the link mechanism is,

$$(8.7 \text{ kips}) = 25.7 \text{ kips}$$

by the two methods are very similar. Since the
has already been determined, the forces gener-
ator method will be used in the calculation of

tions given in ASCE 7, it was determined that the
for the beam outside of the link is,

2S

$$S)P_D + P_E + 0.5P_L + 0.2P_S$$

$$+ 310 \text{ kips} + 0.5(0.7 \text{ kips}) + 0.2(0)$$

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$$V_u = (1.2 + 0.2S_{DS})V_D + V_E + 0.5V_L + 0.2V_S$$

$$V_u = 1.4(6.8 \text{ kips}) + 25.7 \text{ kips} + 0.5(4.8 \text{ kips}) + 0.2(0)$$

$$= 37.6 \text{ kips}$$

$$M_u = (1.2 + 0.2S_{DS})M_D + M_E + 0.5M_L + 0.2M_S$$

$$M_u = 1.4(17 \text{ kip-ft}) + 333 \text{ kip-ft} + 0.5(11.3 \text{ kip-ft}) + 0.2(0)$$

$$= 362 \text{ kip-ft}$$

Check local buckling

From Example 3.14, the flanges are compact. The width-thickness ratio for the web is,

$$\lambda_w = \frac{h}{t_w} = 29.9 \quad (\text{Specification B4.2})$$

From Specification Table B4.1,

$$\begin{aligned} \lambda_p &= 3.76 \sqrt{\frac{E}{F_y}} \\ &= 3.76 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} \\ &= 90.6 \end{aligned}$$

Since $\lambda_w < \lambda_p$, the web meets the local buckling requirements.

Determine unbraced length

From Example 3.14, each end of the link will be braced. Therefore, the unbraced length of the beam outside of the link is,

$$\begin{aligned} L_b &= \frac{L - e - 2\left(\frac{d_c}{2}\right)}{2} \\ &= \frac{30 \text{ ft}(12 \text{ in./ft}) - 48 \text{ in.} - 14 \text{ in.}}{2} \\ &= 149 \text{ in.} = 12.4 \text{ ft} \end{aligned}$$

Consider second-order effects

$$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} \geq 1 \quad (\text{Specification C2.2})$$

$$pP_u + b_x M_{ux} + b_y M_{uy} = 0.374 + 0.566 = 0.940 < 1.0 \quad \text{o.k.}$$

The W16×77 is adequate to resist the loads given for the beam outside of the link segments of Beam BM-1. Additional flange bracing is not required.

Example 3.16. EBF Brace Design

Given: Refer to Brace BR-1 in Figure 3-27. Select an ASTM A992 wide-flange section ($F_y = 50$ ksi, $F_u = 65$ ksi) to resist the following loads. The Applicable Building Code specifies the use of ASCE 7 for calculation of loads.

$$\begin{array}{lll} P_D = 11.8 \text{ kips} & P_L = 8.3 \text{ kips} & P_{Q_E} = 136 \text{ kips} \\ V_D = 0.2 \text{ kips} & V_L = 0.12 \text{ kips} & V_{Q_E} = 3.02 \text{ kips} \\ M_D = 3.2 \text{ kip-ft} & M_L = 2.2 \text{ kip-ft} & M_{Q_E} = 54.5 \text{ kip-ft} \end{array}$$

From ASCE 7, Seismic Design Category is D, $\rho = 1.3$, and $S_{DS} = 1.0$.

$$0.2S_{DS} = 0.2(1.0) = 0.2$$

Assume that link segment and beam outside of the link segments are those selected in Examples 3.13 and 3.14 and that the column-end of the brace is pinned and braced against translation for both the X-X and Y-Y axes.

Solution: Determine the factored loads

Per Seismic Provisions Section 15.6, the required strength of the brace is a combination of the factored gravity forces plus the forces generated by 1.25 times the expected shear strength of the link, $R_y V_n$. From Example 3.14, the nominal shear strength of the link was determined to be 205 kips.

$$1.25R_y V_n = 1.25(1.1)(205 \text{ kips}) = 282 \text{ kips}$$

Using the overstrength factor method described in Example 3.15 with the link shear force, V_{Q_E} , reported in Example 3.15, the overstrength factor is,

$$\frac{1.25R_y V_n}{V_{Q_E}} = \frac{282 \text{ kips}}{84 \text{ kips}} = 3.36$$

The moment in the brace due to the link mechanism is,

$$M_E = 3.36M_{Q_E} = 3.36(54.5 \text{ kip-ft}) = 183 \text{ kip-ft}$$

the link mechanism is,

$$) = 457 \text{ kips}$$

link mechanism is,

$$s) = 10.1 \text{ kips}$$

given in ASCE 7, it was determined that the brace is,

$$+ 0.5P_L + 0.2P_S$$

$$\text{kips} + 0.5(8.3 \text{ kips}) + 0.2(0)$$

$$E + 0.5V_L + 0.2V_S$$

$$\text{kips} + 0.5(0.12 \text{ kips}) + 0.2(0)$$

$$M_E + 0.5M_L + 0.2M_S$$

$$\text{kip-ft} + 0.5(2.2 \text{ kip-ft}) + 0.2(0)$$

ptions used in Examples 3.13 and 3.14 and the W10×112 for the brace.

$$\text{in.}^2 \quad d = 11.4 \text{ in.} \quad t_w = 0.755 \text{ in.}$$

$$\text{in.}^4$$

on 15.6, the stiffened and unstiffened elements of λ_p from Specification Table B4.1.

the flanges is,

(Specification B4.1)

From Specification Table B4.1, for flange compactness,

$$\begin{aligned} \lambda_p &= 0.38 \sqrt{\frac{E_s}{F_y}} \\ &= 0.38 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} \\ &= 9.15 \end{aligned}$$

Since $\lambda_f < \lambda_p$, the flanges meet the local buckling requirements.

$$\lambda_w = \frac{h}{t_w} = 10.4 \quad (\text{Specification B4.2})$$

From Specification Table B4.1, for web compactness,

$$\begin{aligned} \lambda_p &= 3.76 \sqrt{\frac{E}{F_y}} \\ &= 3.76 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} \\ &= 119 \end{aligned}$$

Since $\lambda_w < \lambda_p$, the web meets the local buckling requirements.

Alternatively, using Table 1-2, it can be seen that the W10×112 will satisfy the local buckling requirements for an EBF brace as long as $P_u < 2,880$ kips.

$$P_u < 2,880 \text{ kips} \quad \text{o.k.}$$

Determine unbraced length

$$L_b = \sqrt{(12.5 \text{ ft})^2 + (13 \text{ ft})^2} = 18.0 \text{ ft} = 216 \text{ in.}$$

Note that the unbraced length is based on the work point-to-work point distance. Shorter lengths may be used provided the lateral support is adequate at each end of the assumed brace length.

Consider second-order effects

$$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} \geq 1 \quad (\text{Specification C2-2})$$

(Specification C2-3)

Since $\frac{P_u}{\phi_c P_n} > 0.2$,

$$pP_u + b_x M_{ux} + b_y M_{uy} = 0.516 + 0.29 \times 0.325 = 0.841 < 1.0 \quad \text{o.k.}$$

Check shear strength

$$2.24 \sqrt{\frac{E}{F_{yw}}} = 2.24 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} = 54.0$$

Since $h/t_w < 2.24 \sqrt{E/F_{yw}}$,

$$V_n = 0.6 F_y A_w C_v \quad (\text{Specification G2-1})$$

$$C_v = 1.0$$

$$\phi V_n = 1.0 (0.6) (50 \text{ ksi}) (11.4 \text{ in.}) (0.755 \text{ in.}) \quad (\text{Specification G2-2})$$

$$= 258 \text{ kips} > 10.4 \text{ kips}$$

$$\frac{P_u}{\phi_c P_n} = \frac{458 \text{ kips}}{1130 \text{ kips}} = 0.405$$

Alternatively, using Table 4-2 ($\phi = 1.00$) for the W10×112 brace,

$$\phi V_n = 0.90 (\phi R_{v1}) = 0.90 (258 \text{ kips}) = 232 \text{ kips}$$

$$V_u < \phi V_n \quad \text{o.k.}$$

The W10×112 is adequate to resist the loads given for Brace BR-1.

Example 3.17. EBF Column Design

Given: Refer to Column C-1 in Figure 3-27. Select an ASTM A992 wide-flange section ($F_y = 50 \text{ ksi}$, $F_u = 65 \text{ ksi}$) to resist the following loading between the base and second level. The Applicable Building Code specifies the use of ASCE 7 for calculation of loads.

$$P_D = 151 \text{ kips}$$

$$P_L = 46 \text{ kips}$$

$$P_{Q_E} = 172 \text{ kips}$$

$$M_{Dx} = 15 \text{ kip-ft}$$

$$M_{Lx} = 9 \text{ kip-ft}$$

$$M_{Q_{Ex}} = 0 \text{ kip-ft}$$

$$M_{Dy} = 10 \text{ kip-ft}$$

$$M_{Ly} = 6 \text{ kip-ft}$$

$$M_{Q_{Ey}} = 0 \text{ kip-ft}$$

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ory is D, $\Omega_o = 2.0$, $\rho = 1.3$, and $S_{DS} = 1.0$.

pinned and braced against translation for
at the beam at the third level and brace
are as designed in Examples 3.13, 3.14, and
strengths for the links at the fourth level and

in ASCE 7, it was determined that the gov-
column in compression is,

$$P_{Q_E} + 0.5P_L + 0.2P_S$$

$$172 \text{ kips} + 0.5(46 \text{ kips}) + 0.2(0) = 458 \text{ kips}$$

$$\rho M_{Q_E} + 0.5M_L + 0.2M_S$$

$$0 + 0.5(9 \text{ kip-ft}) + 0.2(0)$$

$$(0) + 0.5(6 \text{ kip-ft}) + 0.2(0)$$

ation for the column in tension is,

$$+ \rho P_{Q_E} + 1.6P_H$$

$$0.3(-172 \text{ kips}) + 1.6(0)$$

$$M_D + \rho M_{Q_E} + 1.6M_H$$

$$0.3(0) + 1.6(0)$$

$$1.3(0) + 1.6(0)$$

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Try a W14×99.

$$A_g = 29.1 \text{ in.}^2$$

$$I_x = 1110 \text{ in.}^4$$

$$I_y = 402 \text{ in.}^4$$

Check $P_u/\phi P_n$

From Manual Table 4-1 with $L = 14 \text{ ft}$,

$$\phi_c P_n = 1,130 \text{ kips}$$

$$\frac{P_u}{\phi_c P_n} = \frac{458 \text{ kips}}{1130 \text{ kips}} = 0.405$$

Per Seismic Provisions Section 8.3, since $P_u/\phi_c P_n > 0.4$, the special seismic load combinations that include the Amplified Seismic Load effects must be used to determine the required axial compression and tension strengths of the column. However, the adequacy of the column to resist these axial loads is permitted to be evaluated in the absence of any applied shear or moment.

Determine factored loads based on amplified seismic load

The required compressive strength of the column including the Amplified Seismic Load effects is,

$$P_u = (1.2 + 0.2S_{DS})P_D + \Omega_o P_{Q_E} + 0.5P_L + 0.2P_S$$

$$P_u = 1.4(151 \text{ kips}) + 2.0(172 \text{ kips}) + 0.5(46 \text{ kips}) + 0.2(0)$$

$$= 578 \text{ kips}$$

The required tensile strength of the column including the Amplified Seismic Load effects is,

$$T_u = (0.9 - 0.2S_{DS})P_D + \Omega_o P_{Q_E} + 1.6P_H$$

$$T_u = 0.7(151 \text{ kips}) + 2.0(-172 \text{ kips}) + 1.6(0 \text{ kips})$$

$$= -238 \text{ kips}$$

Determine required column strength per Seismic Provisions Section 15.8

Seismic Provisions Section 15.8 requires the column to have the strength to resist the forces generated by the sum of the strain hardened expected yield strengths of the links above the level of the column top in addition to the factored gravity forces. From Example 3.14, the nominal shear strength of the link at the third level is 205 kips. Therefore, the sum of the strain hardened expected yield strengths of the links at the third level, fourth level, and roof is,

$$1.1R_y \sum V_n = 1.1(1.1)(318 \text{ kips} + 205 \text{ kips}) = 633 \text{ kips}$$

for the column in compression,

$$+ 0.5P_L + 0.2P_S \\ + 0.5(46 \text{ kips}) + 0.2(0)$$

for the column in tension,

$$+ 1.6P_H \\ + 1.6(0 \text{ kips})$$

calculated to this point, the load combination
column is,

$$M_{ux} = 26.8 \text{ kip-ft} \quad M_{uy} = 17.0 \text{ kip-ft}$$

ffects

(Specification C2-2)

nce the moments are so small, this conservative
he economy of the design. From Specification
ary Table C-C2.2, $K = 1.0$ for both the X-X and

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$$P_{elx} = \frac{\pi^2 EI_x}{(KL)^2} \\ = \frac{\pi^2 (29,000 \text{ ksi}) (1,110 \text{ in.}^4)}{[1.0(14 \text{ ft})(12 \text{ in./ft})]^2} \\ = 11,300 \text{ kips}$$

$$P_{ely} = P_{elx} \frac{I_x}{I_y} = 11,300 \text{ kips} \left(\frac{402 \text{ in.}^4}{1,110 \text{ in.}^4} \right) = 4,080 \text{ kips}$$

Therefore,

$$B_{1x} = \frac{1.0}{1 - \frac{867 \text{ kips}}{11,300 \text{ kips}}} \geq 1.0 \\ = 1.08 \geq 1.0$$

$$B_{1y} = \frac{1.0}{1 - \frac{867 \text{ kips}}{4,080 \text{ kips}}} \geq 1.0 \\ = 1.27 \geq 1.0$$

$$M_u = B_1 M_{nt}$$

$$M_{ux} = 1.08(25.5 \text{ kip-ft}) = 27.5 \text{ kip-ft}$$

$$M_{uy} = 1.27(17.0 \text{ kip-ft}) = 21.6 \text{ kip-ft}$$

Check combined loading

Using Manual Table 6-1 for combined loading with $L_{by} = 14 \text{ ft}$,

$$p = 0.886 \times 10^{-3} \text{ kips}^{-1} \quad b_x = 1.38 \times 10^{-3} (\text{kip-ft})^{-1}$$

$$b_y = 2.85 \times 10^{-3} (\text{kip-ft})^{-1}$$

$$\frac{P_u}{\phi_c P_n} = p P_u = 0.886 \times 10^{-3} \text{ kips}^{-1} (867 \text{ kips}) = 0.768$$

$$\left(\frac{8}{9} \right) \left(\frac{M_{ux}}{\phi_b M_{nx}} \right) = b_x M_{ux} = 1.38 \times 10^{-3} (\text{kip-ft})^{-1} (27.5 \text{ kip-ft}) = 0.0380$$

$$\left(\frac{8}{9} \right) \left(\frac{M_{uy}}{\phi_b M_{ny}} \right) = b_y M_{uy} = 2.85 \times 10^{-3} (\text{kip-ft})^{-1} (21.6 \text{ kip-ft}) = 0.0616$$

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$$0.0380 + 0.0616$$

1.0 o.k.

the loads given for Column C-1 between

Connection Design

Design the connection between Brace BR-1 and beam outside of the link, and brace are 3.14 and 3.15, respectively.

$$\begin{aligned} A_g &= 32.9 \text{ in.}^2 & t_f &= 1.25 \text{ in.} \\ I_x &= 716 \text{ in.}^4 & k &= 1.75 \text{ in.} \\ d &= 16.5 \text{ in.} & t_f &= 0.760 \text{ in.} \end{aligned}$$

of the brace,

$$4 \text{ kips} \quad M_u = 189 \text{ kip-ft}$$

force

ed entirely by the flanges, the force in each

kips

be taken by the flanges, the force in each flange

$$\left[\frac{2 \text{ in./ft}}{0.25 \text{ in.}} \right] = 223 \text{ kips}$$

the flange is,

$$+ 223 \text{ kips} = 472 \text{ kips}$$

force

re shear force will be taken by the web.

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Design brace flange connection

From Example 3.14, the brace-to-beam connection was assumed to be as shown in Seismic Provisions Figure C-I-15.5. The brace must be detailed as fixed to the link in order for this assumption to be valid. Try a fully welded connection.

The axial force in the brace flanges is too large for a single-sided fillet weld to be economical. Additionally, due to the fact that the angle between the brace and the beam is less than 60° , there is not a prequalified double-sided fillet weld detail. Try a complete-joint-penetration groove weld to connect the brace flanges to the beam flange.

From Specification Table J2.5, the strength of the weld is based on the strength of the base material. The yield strength of each flange is,

$$\begin{aligned} \phi R_n &= 0.90 F_y b_f t_f \\ &= 0.90 (50 \text{ ksi}) (10.3 \text{ in.}) (1.25 \text{ in.}) \\ &= 579 \text{ kips} > 472 \text{ kips} \end{aligned}$$

$$\phi R_n > P_f \quad \text{o.k.}$$

Check concentrated forces at brace flange connection

The vertical component of the flange force is,

$$V_f = 472 \text{ kips} \left(\frac{12.5 \text{ ft}}{18 \text{ ft}} \right) = 328 \text{ kips}$$

The local yielding strength of the beam web at the brace flange connection is,

$$\begin{aligned} \phi R_n &= 1.0 (5k + N) F_{yw} t_w && \text{(Specification J10-2)} \\ &= 1.0 [5(1.75 \text{ in.}) + 1.25 \text{ in.}] (50 \text{ ksi}) (0.455 \text{ in.}) \\ &= 228 \text{ kips} < 328 \text{ kips} \end{aligned}$$

$$\phi R_n < V_f \quad \text{n.g.}$$

With the concentrated force applied at a distance from the beam end that is greater than $d/2$, the beam web crippling strength at the brace flange connection is,

$$\begin{aligned} R_n &= 0.80 t_w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E F_{yw} t_f}{t_w}} && \text{(Specification J10-4)} \\ R_n &= (0.80) (0.455)^2 \left[1 + 3 \left(\frac{1.25}{16.5} \right) \left(\frac{0.455}{0.760} \right)^{1.5} \right] \sqrt{\frac{29,000 (50) (0.760)}{0.455}} \\ &= 285 \text{ kips} \end{aligned}$$

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Assuming 1-in. corner clips on the stiffener, the length of stiffener adjacent to the beam web is,

$$\begin{aligned} L &= d - 2\left(t_f + \frac{3}{4} \text{ in.}\right) \\ &= 16.5 \text{ in.} - 2(0.760 \text{ in.} + 1 \text{ in.}) \\ &= 13.0 \text{ in.} \end{aligned}$$

The minimum single-sided fillet weld size required to transfer the stiffener force to the web is,

$$D_{\min} = \frac{57.0 \text{ kips}}{1.392 \text{ kips/in.} (13.0 \text{ in.})} = 3.15 \text{ sixteenths}$$

Use double-sided, $\frac{1}{4}$ -in. fillet welds to connect the stiffener to the beam flanges and single-sided, $\frac{1}{4}$ -in. fillet welds to connect the stiffener to the beam web.

Design the brace web connection

Try a $\frac{3}{8}$ -in. \times 4-in. \times 0-ft 6-in. single-plate connection with $\frac{5}{16}$ -in. fillet welds to connect the plate to the beam and brace. By inspection, this connection will be adequate to transfer the resultant load of 10.4 kips.

The final connection design and geometry is shown in Figure 3-29.

Example 3.19. EBF Brace-to-Beam/Column Connection Design

Given: Refer to Joint J-2 in Figure 3-27. Design the connection between brace, beam, and column. Use ASTM A572 Grade 50 ($F_y = 50$ ksi, $F_u = 65$ ksi) for all plate material and 70-ksi electrodes for all welds. Assume that the beam is as designed in Example 3.15, the brace size is the same as that determined in Example 3.16, and the column is as designed in Example 3.17. The Applicable Building Code specifies the use of ASCE 7 for calculation of loads.

From Example 3.15 for the design of the beam outside of the link,

W16 \times 77	$F_y = 50$ ksi	$F_u = 65$ ksi	$t_w = 0.455$ in.
	$b_f = 10.3$ in.	$t_f = 0.760$ in.	$d = 16.5$ in.
	$k = 1.47$ in.	$T = 13\frac{1}{4}$ in.	

From Example 3.16 for the design of the brace,

W10 \times 112	$F_y = 50$ ksi	$F_u = 65$ ksi	$T = 7\frac{1}{2}$ in.
	$t_w = 0.755$ in.	$A_g = 32.9$ in. ²	$d = 11.4$ in.
	$\bar{y}_{WT} = 1.21$ in.		

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the column,

$$F_u = 65 \text{ ksi} \quad t_w = 0.485 \text{ in.}$$

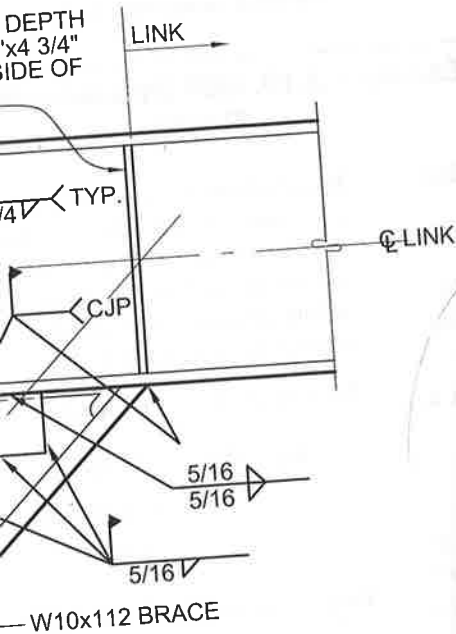
$$d = 14.2 \text{ in.} \quad k = 1.38 \text{ in.}$$

Category is D, $\Omega_o = 2.0$, and $S_{DS} = 1.0$.

ons should be examined.

velop the strain-hardened expected yield level must be transferred through the con- and beam outside of the link. Any additional e strain-hardened expected yield strength of be transferred through the beam-to-column d not exceed the amplified drag load. The e link must be transferred into the column.

be transferred into the beam outside of the ce required to develop the strain-hardened e link at the third level must be transferred



ion as designed in Example 3.18.

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through the connection and into the column and beam outside of the link. The brace force need not exceed that required to develop the strain-hardened expected yield strength of the link at the fourth level. The shear in the beam outside of the link must be transferred into the column.

Considering the load combinations given in ASCE 7, it was determined that the governing load combination for the design of the connection is,

$$1.2D + 1.0E + 0.5L_L + 0.2S$$

and the governing earthquake load case causes compression in the brace. Assume the connection forces are as shown in Figure 3-30.

Determine the required strength of the brace

Seismic Provisions Section 15.6 requires that the brace have sufficient strength to develop $1.25R_y V_n$ of the link. Using the overstrength factor method described

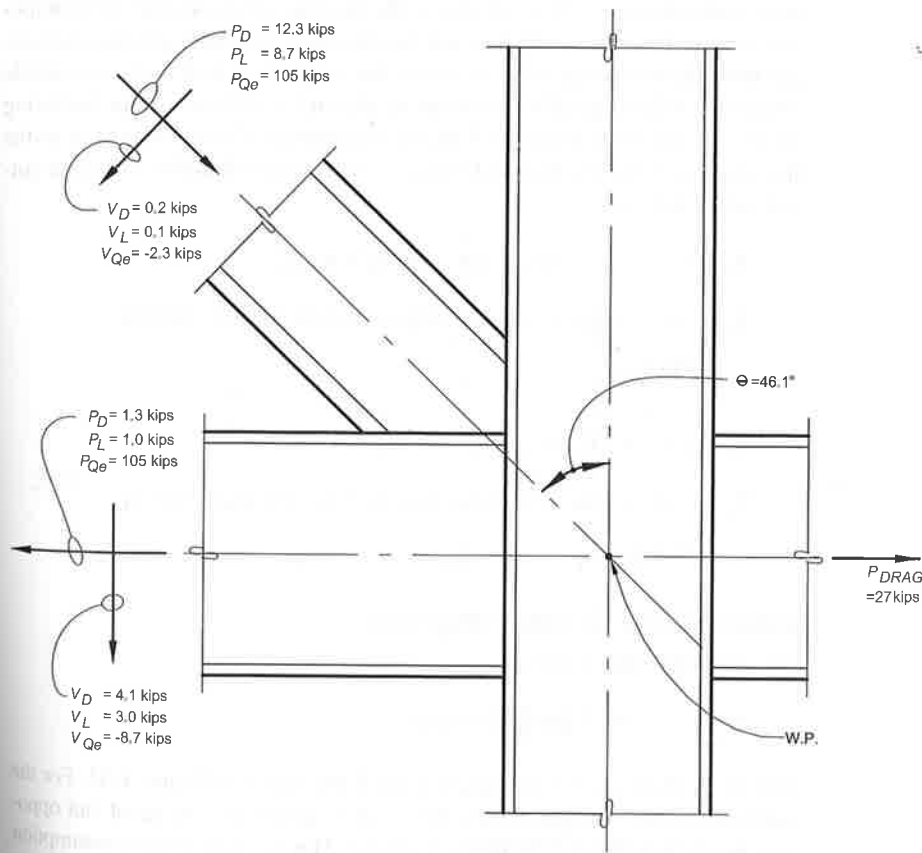


Figure 3-30. Connection forces for Example 3.19.

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or for the link shear was determined to
nection due to the brace are,

$$.5P_L + 0.2P_S$$

$$\text{ips}) + 0.5(8.7 \text{ kips}) + 0.2(0)$$

$$.5V_L + 0.2V_S$$

$$\text{ips}) + 0.5(0.1 \text{ kips}) + 0.2(0)$$

gth of the beam outside of the link
e of the link was designed to develop the
ngth of the link at the third level. The strain-
of the beam outside of the link was allowed
e to the presence of a concrete slab compo-
ed that the connection strength does not ben-
efore, the forces for which the beam outside
d to be adjusted to reflect a strain-hardening
6, the overstrength factor for the link at the
forces at the connection due to the beam out-

$$+ 0.5P_L + 0.2P_S$$

$$05 \text{ kips}) + 0.5(0.7 \text{ kips}) + 0.2(0)$$

$$E + 0.5V_L + 0.2V_S$$

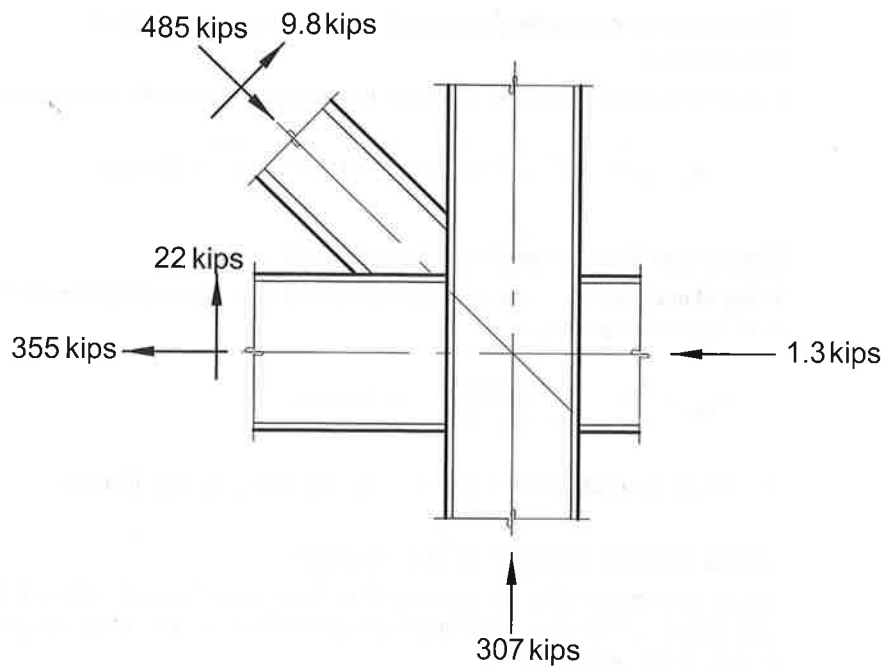
$$8.7 \text{ kips}) + 0.5(3.0 \text{ kips}) + 0.2(0)$$

drag force

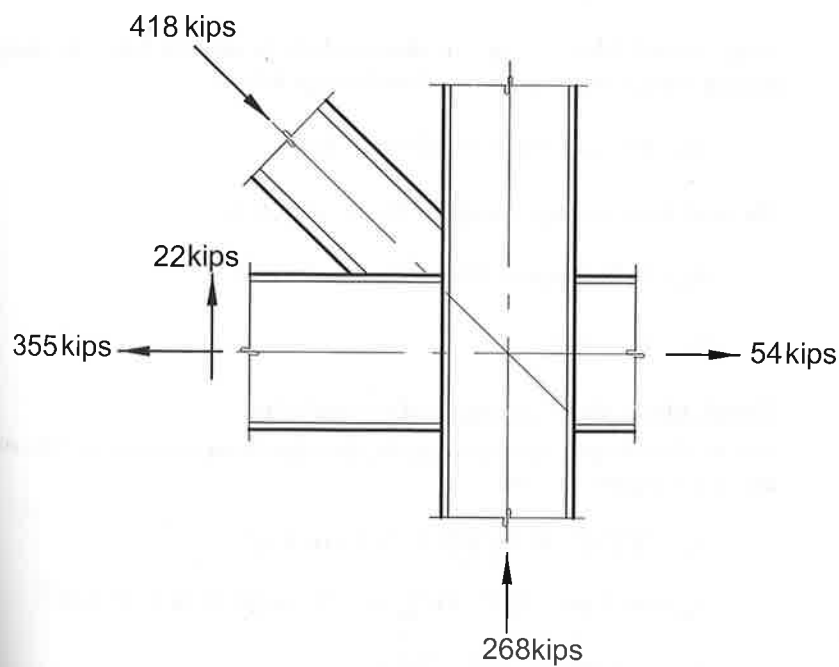
4.0 kips

itions 1 and 2 are shown in Figure 3-31. For the
se forces will be assumed to be equal, but oppo-
orce in tension. This is a conservative assumption
igned in this example. However, this will not be a
all connection geometries and loading conditions.

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(a) CONDITION 1



(b) CONDITION 2

Figure 3-31. Free-body diagrams for Example 3.19.

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h of the brace-to-gusset

ce, the resultant force on the connection is,

$$s)^2 + (10.5 \text{ kips})^2 = 485 \text{ kips}$$

er of bolts

m number of 1-in.-diameter ASTM A325X

= 6.86 bolts

3-in. spacing and 2-in. edge distance.

gusset plate

s equal to $\frac{3}{4}$ in. Using Manual Table 7-5 for
bles, the design bearing strength of the plate

= 84.8 kips

-diameter bolts in standard holes, the design
ch of the edge bolts is,

n.) = 64.4 kips

of the gusset plate is,

4.4 kips) = 638 kips

th of gusset plate

l to 3 in., the edge distance is equal to 2 in., and

$$(0.75 \text{ in.}) = 16.5 \text{ in.}^2$$

$$\left(1 \frac{1}{16} \text{ in.} + \frac{1}{16} \text{ in.}\right)(0.75 \text{ in.}) = 10.6 \text{ in.}^2$$

$$= 2.63 \text{ in.}^2$$

$$\text{in.} + \frac{1}{16} \text{ in.})(0.75 \text{ in.}) = 1.79 \text{ in.}^2$$

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$$F_u A_{nt} = 65 \text{ ksi} \left(1.79 \text{ in.}^2\right) = 116 \text{ kips}$$

$$0.6 F_u A_{nv} = 0.6 (65 \text{ ksi}) \left(10.6 \text{ in.}^2\right) = 413 \text{ kips}$$

$$0.6 F_y A_{gv} = 0.6 (50 \text{ ksi}) \left(16.5 \text{ in.}^2\right) = 495 \text{ kips}$$

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

$$\phi R_n = 0.75 [413 \text{ kips} + (1.0)(116 \text{ kips})] \quad (\text{Specification J4-5})$$

$$\leq 0.75 [495 \text{ kips} + (1.0)(116 \text{ kips})]$$

$$= 397 \text{ kips} < 459 \text{ kips}$$

$$R_u > \phi R_n \quad \text{n.g.}$$

Try changing the bolt spacing to $3 \frac{1}{2}$ in. and the edge distance to $2 \frac{1}{2}$ in.

$$A_{gv} = 2 \left[2 \frac{1}{2} \text{ in.} + 3 \left(3 \frac{1}{2} \text{ in.}\right)\right] (0.75 \text{ in.}) = 19.5 \text{ in.}^2$$

$$A_{nv} = 19.5 \text{ in.}^2 - 2 \left(3 \frac{1}{2}\right) \left(1 \frac{1}{16} \text{ in.} + \frac{1}{16} \text{ in.}\right) (0.75 \text{ in.}) = 13.6 \text{ in.}^2$$

$$0.6 F_u A_{nv} = 0.6 (65 \text{ ksi}) (13.6 \text{ in.}^2) = 530 \text{ kips}$$

$$0.6 F_y A_{gv} = 0.6 (50 \text{ ksi}) (19.5 \text{ in.}^2) = 585 \text{ kips}$$

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

(Specification J4-5)

$$\phi R_n = 0.75 [530 \text{ kips} + (1.0)(116 \text{ kips})]$$

$$\leq 0.75 [585 \text{ kips} + (1.0)(116 \text{ kips})]$$

$$= 485 \text{ kips} \leq 526 \text{ kips}$$

$$\phi R_n \geq R_u \quad \text{o.k.}$$

See Figure 3-32 for initial connection geometry.

Check the compression buckling strength of the gusset

As can be seen in Figure 3-32, the Whitmore section passes outside of the gusset and into the beam flange. Since the section does not pass into the beam web, the full width can be considered effective. The length of the Whitmore section is,

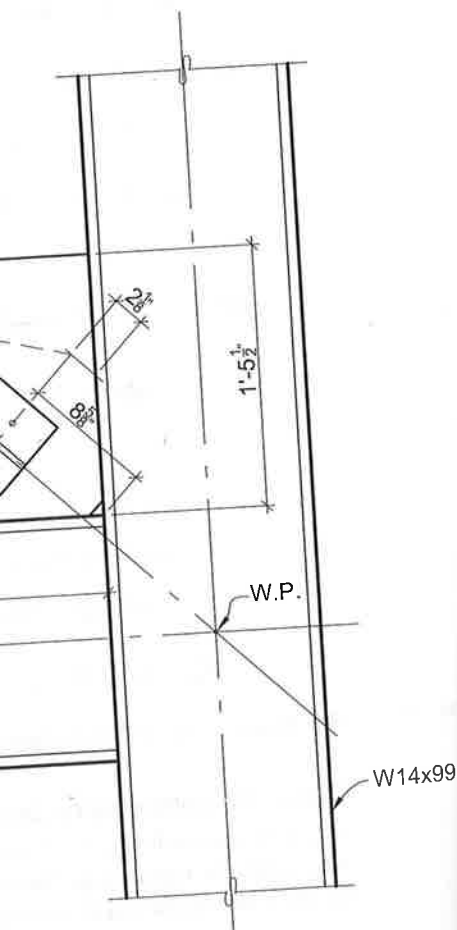
$$L_w = 3 \frac{1}{2} \text{ in.} + 2(3) \left(3 \frac{1}{2} \text{ in.}\right) \tan(30^\circ) = 15.6 \text{ in.}$$

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gusset plate is,

0.

0.



tion geometry for Example 3.19.

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From Specification Table 4-22, $\phi F_{cr} = 44.5$ ksi. The design strength of the gusset is,

$$\begin{aligned}\phi_c P_n &= \phi F_{cr} A_g \\ &= 44.5 \text{ ksi} (15.6 \text{ in.})(0.75 \text{ in.}) \\ &= 521 \text{ kips}\end{aligned}$$

$$\phi_c P_n > R_u \quad \text{o.k.}$$

Alternatively, Table 1-8 can be used. The effective length of the gusset in compression is,

$$KL = 0.5(5.38 \text{ in.}) = 2.69 \text{ in.}$$

Interpolating from Table 1-8 for a $3/8$ -in.-thick gusset with $KL = 2.69$ in., the compression buckling strength of the gusset is,

$$\phi R_n = \phi r_n L_w = 31.5 \text{ kips/in.}(15.6 \text{ in.}) = 491 \text{ kips}$$

Use a $3/4$ -in.-thick gusset plate.

Select trial connection between gusset and brace

Use a pair of bolted WT-sections to connect the brace to the gusset plate. The flange width of the WT-sections must be less than or equal to the T -dimension of the brace. Try (2) WT8x28.5.

$$\begin{array}{llll} d = 8.22 \text{ in.} & b_f = 7.12 \text{ in.} & A = 8.39 \text{ in.} & \bar{y} = 1.94 \text{ in.} \\ t_w = 0.430 \text{ in.} & t_f = 0.715 \text{ in.} & r_y = 1.60 \text{ in.} & \end{array}$$

$$b_{f_{WT}} = 7.12 \text{ in.} < T_{brace} = 7 \frac{1}{2} \text{ in.} \quad \text{o.k.}$$

Check tension yielding strength of WTs

The tension yielding strength of the WT-sections is,

$$\begin{aligned}\phi R_n &= 0.90 F_y A_g \\ &= 0.90 (50 \text{ ksi})(2)(8.39 \text{ in.}^2) \\ &= 755 \text{ kips}\end{aligned}$$

$$\phi R_n > R_u \quad \text{o.k.}$$

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