# Appendix XXI

# Moment Strength of the Pier Cap Beam of the Steel Girder Bridge

<u>West Bound Bridge</u>

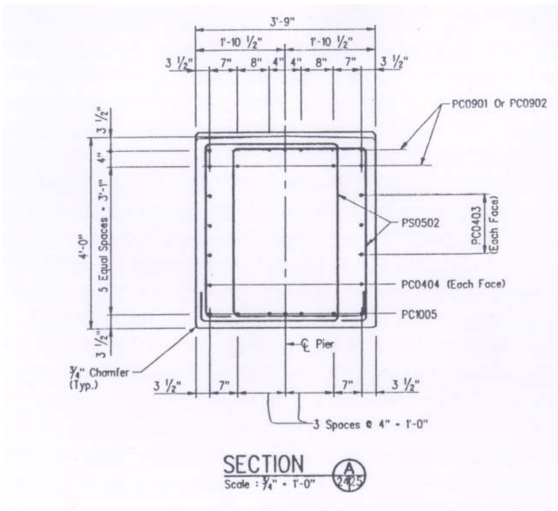


Figure XXI-1. The actual pier cap beam cross section [Brown, 1993]. The ¾" = 1'-0" scale is no longer correct.

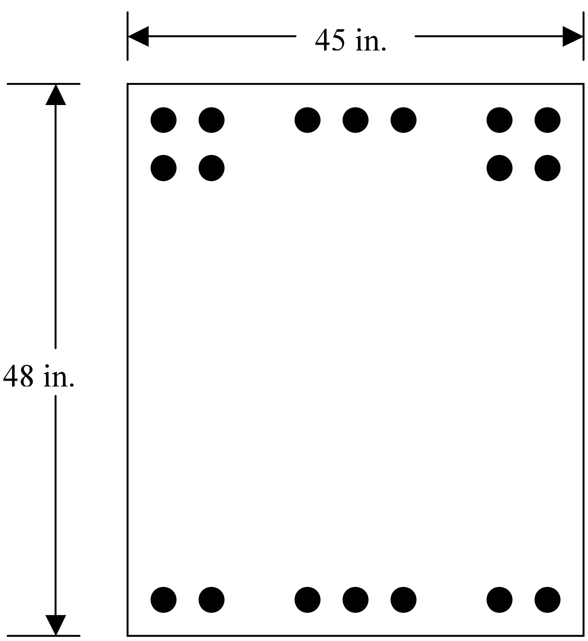


Figure XXI-2. The simplified pier cap beam cross section. This figure was not drawn to scale.

$f_c' = 3ksi$
$f_y = 60ksi$
$A_g = (48in.)(45in.) = 2{,}160in.^2$

Assume the compression steel will yield.

$A_{s1} = A_s' = 7(1.27in^2) = 8.89in.^2$
$A_{s2} = A_s - A_{s1} = (11)(1.00in.^2) - 8.89in.^2 = 2.11in.^2$
$Beam2:$

$$a = \frac{(A_s - A_s')f_y}{0.85f_c'b} = \frac{(2.11in.^2)(60ksi)}{0.85(3ksi)(45in.)} = 1.10in.$$

$d' = 3.5in.$

$$\frac{d'}{a} = \frac{3.5in.}{1.10in.} = 3.18$$

$$\left(\frac{d'}{a}\right)_{\lim} = \frac{1}{\beta_1}\left(1 - \frac{f_y}{87{,}000}\right) = \frac{1}{0.85}\left(1 - \frac{60{,}000}{87{,}000}\right) = 0.365$$

$$\frac{d'}{a} > \left(\frac{d'}{a}\right)_{\lim}$$

So the compression steel will not yield, the initial assumption was incorrect.

$$(0.85f_c'b)a^2 + (0.003E_sA_s' - A_sf_y)a - (0.003E_sA_s'\beta_1d') = 0$$
$$(0.85)(3)(45)a^2 + [(0.003)(29{,}000)(8.89) - (11.0)(60)]a - (0.003)(29{,}000)(8.89)(0.85)(3.5) = 0$$
$$114.75a^2 - 113.43a - 2300.95 = 0$$

$a = 5.00in.$

$$\frac{a}{d} = \frac{5.00in.}{43.05in.} = 0.116$$

$$\frac{a_b}{d} = \beta_1\left(\frac{87{,}000}{87{,}000 + f_y}\right) = 0.85\left(\frac{87{,}000}{87{,}000 + 60{,}000}\right) = 0.503$$

$$\frac{a}{d} < \frac{a_b}{d}$$

Thus the tension steel yields.

$$d_t = 3.5in. + 37in. + 4in.$$

$$\frac{a}{d_t} = \frac{a}{d} = \frac{5.00in.}{44.5in.} = 0.112 < \frac{a_{tcl}}{d_t} = 0.375\beta_1 = 0.375(0.85) = 0.31875$$

Thus the section is tension controlled, and $\phi = 0.90$.

$$A_{s,\min} = \frac{3\sqrt{f_c'}}{f_y} b_w d \geq \frac{200 b_w d}{f_y}$$

$$= \frac{3\sqrt{3000}}{60{,}000}(45)(43.05) \geq \frac{200(45)(43.05)}{60{,}000}$$

$$= 5.31in.^2 \geq 6.46in.^2$$

$$A_s = 11.0in.^2$$

$$A_s > A_{s,\min} = 6.46in.^2$$

$$C_c = 0.85 f_c' ba = 0.85(3ksi)(45in.)(5.00in.) = 573.75k$$

$$C_s = E_s A_s'\left(1 - \frac{\beta_1 d'}{a}\right)0.003$$

$$= (29{,}000ksi)(8.89in.^2)\left(1 - \frac{0.85(3.5in.)}{5.00in.}\right)(0.003)$$

$$= 313k$$

$$\phi M_n = \phi\left[C_c\left(d - \frac{a}{2}\right) + C_s(d - d')\right]$$

$$= 0.90[(573.75k)(43.05in. - 2.50in.) + (313.24k)(43.05in. - 3.50in.)]$$

$$= 32{,}100kips - in.$$

$$= 2{,}674kips - ft$$

This strength is larger than the ultimate moment (560 kips-ft) shown in Appendix XIX. Thus the pier cap beam is good enough to carry the ultimate load due to dead, live and earthquake loads.

## East Bound Bridge

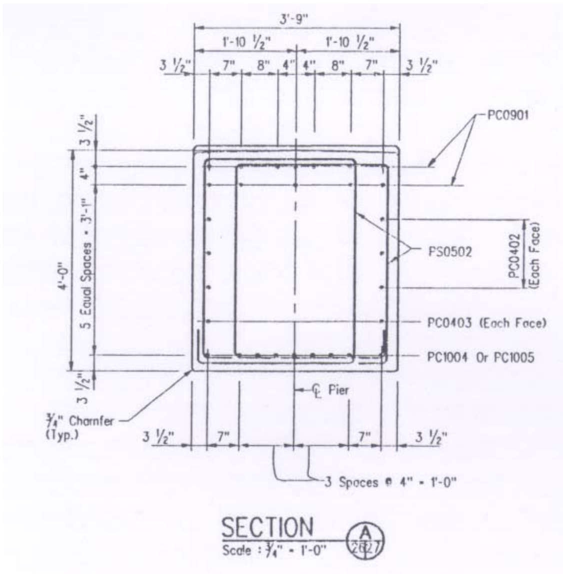


Figure XXI-3. The actual pier cap beam cross section [Brown, 1993]. The ¾" = 1'-0" scale is no longer correct.

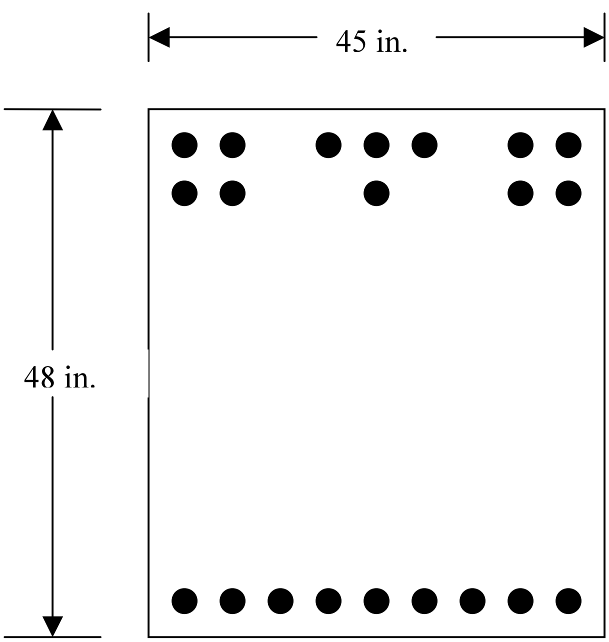


Figure XXI-4. The simplified pier cap beam cross section. This figure was not drawn to scale.

$f_c' = 3ksi$

$f_y = 60ksi$

$A_g = (48in.)(45in.) = 2{,}160in.^2$

Assume the compression steel will yield.

$A_{s1} = A_s' = 9(1.27in^2) = 11.43in.^2$

$A_{s2} = A_s - A_{s1} = (12)(1.00in.^2) - 11.43in.^2 = 0.57in.^2$

$Beam2:$

$$a = \frac{(A_s - A_s')f_y}{0.85 f_c' b} = \frac{(0.57in.^2)(60ksi)}{0.85(3ksi)(45in.)} = 0.298in.$$

$d' = 3.5in.$

$$\frac{d'}{a} = \frac{3.5in.}{0.298in.} = 11.7$$

$$\left(\frac{d'}{a}\right)_{\lim} = \frac{1}{\beta_1}\left(1 - \frac{f_y}{87{,}000}\right) = \frac{1}{0.85}\left(1 - \frac{60{,}000}{87{,}000}\right) = 0.365$$

$$\frac{d'}{a} > \left(\frac{d'}{a}\right)_{\lim}$$

So the compression steel will not yield, the initial assumption was incorrect.

$$(0.85 f_c' b)a^2 + (0.003E_s A_s' - A_s f_y)a - (0.003E_s A_s' \beta_1 d') = 0$$

$$(0.85)(3)(45)a^2 + [(0.003)(29{,}000)(11.43) - (12)(60)]a - (0.003)(29{,}000)(11.43)(0.85)(3.5) = 0$$

$$114.75a^2 - 274.41a - 2958.37 = 0$$

$a = 4.02in.$

$$\frac{a}{d} = \frac{4.02in.}{42.83in.} = 0.0939$$

$$\frac{a_b}{d} = \beta_1\left(\frac{87{,}000}{87{,}000 + f_y}\right) = 0.85\left(\frac{87{,}000}{87{,}000 + 60{,}000}\right) = 0.503$$

$$\frac{a}{d} < \frac{a_b}{d}$$

Thus the tension steel yields.

$d_t = 48in. - 3.5in. = 44.5in.$

$$\frac{a}{d_t} = \frac{a}{d} = \frac{4.02in.}{44.5in.} = 0.0903 < \frac{a_{tcl}}{d_t} = 0.375\beta_1 = 0.375(0.85) = 0.31875$$

Thus the section is tension controlled, and $\phi = 0.90$.

$$A_{s,\min} = \frac{3\sqrt{f_c'}}{f_y} b_w d \geq \frac{200 b_w d}{f_y}$$

$$= \frac{3\sqrt{3000}}{60{,}000}(45)(42.83) \geq \frac{200(45)(42.83)}{60{,}000}$$

$$= 5.28 in.^2 \geq 6.42 in.^2$$

$$A_s = 12.0 in.^2$$

$$A_s > A_{s,\min} = 6.42 in.^2$$

$$C_c = 0.85 f_c' ba = 0.85(3ksi)(45in.)(4.02in.) = 461.30k$$

$$C_s = E_s A_s' \left(1 - \frac{\beta_1 d'}{a}\right) 0.003$$

$$= (29{,}000ksi)(11.43in.^2)\left(1 - \frac{0.85(3.5in.)}{4.02in.}\right)(0.003)$$

$$= 258k$$

$$\phi M_n = \phi\left[C_c\left(d - \frac{a}{2}\right) + C_s(d - d')\right]$$

$$= 0.90[(461.30k)(42.83in. - 2.01in.) + (258.50k)(42.83in. - 3.50in.)]$$

$$= 26{,}100 kips - in.$$

$$= 2{,}174 kips - ft$$

This strength is larger than the ultimate moment (522 kips-ft) shown in Appendix XIX. Thus the pier cap beam is good enough to carry the ultimate load due to dead, live and earthquake loads.