Dimensions and Weights for Concrete Design

Footing width, w_{foot} = 9.50 ft = 114.00 in
Footing heel width, w_{heel} = 6.00 ft
Footing heel height, h_{heel} = 1.50 ft = 18.00 in

Footing toe width, w_{toe} = 2.00 ft
Footing toe height, h_{toe} = 1.50 ft = 18.00 in

Wall width at top, t_{wt} = 1.50 ft = 18.00 in
Wall width at base, t_{wb} = 1.50 ft = 18.00 in

Concrete strength, f_c' = 4.00 ksi
Rebar strength, f_y = 60.00 ksi
Steel mod. of elast., E_s = 29,000 ksi

Concrete weight, w_c = 0.150 kcf
Water weight, w_w = 0.062 kcf
Saturated soil weight, w_{ss} = 0.130 kcf
Buoyant soil weight, w_{sb} = 0.068 kcf

Height of wall, h_w = 5.88 ft top of heel to top of wall
Height of water, h_{water} = 0.00 ft top of heel to water line

Height of soil, h_s = 5.88 ft top of heel to ground line
Height of satur. soil, h_{ss} = 5.88 ft height of satur. soil above top of heel
Height of buoy. soil, h_{sb} = 0.00 ft height of buoy. soil above top of heel

Active pressure coeff., K_a = 0.280
LL surcharge soil ht., h_{LL} = 4.29 ft

Optional Collision Loading for Barrier on Top of Wall

Per ODOT comments for I-70/71, use the transverse loading of 54 kips for a TL-4 test level railing [AASHTO, Table A13.2-1] distributed over the retaining wall's joint spacing.

Collision Loading = y (y or n)
Joint Spacing = 24.43 ft
Barrier Height = 3.50 ft

Design Summary

Summary of Design Status

Design Item	Footing Heel	Footing Toe	Wall Stem
Shear	OK	OK	OK
Minimum Reinforcement	OK	OK	OK
Shrinkage & Temperature	OK	OK	OK
Crack Control	N/A	N/A	OK

For calculations related to each design item, see below.

Reinforcing Steel Summary

Footing:	Top transverse:	#8 bars at 12.00 in c/c
	Bottom transverse:	#4 bars at 12.00 in c/c
	Longitudinal:	#4 bars at 12.00 in c/c
Wall Stem:	Back face vertical:	#6 bars at 12.00 in c/c
	Front face vertical:	#4 bars at 12.00 in c/c
	Horizontal:	#4 bars at 12.00 in c/c

Design Footing for Shear [5.13.3.6]

Design footings to have adequate shear capacity without transverse reinforcement.

Determine d_v

Assume: #8 bars at 12.00 in c/c for the top transverse bars in the heel A_s = 0.79 in^2/ft 2.0 in cover
#4 bars at 12.00 in c/c for the bottom transverse bars in the toe A_s = 0.20 in^2/ft 3.0 in cover

For the heel:

$d_{sheel} = h - cover - d_{bar}/2$ = 18.00 - 2.00 - (1.000 / 2) = 15.50 in

$a_{heel} = \frac{A_s f_y}{0.85 f_c' b} = \frac{0.79 \times 60}{0.85 \times 4.0 \times 12}$ = 1.16 in

[5.8.2.9] $d_{vheel} = d_{sheel} - a/2$ = 15.50 - (1.16 / 2) = 14.92 in GOVERNS
or $d_{vheel} = 0.90\ d_e$ = 0.90 x 15.50 = 13.95 in
or $d_{vheel} = 0.72\ h$ = 0.72 x 18.00 = 12.96 in

For the toe:

$d_{stoe} = h - cover - d_{bar}/2$ = 18.00 - 3.00 - (0.500 / 2) = 14.75 in

$a_{toe} = \frac{A_s f_y}{0.85 f_c' b} = \frac{0.20 \times 60}{0.85 \times 4.0 \times 12}$ = 0.29 in

[5.8.2.9] $d_{vtoe} = d_{stoe} - a/2$ = 14.75 - (0.29 / 2) = 14.60 in GOVERNS
or $d_{vtoe} = 0.90\ d_e$ = 0.90 x 14.75 = 13.28 in

or d_{vtoe} = 0.72 h = 0.72 x 18.00 = 12.96 in

Check Heel for Shear

The critical shear section for the heel of the footing is located at the back face of the wall. The heel of the footing is assumed to carry its self weight and the rectangular soil block above it. This neglects the benefit of any upward soil pressure below the footing (conservative).

$V_u = (\gamma_{EV} w_{ss} h_{ss} + \gamma_{EV} w_{sb} h_{sb} + \gamma_{DC} w_c h_{heel} + \gamma_{LL} w_{ss} h_{LL} + \gamma_{WA} w_w h_{water}) \times w_{heel}$

V_u = (1.35 x 0.130 x 5.88 + 1.35 x 0.068 x 0.00 + 1.25 x 0.150 x 1.50 +
1.75 x 0.130 x 4.29 + 1.00 x 0.062 x 0.00) x 6.00 = 13.73 k/ft

[5.8.3.3] Using β = 2.00 and assuming bars in the top mat as above:

[5.8.3.4] $\phi V_c = \phi\, 0.0316\, \beta\, (f_c')^{0.5}\, b_v\, d_v$
[5.8.2.9]

ϕV_c = 0.90 x 0.0316 x 2.00 x (4.0)$^{0.50}$ x 12.00 x 14.92 = 20.37 k

20.37 k > 13.73 k OK

Check Toe for Shear

The peak bearing stress is 2.36 ksf for the Extreme IIb load case.

The critical section for the toe of the footing is at dv from the front face of the wall. For a quick simplified check, try applying the peak bearing stress over the entire length of the toe (conservative).

$V_u = \sigma_V w_{toe}$ = 2.36 x 2.00 = 4.72 k/ft

[5.8.3.3] Using β = 2.00 and assuming bars in the bottom mat as above:

[5.8.3.4] $\phi V_c = \phi\, 0.0316\, \beta\, (f_c')^{0.5}\, b_v\, d_v$
[5.8.2.9]

ϕV_c = 0.90 x 0.0316 x 2.00 x (4.0)$^{0.50}$ x 12.00 x 14.60 = 19.93 k

19.93 k > 4.72 k OK

Design Footing Reinforcement [5.13.3.4]

Each mat of reinforcement is checked to ensure that it has adequate capacity and that the maximum and minimum reinforcement checks are satisfied. The critical section for flexure in the footing is at the face of the wall.

Top Transverse Reinforcement

From the shear check of the heel, V_u = 13.73 k/ft

$M_u = V_u \times (w_{heel} / 2)$ = 13.73 x (6.00 / 2) = 41.20 k-ft

Set up the equation to solve for the required steel area:

$$M_u = \phi M_n = \phi A_s f_y (d_s - a/2) = \phi A_s f_y \left(d_s - \frac{A_s f_y}{1.7 f_c' b} \right)$$

$$M_u = 0.90 \times A_s \times 60 \left(ds - \frac{As \times 60}{1.7 \times 4.0 \times 12} \right) \times \left(\frac{1}{12} \right)$$

$$3.309\ A_s^2 - 4.50\ d_s A_s + M_u = 0$$

For the reinforcing steel assumed for the heel, d_s = 15.50 in

Substituting and solving for A_s, it is found that required A_s = 0.61 in^2/ft

Try: #8 bars at 12.00 in c/c for the top transverse bars in the heel As = 0.79 in^2/ft

Check Minimum Reinforcement [5.7.3.3.2]

Determine the cracking moment:

$f_r = 0.24\,(f_c')^{0.5}$ = 0.24 x (4.0)$^{0.50}$ = 0.48 ksi

$I_g = (1/12)\,b\,h^3$ = 0.0833 x 12.00 x (18.00)^3 = 5832.0 in^4

$y_t = (1/2)\,h$ = 0.5000 x 18.00 = 9.00 in

$M_{CR} = \gamma_1\,\gamma_3\,f_r\,I_g\,/\,y_t$ = 1.60 x 0.67 x 0.48 x 5832.0 / (9.00 x 12.00) = 27.79 k-ft

The capacity of the section must be greater than or equal to the smaller of:

M_{CR} = 27.79 = 27.79 k-ft <u>GOVERNS</u>
(4/3) M_u = 1.33 x 41.20 = 54.94 k-ft

The capacity of the top mat of reinforcement is:

$M_r = \phi\,A_s\,f_y\,(d_s - a/2)$

For the reinforcing steel used, d_s = 18.00 - 2.00 - (1.000 / 2) = 15.50 in

$$M_r = 0.90 \times 0.79 \times 60 \times \left(15.50 - \frac{0.79 \times 60}{1.7 \times 4.0 \times 12}\right) \times \left(\frac{1}{12}\right) = 53.04 \text{ k-ft}$$

53.04 k-ft > 27.79 k-ft OK

Check minimum reinforcement for temperature and shrinkage (5.10.8) A_s = 0.79 in^2/ft > 0.17 in^2/ft OK

Use #8 bars at 12.00 in c/c for top transverse reinforcement in the footing.

Bottom Transverse Reinforcement

From the shear check of the toe, V_u = 4.72 k/ft

$M_u = V_u \times (w_{toe} / 2)$ = 4.72 x (2.00 / 2) = 4.72 k-ft

Set up the equation to solve for the required steel area and again use:

3.309 A_s^2 - 4.50 $d_s\,A_s + M_u = 0$

For the reinforcing steel assumed for the heel, d_s = 14.75 in

Substituting and solving for A_s, it is found that required A_s = 0.07 in^2/ft

Try: #4 bars at 12.00 in c/c for the bottom transverse bars in the toe As = 0.20 in^2/ft

Check Minimum Reinforcement [5.7.3.3.2]

Determine the cracking moment:

$f_r = 0.24\,(f_c')^{0.5}$ = 0.24 x (4.0)$^{0.50}$ = 0.48 ksi

$I_g = (1/12)\,b\,h^3$ = 0.0833 x 12.00 x (18.00)^3 = 5832.0 in^4

$y_t = (1/2)\,h$ = 0.5000 x 18.00 = 9.00 in

$M_{CR} = \gamma_1\,\gamma_3\,f_r\,I_g\,/\,y_t$ = 1.60 x 0.67 x 0.48 x 5832.0 / (9.00 x 12.00) = 27.79 k-ft

The capacity of the section must be greater than or equal to the smaller of:

M_{CR} = 27.79 = 27.79 k-ft
(4/3) M_u = 1.33 x 4.72 = 6.29 k-ft <u>GOVERNS</u>

The capacity of the bottom mat of reinforcement is:

$M_r = \phi\,A_s\,f_y\,(d_s - a/2)$

For the reinforcing steel used, d_s = 18.00 - 3.00 - (0.500 / 2) = 14.75 in

$$M_r = 0.90 \times 0.20 \times 60 \times \left(14.75 - \frac{0.20 \times 60}{1.7 \times 4.0 \times 12}\right) \times \left(\frac{1}{12}\right) = 13.14 \text{ k-ft}$$

13.14 k-ft > 6.29 k-ft OK

Check minimum reinforcement for temperature and shrinkage (5.10.8) A_s = 0.20 in^2/ft > 0.17 in^2/ft OK

Use #4 bars at 12.00 in c/c for bottom transverse reinforcement in the footing.

Longitudinal Reinforcement [5.10.8]

Provide longitudinal reinforcement in the footing based on shrinkage and temperature requirements.

$h_{max} = max(h_{heel}, h_{toe})$ = 18.00 in

$$\text{Min. } A_s = \frac{1.30\, w_{foot}\, h_{max}}{2\,(w_{foot} + h_{max})\, f_y} = \frac{1.30 \times 114.00 \times 18.00}{2 \times (114.00 + 18.00) \times 60} = 0.17 \text{ in}^2/\text{ft}$$

The maximum spacing of reinforcement is:

$h_{min} = min(h_{heel}, h_{toe})$ = 18.00 in

Max. spacing = 3 h_{min} = 54.00 in
or Max. spacing = 18 in = 18.00 in
or Max. spacing = 12 in (for walls and footings ≥ 18 in thick) = 12.00 in GOVERNS

Try: #4 bars at 12.00 in c/c for the top and bottom longitudinal bars A_s = 0.20 in^2/ft > 0.17 in^2/ft OK

0.11 < A_s = 0.20 in^2/ft < 0.60 OK

spacing = 12.00 in = 12.00 in OK

Use #4 bars at 12.00 in c/c for top and bottom longitudinal reinforcement in the footing.

Determine Loads for Wall Stem Design

The loads on the stem at the top of the footing are determined to arrive at the design forces for the wall.

Saturated Earth Pressure:

$P_{EH(S)} = (1/2)\, w_{ss}\, K_a\, h_{ss}^2$ = 0.5 x 0.130 x 0.280 x (5.88)^2 = 0.63 k

$M_{EH(S)} = P_{EH(S)} \times [\,(1/3)\, h_{ss} + h_{sb}\,]$ = 0.63 x [0.333 x 5.88 + 0.00] = 1.23 k-ft

Buoyant Earth Pressure:

$P_{EH(B)} = K_a\, h_{sb}\, [w_{ss}\, h_{ss} + (1/2)\, w_{sb}\, h_{sb}]$

$P_{EH(B)}$ = 0.280 x 0.00 x [0.130 x 5.88 + 0.5 x 0.068 x 0.00] = 0.00 k

$y_B = [\, h_{sb}\, (\, w_{ss}\, h_{ss} + (1/3)\, w_{sb}\, h_{sb}\,)\,] / (2\, w_{ss}\, h_{ss} + w_{sb}\, h_{sb}\,)$

y_B = [0.00 x (0.130 x 5.88 + 0.333 x 0.068 x 0.00)]
/ (2.0 x 0.130 x 5.88 + 0.068 x 0.00) = 0.00 ft

$M_{EH(B)} = P_{EH(B)} \times y_B$ = 0.00 x 0.00 = 0.00 k-ft

Water Pressure:

$P_{WA} = (1/2)\, w_w\, h_{water}^2$ = 0.5 x 0.062 x (0.00)^2 = 0.00 k

$M_{WA} = P_{WA} \times (1/3)\, h_{water}$ = 0.00 x 0.333 x 0.00 = 0.00 k-ft

Live Load Surcharge:

$P_{LS} = w_{ss}\, K_a\, h_{LL}\, h_s$ = 0.130 x 0.280 x 4.29 x 5.88 = 0.92 k

$M_{LS} = P_{LS}$ x (1/2) h_s = 0.92 x 0.500 x 5.88 = 2.70 k-ft

Collision Load at Top of Parapet:

Use a Live Load of 2210 lbs/ft applied at h_r = 3.5 ft above the top of the wall.

P_{CT} = 2.21 k

$M_{CT} = P_{CT}$ x ($h_w + h_r$) = 2.21 x (5.88 + 3.50) = 20.73 k-ft

Using the Strength I load combination, the factored design forces for the wall stem are:

$H_u = 1.50 (P_{EH(S)} + P_{EH(B)}) + 1.00 P_{WA} + 1.75 (P_{LS} + P_{LL})$

H_u = 1.50 x (0.63 + 0.00) + 1.00 x 0.00 + 1.75 x (0.92 + 0.00) = 2.55 k

$M_u = 1.50 (M_{EH(S)} + M_{EH(B)}) + 1.00 M_{WA} + 1.75 (M_{LS} + M_{LL})$

M_u = 1.50 x (1.23 + 0.00) + 1.00 x 0.00 + 1.75 x (2.70 + 0.00) = 6.57 k-ft

The Extreme Event II design forces for the wall stem are:

$H_{ext} = 1.50 (P_{EH(S)} + P_{EH(B)}) + 1.00 P_{CT} + 0.50 (P_{LS} + P_{LL})$

H_{ext} = 1.50 x (0.63 + 0.00) + 1.00 x 2.21 + 0.50 x (0.92 + 0.00) = 3.61 k

$M_{ext} = 1.50 (M_{EH(S)} + M_{EH(B)}) + 1.00 M_{CT} + 0.50 (M_{LS} + M_{LL})$

M_{ext} = 1.50 x (1.23 + 0.00) + 1.00 x 20.73 + 0.50 x (2.70 + 0.00) = 23.93 k-ft

The service design forces for the wall stem are:

$H_{serv} = 1.00 (P_{EH(S)} + P_{EH(B)}) + 1.00 P_{WA} + 1.00 (P_{LS} + P_{LL})$

H_{serv} = 1.00 x (0.63 + 0.00) + 1.00 x 0.00 + 1.00 x (0.92 + 0.00) = 1.55 k

$M_{serv} = 1.00 (M_{EH(S)} + M_{EH(B)}) + 1.00 M_{WA} + 1.00 (M_{LS} + M_{LL})$

M_{serv} = 1.00 x (1.23 + 0.00) + 1.00 x 0.00 + 1.00 x (2.70 + 0.00) = 3.93 k-ft

Wall Stem Design - Investigate Shear

Shear typically does not govern the design of retaining walls. If shear does become an issue, the thickness of the stem should be increased such that transverse reinforcement is not required.

Ignoring the benefits of the shear key and axial compression, the shear capacity of the stem can be shown to be greater than that required.

$V_n = V_c + V_s + V_p$ [5.8.3.3-1]

Recognizing that V_s and V_p are zero

$V_n = V_c$

$V_c = 0.0316\, \beta\, (f_c')^{0.5}\, b_v\, d_v$ [5.8.3.3-3]

The maximum effective shear depth is:

For 2.0 inch clear cover and #6 bars, d_s = 18.00 - 2.00 - (0.750 / 2) = 15.63 in

$d_v = 0.9\, d_e = 0.9\, d_s$ = 0.90 x 15.63 = 14.06 in GOVERNS
$d_v = 0.72\, h = 0.72\, t_{wb}$ = 0.72 x 18.00 = 12.96 in

Follow the General Procedure using the provisions of Appendix B5, as per Section [5.8.3.4.2]:

β = 2.011897 ε_s= 0.00188 s_{xe}= 11.42945 Max Aggregate Size = 1 in (BDM S5.10.3.1.1)

ϕV_c = 0.90 x 0.0316 x 2.01 x (4.0)$^{0.50}$ x 12.00 x 14.06 = 19.31 k

19.31 k > 3.61 k OK

Wall Stem Design - Investigate Strength Limit State

Determine the area of back-face flexural reinforcement necessary to satisfy the design moment:

M_u = 23.93 k-ft

Again use the equation:

3.309 A_s^2 - 4.50 $d_s A_s + M_u = 0$

Substituting and solving for A_s, it is found that required A_s = 0.35 in^2/ft

Try: #6 bars at 12.00 in c/c A_s = 0.44 in^2/ft > 0.35 in^2/ft OK

Wall Stem Design - Investigate Service Limit State [5.7.3.4]

Check the crack control equations to ensure that the primary flexural reinforcement is well distributed. The service load bending moment is:

M_{serv} = 3.93 k-ft

Check the modulus of rupture for concrete:

$\phi f_r = \phi\, 0.24\, (f_c')^{0.5}$ = 0.80 x 0.24 x (4.0)$^{0.50}$ = 0.384 ksi

$S = \dfrac{b\, t_{wb}^2}{6} = \dfrac{12.00 \times (18.00)^2}{6}$ = 648.00 in^3

$f_{act} = \dfrac{M_{serv}}{S} = \dfrac{3.93 \times 12}{648.00}$ = 0.073 ksi < 0.384 ksi OK

If modulus of rupture check is "NG", then check the spacing of the reinforcement. First, determine the modular ratio:

$E_c = 33{,}000\, w_c^{1.5}\, (f_c')^{0.5}$ = 33,000 x (0.150)$^{1.5}$ x (4.00)$^{0.5}$ = 3,834 ksi

$n = E_s/E_c$ = 29,000 / 3,834 = 7.56 , use n = 8.00

For 2.0 inch clear cover and #6 bars, d_s = 18.00 - 2.00 - (0.750 / 2) = 15.63 in

Determine the location of the neutral axis:

$0.5\, b\, x^2 = n\, A_s\, (d_s - x)$

$0.5\,(12)\,x^2$ = 8 x 0.44 x (15.63 - x)

solving, x = 2.75 in

Check the spacing of the reinforcement to control cracking:

$y = d_s - x$ = 15.63 - 2.75 = 12.88 in

$I_{cr} = \dfrac{b\,x^3}{3} + n\,A_s\,(d_s - x)^2$

$I_{cr} = \dfrac{12.00 \times (2.75)^3}{3}$ + 8 x 0.44 x (15.63 - 2.75)^2 = 666.68 in^4

γ_e = 1.00

For 2.0 inch clear cover and #6 bars, d_c = 2.00 + (0.750 / 2) = 2.38 in

$\beta_s = 1.0 + \dfrac{d_c}{0.7\,(t_{wb} - d_c)} = 1.0 + \dfrac{2.38}{0.7 \times (18.00 - 2.38)}$ = 1.22

$f_{ss} = n\,\dfrac{M_{serv}\,y}{I_{cr}} = 8 \times \dfrac{3.93 \times 12.88 \times 12.00}{666.68}$ = 7.29 ksi

$s_{max} = \dfrac{700\,\gamma_e}{\beta_s f_{ss}} - 2.0\,d_c = \dfrac{700 \times 1.00}{1.22 \times 7.29} - 2.0 \times 2.38 = 74.12$ in

$s_{max} =$ 74.12 in > 12.00 in OK

Wall Stem Design - Check Reinforcement Limits

Check Minimum Reinforcement [5.7.3.3.2]

Determine the cracking moment:

$f_r = 0.24\,(f_c')^{0.5} = 0.24 \times (4.0)^{0.50} = 0.48$ ksi

$I_g = (1/12)\,b\,h^3 = 0.0833 \times 12.00 \times (18.00)^3 = 5832.0$ in^4

$y_t = (1/2)\,h = 0.5000 \times 18.00 = 9.00$ in

$M_{CR} = \gamma_1\,\gamma_3\,f_r\,I_g / y_t = 0.67 \times 1.6 \times 0.48 \times 5832.0\,/\,(9.00 \times 12.00) = 27.79$ k-ft

The capacity of the section must be greater than or equal to the smaller of:

M_{CR} = 27.79 = 27.79 k-ft GOVERNS
$(4/3)\,M_u$ = 1.33 x 23.93 = 31.91 k-ft

The capacity of the reinforcement is:

$M_r = \phi\,A_s\,f_y\,(d_s - a/2)$

$M_r = 0.90 \times 0.44 \times 60 \times \left(15.63 - \dfrac{0.44 \times 60}{1.7 \times 4.0 \times 12}\right) \times \left(\dfrac{1}{12}\right) = 30.30$ k-ft

30.30 k-ft > 27.79 k-ft OK

Check minimum reinforcement for temperature and shrinkage (5.10.8) $A_s =$ 0.44 in^2/ft > 0.16 in^2/ft OK

Use #6 bars at 12.00 in c/c for wall stem back face vertical reinforcing.

Wall Stem Design - Shrinkage and Temperature Reinforcement [5.10.8]

A minimum amount of reinforcement should be placed near each face of concrete elements to limit the size of cracks associated with concrete shrinkage and temperature changes.

$h_{max} = \max(t_{wt}, t_{wb}) = 18.00$ in

$h_w =$ 5.88 ft = 70.56 in

$\text{Min. } A_s = \dfrac{1.30\,h_w\,h_{max}}{2\,(h_w + h_{max})\,f_y} = \dfrac{1.30 \times 70.56 \times 18.00}{2 \times (70.56 + 18.00) \times 60} = 0.16$ in^2/ft

The maximum spacing of reinforcement is:

$h_{min} = \min(t_{wt}, t_{wb}) = 18.00$ in

Max. spacing = 3 h_{min} = 54.00 in
or Max. spacing = 18 in = 18.00 in
or Max. spacing = 12 in (for walls and footings ≥ 18 in thick) = 12.00 in GOVERNS

Try: #4 bars at 12.00 in c/c

$A_s =$ 0.20 in^2/ft > 0.16 in^2/ft OK

0.11 < $A_s =$ 0.20 in^2/ft < 0.60 OK

spacing = 12.00 in = 12.00 in OK

Use #4 bars at 12.00 in c/c for wall stem front face reinforcing and back face horizontal reinforcing.

Dimensions and Weights for Concrete Design

Footing width, w_{foot} =	9.50	ft	=	114.00	in	Concrete weight, w_c =	0.150	kcf	
						Water weight, w_w =	0.062	kcf	
Footing heel width, w_{heel} =	6.00	ft				Saturated soil weight, w_{ss} =	0.130	kcf	
Footing heel height, h_{heel} =	1.50	ft	=	18.00	in	Buoyant soil weight, w_{sb} =	0.068	kcf	
Footing toe width, w_{toe} =	2.00	ft				Height of wall, h_w =	6.92	ft	top of heel to top of wall
Footing toe height, h_{toe} =	1.50	ft	=	18.00	in	Height of water, h_{water} =	0.00	ft	top of heel to water line
Wall width at top, t_{wt} =	1.50	ft	=	18.00	in	Height of soil, h_s =	6.92	ft	top of heel to ground line
Wall width at base, t_{wb} =	1.50	ft	=	18.00	in	Height of satur. soil, h_{ss} =	6.92	ft	height of satur. soil above top of heel
						Height of buoy. soil, h_{sb} =	0.00	ft	height of buoy. soil above top of heel
Concrete strength, f_c' =	4.00	ksi							
Rebar strength, f_y =	60.00	ksi				Active pressure coeff., K_a =	0.280		
Steel mod. of elast., E_s =	29,000	ksi				LL surcharge soil ht., h_{LL} =	3.97	ft	

Optional Collision Loading for Barrier on Top of Wall

Per ODOT comments for I-70/71, use the transverse loading of 54 kips for a TL-4 test level railing [AASHTO, Table A13.2-1] distributed over the retaining wall's joint spacing.

Collision Loading =	y	(y or n)
Joint Spacing =	28.00	ft
Barrier Height =	3.50	ft

Design Summary

Summary of Design Status

Design Item	Footing		Wall
	Heel	Toe	Stem
Shear	OK	OK	OK
Minimum Reinforcement	OK	OK	OK
Shrinkage & Temperature	OK	OK	OK
Crack Control	N/A	N/A	OK

For calculations related to each design item, see below.

Reinforcing Steel Summary

Footing:	Top transverse:	#8 bars at 12.00 in c/c
	Bottom transverse:	#4 bars at 12.00 in c/c
	Longitudinal:	#4 bars at 12.00 in c/c
Wall Stem:	Back face vertical:	#6 bars at 12.00 in c/c
	Front face vertical:	#4 bars at 12.00 in c/c
	Horizontal:	#4 bars at 12.00 in c/c

Design Footing for Shear [5.13.3.6]

Design footings to have adequate shear capacity without transverse reinforcement.

Determine d_v

Assume: #8 bars at 12.00 in c/c for the top transverse bars in the heel A_s = 0.79 in^2/ft 2.0 in cover

#4 bars at 12.00 in c/c for the bottom transverse bars in the toe A_s = 0.20 in^2/ft 3.0 in cover

For the heel:

$d_{sheel} = h - cover - d_{bar} / 2 = 18.00 - 2.00 - (1.000 / 2) = 15.50$ in

$a_{heel} = \frac{A_s f_y}{0.85 f_c' b} = \frac{0.79 \times 60}{0.85 \times 4.0 \times 12} = 1.16$ in

[5.8.2.9] $d_{vheel} = d_{sheel} - a / 2 = 15.50 - (1.16 / 2) = 14.92$ in GOVERNS

or $d_{vheel} = 0.90 d_e = 0.90 \times 15.50 = 13.95$ in

or $d_{vheel} = 0.72 h = 0.72 \times 18.00 = 12.96$ in

For the toe:

$d_{stoe} = h - cover - d_{bar} / 2 = 18.00 - 3.00 - (0.500 / 2) = 14.75$ in

$a_{toe} = \frac{A_s f_y}{0.85 f_c' b} = \frac{0.20 \times 60}{0.85 \times 4.0 \times 12} = 0.29$ in

[5.8.2.9] $d_{vtoe} = d_{stoe} - a / 2 = 14.75 - (0.29 / 2) = 14.60$ in GOVERNS

or $d_{vtoe} = 0.90 d_e = 0.90 \times 14.75 = 13.28$ in or $d_{vtoe} = 0.72\ h$ = 0.72 x 18.00 = 12.96 in

Check Heel for Shear

The critical shear section for the heel of the footing is located at the back face of the wall. The heel of the footing is assumed to carry its self weight and the rectangular soil block above it. This neglects the benefit of any upward soil pressure below the footing (conservative).

$V_u = (\gamma_{EV}\, w_{ss}\, h_{ss} + \gamma_{EV}\, w_{sb}\, h_{sb} + \gamma_{DC}\, w_c\, h_{heel} + \gamma_{LL}\, w_{ss}\, h_{LL} + \gamma_{WA}\, w_w\, h_{water}) \times w_{heel}$

V_u = (1.35 x 0.130 x 6.92 + 1.35 x 0.068 x 0.00 + 1.25 x 0.150 x 1.50 +
1.75 x 0.130 x 3.97 + 1.00 x 0.062 x 0.00) x 6.00 = 14.39 k/ft

[5.8.3.3] Using β = 2.00 and assuming bars in the top mat as above:

[5.8.3.4] $\phi V_c = \phi\ 0.0316\ \beta\ (f_c')^{0.5}\ b_v\ d_v$

[5.8.2.9]

ϕV_c = 0.90 x 0.0316 x 2.00 x (4.0)$^{0.50}$ x 12.00 x 14.92 = 20.37 k

20.37 k > 14.39 k OK

Check Toe for Shear

The peak bearing stress is 2.27 ksf for the Extreme IIb load case.

The critical section for the toe of the footing is at dv from the front face of the wall. For a quick simplified check, try applying the peak bearing stress over the entire length of the toe (conservative).

$V_u = \sigma_V\, w_{toe}$ = 2.27 x 2.00 = 4.54 k/ft

[5.8.3.3] Using β = 2.00 and assuming bars in the bottom mat as above:

[5.8.3.4] $\phi V_c = \phi\ 0.0316\ \beta\ (f_c')^{0.5}\ b_v\ d_v$

[5.8.2.9]

ϕV_c = 0.90 x 0.0316 x 2.00 x (4.0)$^{0.50}$ x 12.00 x 14.60 = 19.93 k

19.93 k > 4.54 k OK

Design Footing Reinforcement [5.13.3.4]

Each mat of reinforcement is checked to ensure that it has adequate capacity and that the maximum and minimum reinforcement checks are satisfied. The critical section for flexure in the footing is at the face of the wall.

Top Transverse Reinforcement

From the shear check of the heel, V_u = 14.39 k/ft

$M_u = V_u \times (w_{heel} / 2)$ = 14.39 x (6.00 / 2) = 43.18 k-ft

Set up the equation to solve for the required steel area:

$$M_u = \phi M_n = \phi\, A_s\, f_y\, (d_s - a/2) = \phi\, A_s\, f_y \left(d_s - \frac{A_s\, f_y}{1.7\, f_c'\, b} \right)$$

$$M_u = 0.90 \times A_s \times 60 \left(ds - \frac{As \times 60}{1.7 \times 4.0 \times 12} \right) \times \left(\frac{1}{12} \right)$$

$$3.309\ A_s^2 - 4.50\ d_s\, A_s + M_u = 0$$

For the reinforcing steel assumed for the heel, d_s = 15.50 in

Substituting and solving for A_s, it is found that required A_s = 0.64 in^2/ft

Try: #8 bars at 12.00 in c/c for the top transverse bars in the heel As = 0.79 in^2/ft

Check Minimum Reinforcement [5.7.3.3.2]

Determine the cracking moment:

$f_r = 0.24\ (f_c')^{0.5}$ = 0.24 x (4.0)$^{0.50}$ = 0.48 ksi

$I_g = (1/12)\ b\ h^3$ = 0.0833 x 12.00 x (18.00)^3 = 5832.0 in^4

$y_t = (1/2)\ h$ = 0.5000 x 18.00 = 9.00 in

$M_{CR} = \gamma_1\ \gamma_3\ f_r\ I_g\ /\ y_t$ = 1.60 x 0.67 x 0.48 x 5832.0 / (9.00 x 12.00) = 27.79 k-ft

The capacity of the section must be greater than or equal to the smaller of:

M_{CR} = 27.79 = 27.79 k-ft <u>GOVERNS</u>
(4/3) M_u = 1.33 x 43.18 = 57.57 k-ft

The capacity of the top mat of reinforcement is:

$M_r = \phi\ A_s\ f_y\ (d_s - a/2)$

For the reinforcing steel used, d_s = 18.00 - 2.00 - (1.000 / 2) = 15.50 in

$$M_r = 0.90 \times 0.79 \times 60 \times \left(15.50 - \frac{0.79 \times 60}{1.7 \times 4.0 \times 12}\right) \times \left(\frac{1}{12}\right) = 53.04 \text{ k-ft}$$

53.04 k-ft > 27.79 k-ft OK

Check minimum reinforcement for temperature and shrinkage (5.10.8) A_s = 0.79 in^2/ft > 0.17 in^2/ft OK

<u>Use #8 bars at 12.00 in c/c for top transverse reinforcement in the footing.</u>

Bottom Transverse Reinforcement

From the shear check of the toe, V_u = 4.54 k/ft

$M_u = V_u \times (w_{toe} / 2)$ = 4.54 x (2.00 / 2) = 4.54 k-ft

Set up the equation to solve for the required steel area and again use:

3.309 A_s^2 - 4.50 $d_s\ A_s + M_u = 0$

For the reinforcing steel assumed for the heel, d_s = 14.75 in

Substituting and solving for A_s, it is found that required A_s = 0.07 in^2/ft

Try: #4 bars at 12.00 in c/c for the bottom transverse bars in the toe As = 0.20 in^2/ft

Check Minimum Reinforcement [5.7.3.3.2]

Determine the cracking moment:

$f_r = 0.24\ (f_c')^{0.5}$ = 0.24 x (4.0)$^{0.50}$ = 0.48 ksi

$I_g = (1/12)\ b\ h^3$ = 0.0833 x 12.00 x (18.00)^3 = 5832.0 in^4

$y_t = (1/2)\ h$ = 0.5000 x 18.00 = 9.00 in

$M_{CR} = \gamma_1\ \gamma_3\ f_r\ I_g\ /\ y_t$ = 1.60 x 0.67 x 0.48 x 5832.0 / (9.00 x 12.00) = 27.79 k-ft

The capacity of the section must be greater than or equal to the smaller of:

M_{CR} = 27.79 = 27.79 k-ft
(4/3) M_u = 1.33 x 4.54 = 6.05 k-ft <u>GOVERNS</u>

The capacity of the bottom mat of reinforcement is:

$M_r = \phi\ A_s\ f_y\ (d_s - a/2)$

For the reinforcing steel used, d_s = 18.00 - 3.00 - (0.500 / 2) = 14.75 in

$$M_r = 0.90 \times 0.20 \times 60 \times \left(14.75 - \frac{0.20 \times 60}{1.7 \times 4.0 \times 12} \right) \times \left(\frac{1}{12} \right) = 13.14 \text{ k-ft}$$

13.14 k-ft > 6.05 k-ft OK

Check minimum reinforcement for temperature and shrinkage (5.10.8) A_s = 0.20 in^2/ft > 0.17 in^2/ft OK

Use #4 bars at 12.00 in c/c for bottom transverse reinforcement in the footing.

Longitudinal Reinforcement [5.10.8]

Provide longitudinal reinforcement in the footing based on shrinkage and temperature requirements.

$h_{max} = \max(h_{heel}, h_{toe})$ = 18.00 in

$$\text{Min. } A_s = \frac{1.30\ w_{foot}\ h_{max}}{2\ (w_{foot} + h_{max})\ f_y} = \frac{1.30 \times 114.00 \times 18.00}{2 \times (114.00 + 18.00) \times 60} = 0.17\ in^2/ft$$

The maximum spacing of reinforcement is:

$h_{min} = \min(h_{heel}, h_{toe})$ = 18.00 in

Max. spacing = 3 h_{min} = 54.00 in
or Max. spacing = 18 in = 18.00 in
or Max. spacing = 12 in (for walls and footings ≥ 18 in thick) = 12.00 in GOVERNS

Try: #4 bars at 12.00 in c/c for the top and bottom longitudinal bars A_s = 0.20 in^2/ft > 0.17 in^2/ft OK

0.11 < A_s = 0.20 in^2/ft < 0.60 OK

spacing = 12.00 in = 12.00 in OK

Use #4 bars at 12.00 in c/c for top and bottom longitudinal reinforcement in the footing.

Determine Loads for Wall Stem Design

The loads on the stem at the top of the footing are determined to arrive at the design forces for the wall.

Saturated Earth Pressure:

$P_{EH(S)} = (1/2)\ w_{ss}\ K_a\ h_{ss}^2$ = 0.5 x 0.130 x 0.280 x (6.92)^2 = 0.87 k

$M_{EH(S)} = P_{EH(S)} \times [\ (1/3)\ h_{ss} + h_{sb}\ ]$ = 0.87 x [0.333 x 6.92 + 0.00] = 2.01 k-ft

Buoyant Earth Pressure:

$P_{EH(B)} = K_a\ h_{sb}\ [w_{ss}\ h_{ss} + (1/2)\ w_{sb}\ h_{sb}]$

$P_{EH(B)}$ = 0.280 x 0.00 x [0.130 x 6.92 + 0.5 x 0.068 x 0.00] = 0.00 k

$y_B = [\ h_{sb}\ (\ w_{ss}\ h_{ss} + (1/3)\ w_{sb}\ h_{sb}\ )\ ] / (2\ w_{ss}\ h_{ss} + w_{sb}\ h_{sb}\ )$

y_B = [0.00 x (0.130 x 6.92 + 0.333 x 0.068 x 0.00)]
/ (2.0 x 0.130 x 6.92 + 0.068 x 0.00) = 0.00 ft

$M_{EH(B)} = P_{EH(B)} \times y_B$ = 0.00 x 0.00 = 0.00 k-ft

Water Pressure:

$P_{WA} = (1/2)\ w_w\ h_{water}^2$ = 0.5 x 0.062 x (0.00)^2 = 0.00 k

$M_{WA} = P_{WA} \times (1/3)\ h_{water}$ = 0.00 x 0.333 x 0.00 = 0.00 k-ft

Live Load Surcharge:

$P_{LS} = w_{ss}\ K_a\ h_{LL}\ h_s$ = 0.130 x 0.280 x 3.97 x 6.92 = 1.00 k

$M_{LS} = P_{LS}$ x (1/2) h_s = 1.00 x 0.500 x 6.92 = 3.46 k-ft

Collision Load at Top of Parapet:

Use a Live Load of 1929 lbs/ft applied at h_r = 3.5 ft above the top of the wall.

P_{CT} = 1.93 k

$M_{CT} = P_{CT}$ x ($h_w + h_r$) = 1.93 x (6.92 + 3.50) = 20.10 k-ft

Using the Strength I load combination, the factored design forces for the wall stem are:

$H_u = 1.50\ (P_{EH(S)} + P_{EH(B)}) + 1.00\ P_{WA} + 1.75\ (P_{LS} + P_{LL})$

H_u = 1.50 x (0.87 + 0.00) + 1.00 x 0.00 + 1.75 x (1.00 + 0.00) = 3.06 k

$M_u = 1.50\ (M_{EH(S)} + M_{EH(B)}) + 1.00\ M_{WA} + 1.75\ (M_{LS} + M_{LL})$

M_u = 1.50 x (2.01 + 0.00) + 1.00 x 0.00 + 1.75 x (3.46 + 0.00) = 9.07 k-ft

The Extreme Event II design forces for the wall stem are:

$H_{ext} = 1.50\ (P_{EH(S)} + P_{EH(B)}) + 1.00\ P_{CT} + 0.50\ (P_{LS} + P_{LL})$

H_{ext} = 1.50 x (0.87 + 0.00) + 1.00 x 1.93 + 0.50 x (1.00 + 0.00) = 3.74 k

$M_{ext} = 1.50\ (M_{EH(S)} + M_{EH(B)}) + 1.00\ M_{CT} + 0.50\ (M_{LS} + M_{LL})$

M_{ext} = 1.50 x (2.01 + 0.00) + 1.00 x 20.10 + 0.50 x (3.46 + 0.00) = 24.84 k-ft

The service design forces for the wall stem are:

$H_{serv} = 1.00\ (P_{EH(S)} + P_{EH(B)}) + 1.00\ P_{WA} + 1.00\ (P_{LS} + P_{LL})$

H_{serv} = 1.00 x (0.87 + 0.00) + 1.00 x 0.00 + 1.00 x (1.00 + 0.00) = 1.87 k

$M_{serv} = 1.00\ (M_{EH(S)} + M_{EH(B)}) + 1.00\ M_{WA} + 1.00\ (M_{LS} + M_{LL})$

M_{serv} = 1.00 x (2.01 + 0.00) + 1.00 x 0.00 + 1.00 x (3.46 + 0.00) = 5.47 k-ft

Wall Stem Design - Investigate Shear

Shear typically does not govern the design of retaining walls. If shear does become an issue, the thickness of the stem should be increased such that transverse reinforcement is not required.

Ignoring the benefits of the shear key and axial compression, the shear capacity of the stem can be shown to be greater than that required.

$V_n = V_c + V_s + V_p$ [5.8.3.3-1]

Recognizing that V_s and V_p are zero

$V_n = V_c$

$V_c = 0.0316\ \beta\ (f_c')^{0.5}\ b_v\ d_v$ [5.8.3.3-3]

The maximum effective shear depth is:

For 2.0 inch clear cover and #6 bars, d_s = 18.00 - 2.00 - (0.750 / 2) = 15.63 in

$d_v = 0.9\ d_e = 0.9\ d_s$ = 0.90 x 15.63 = 14.06 in GOVERNS
$d_v = 0.72\ h = 0.72\ t_{wb}$ = 0.72 x 18.00 = 12.96 in

Follow the General Procedure using the provisions of Appendix B5, as per Section [5.8.3.4.2]:

β = 1.968862 ε_s= 0.00195 s_{xe}= 11.42945 Max Aggregate Size = 1 in (BDM S5.10.3.1.1)

$\phi\ V_c$ = 0.90 x 0.0316 x 1.97 x (4.0)$^{0.50}$ x 12.00 x 14.06 = 18.90 k

18.90 k > 3.74 k OK

Wall Stem Design - Investigate Strength Limit State

Determine the area of back-face flexural reinforcement necessary to satisfy the design moment:

M_u = 24.84 k-ft

Again use the equation:

3.309 A_s^2 - 4.50 $d_s A_s + M_u = 0$

Substituting and solving for A_s, it is found that required A_s = 0.36 in^2/ft

Try: #6 bars at 12.00 in c/c A_s = 0.44 in^2/ft > 0.36 in^2/ft OK

Wall Stem Design - Investigate Service Limit State [5.7.3.4]

Check the crack control equations to ensure that the primary flexural reinforcement is well distributed. The service load bending moment is:

M_{serv} = 5.47 k-ft

Check the modulus of rupture for concrete:

$\phi f_r = \phi 0.24 (f_c')^{0.5}$ = 0.80 x 0.24 x (4.0)$^{0.50}$ = 0.384 ksi

$S = \frac{b t_{wb}^2}{6} = \frac{12.00 \times (18.00)^2}{6}$ = 648.00 in^3

$f_{act} = \frac{M_{serv}}{S} = \frac{5.47 \times 12}{648.00}$ = 0.101 ksi < 0.384 ksi OK

If modulus of rupture check is "NG", then check the spacing of the reinforcement. First, determine the modular ratio:

$E_c = 33{,}000 w_c^{1.5} (f_c')^{0.5}$ = 33,000 x (0.150)$^{1.5}$ x (4.00)$^{0.5}$ = 3,834 ksi

$n = E_s/E_c$ = 29,000 / 3,834 = 7.56 , use n = 8.00

For 2.0 inch clear cover and #6 bars, d_s = 18.00 - 2.00 - (0.750 / 2) = 15.63 in

Determine the location of the neutral axis:

$0.5 b x^2 = n A_s (d_s - x)$

$0.5 (12) x^2$ = 8 x 0.44 x (15.63 - x)

solving, x = 2.75 in

Check the spacing of the reinforcement to control cracking:

$y = d_s - x$ = 15.63 - 2.75 = 12.88 in

$I_{cr} = \frac{b x^3}{3} + n A_s (d_s - x)^2$

$I_{cr} = \frac{12.00 \times (2.75)^3}{3}$ + 8 x 0.44 x (15.63 - 2.75)^2 = 666.68 in^4

γ_e = 1.00

For 2.0 inch clear cover and #6 bars, d_c = 2.00 + (0.750 / 2) = 2.38 in

$\beta_s = 1.0 + \frac{d_c}{0.7 (t_{wb} - d_c)} = 1.0 + \frac{2.38}{0.7 \times (18.00 - 2.38)}$ = 1.22

$f_{ss} = n \frac{M_{serv}\, y}{I_{cr}} = 8 \times \frac{5.47 \times 12.88 \times 12.00}{666.68}$ = 10.14 ksi

$s_{max} = \dfrac{700\,\gamma_e}{\beta_s f_{ss}} - 2.0\,d_c = \dfrac{700 \times 1.00}{1.22 \times 10.14} - 2.0 \times 2.38 = 51.95$ in

$s_{max} =$ 51.95 in > 12.00 in OK

Wall Stem Design - Check Reinforcement Limits

Check Minimum Reinforcement [5.7.3.3.2]

Determine the cracking moment:

$f_r = 0.24\,(f_c')^{0.5} = 0.24 \times (4.0)^{0.50} = 0.48$ ksi

$I_g = (1/12)\,b\,h^3 = 0.0833 \times 12.00 \times (18.00)^3 = 5832.0$ in^4

$y_t = (1/2)\,h = 0.5000 \times 18.00 = 9.00$ in

$M_{CR} = \gamma_1\,\gamma_3\,f_r\,I_g / y_t = 0.67 \times 1.6 \times 0.48 \times 5832.0 / (9.00 \times 12.00) = 27.79$ k-ft

The capacity of the section must be greater than or equal to the smaller of:

$M_{CR} = 27.79 = 27.79$ k-ft GOVERNS

$(4/3)\,M_u = 1.33 \times 24.84 = 33.12$ k-ft

The capacity of the reinforcement is:

$M_r = \phi\,A_s\,f_y\,(d_s - a/2)$

$M_r = 0.90 \times 0.44 \times 60 \times \left(15.63 - \dfrac{0.44 \times 60}{1.7 \times 4.0 \times 12}\right) \times \left(\dfrac{1}{12}\right) = 30.30$ k-ft

30.30 k-ft > 27.79 k-ft OK

Check minimum reinforcement for temperature and shrinkage (5.10.8) $A_s =$ 0.44 in^2/ft > 0.16 in^2/ft OK

Use #6 bars at 12.00 in c/c for wall stem back face vertical reinforcing.

Wall Stem Design - Shrinkage and Temperature Reinforcement [5.10.8]

A minimum amount of reinforcement should be placed near each face of concrete elements to limit the size of cracks associated with concrete shrinkage and temperature changes.

$h_{max} = \max(t_{wt}, t_{wb}) = 18.00$ in

$h_w =$ 6.92 ft = 83.04 in

$\text{Min. } A_s = \dfrac{1.30\,h_w\,h_{max}}{2\,(h_w + h_{max})\,f_y} = \dfrac{1.30 \times 83.04 \times 18.00}{2 \times (83.04 + 18.00) \times 60} = 0.16$ in^2/ft

The maximum spacing of reinforcement is:

$h_{min} = \min(t_{wt}, t_{wb}) = 18.00$ in

Max. spacing = 3 h_{min} = 54.00 in
or Max. spacing = 18 in = 18.00 in
or Max. spacing = 12 in (for walls and footings ≥ 18 in thick) = 12.00 in GOVERNS

Try: #4 bars at 12.00 in c/c $A_s =$ 0.20 in^2/ft > 0.16 in^2/ft OK

0.11 < $A_s =$ 0.20 in^2/ft < 0.60 OK

spacing = 12.00 in = 12.00 in OK

Use #4 bars at 12.00 in c/c for wall stem front face reinforcing and back face horizontal reinforcing.

Dimensions and Weights for Concrete Design

Footing width, w_{foot} =	9.50	ft	=	114.00	in	Concrete weight, w_c =	0.150	kcf		
						Water weight, w_w =	0.062	kcf		
Footing heel width, w_{heel} =	6.00	ft				Saturated soil weight, w_{ss} =	0.130	kcf		
Footing heel height, h_{heel} =	1.50	ft	=	18.00	in	Buoyant soil weight, w_{sb} =	0.068	kcf		
Footing toe width, w_{toe} =	2.00	ft				Height of wall, h_w =	7.82	ft	top of heel to top of wall	
Footing toe height, h_{toe} =	1.50	ft	=	18.00	in	Height of water, h_{water} =	0.00	ft	top of heel to water line	
Wall width at top, t_{wt} =	1.50	ft	=	18.00	in	Height of soil, h_s =	7.82	ft	top of heel to ground line	
Wall width at base, t_{wb} =	1.50	ft	=	18.00	in	Height of satur. soil, h_{ss} =	7.82	ft	height of satur. soil above top of heel	
						Height of buoy. soil, h_{sb} =	0.00	ft	height of buoy. soil above top of heel	
Concrete strength, f_c' =	4.00	ksi								
Rebar strength, f_y =	60.00	ksi				Active pressure coeff., K_a =	0.280			
Steel mod. of elast., E_s =	29,000	ksi				LL surcharge soil ht., h_{LL} =	3.70	ft		

Optional Collision Loading for Barrier on Top of Wall

Per ODOT comments for I-70/71, use the transverse loading of 54 kips for a TL-4 test level railing [AASHTO, Table A13.2-1] distributed over the retaining wall's joint spacing.

Collision Loading = y (y or n)
Joint Spacing = 28.00 ft
Barrier Height = 3.50 ft

Design Summary

Summary of Design Status

Design Item	Footing Heel	Footing Toe	Wall Stem
Shear	OK	OK	OK
Minimum Reinforcement	OK	OK	OK
Shrinkage & Temperature	OK	OK	OK
Crack Control	N/A	N/A	OK

For calculations related to each design item, see below.

Reinforcing Steel Summary

Footing:	Top transverse:	#8 bars at 12.00 in c/c
	Bottom transverse:	#4 bars at 12.00 in c/c
	Longitudinal:	#4 bars at 12.00 in c/c
Wall Stem:	Back face vertical:	#6 bars at 12.00 in c/c
	Front face vertical:	#4 bars at 12.00 in c/c
	Horizontal:	#4 bars at 12.00 in c/c

Design Footing for Shear [5.13.3.6]

Design footings to have adequate shear capacity without transverse reinforcement.

Determine d_v

Assume: #8 bars at 12.00 in c/c for the top transverse bars in the heel A_s = 0.79 in^2/ft 2.0 in cover
#4 bars at 12.00 in c/c for the bottom transverse bars in the toe A_s = 0.20 in^2/ft 3.0 in cover

For the heel:

$d_{sheel} = h - cover - d_{bar}/2 = 18.00 - 2.00 - (1.000 / 2) = 15.50$ in

$a_{heel} = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{0.79 \times 60}{0.85 \times 4.0 \times 12} = 1.16$ in

[5.8.2.9] $d_{vheel} = d_{sheel} - a/2 = 15.50 - (1.16 / 2) = 14.92$ in GOVERNS
or $d_{vheel} = 0.90\, d_e = 0.90 \times 15.50 = 13.95$ in
or $d_{vheel} = 0.72\, h = 0.72 \times 18.00 = 12.96$ in

For the toe:

$d_{stoe} = h - cover - d_{bar}/2 = 18.00 - 3.00 - (0.500 / 2) = 14.75$ in

$a_{toe} = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{0.20 \times 60}{0.85 \times 4.0 \times 12} = 0.29$ in

[5.8.2.9] $d_{vtoe} = d_{stoe} - a/2 = 14.75 - (0.29 / 2) = 14.60$ in GOVERNS
or $d_{vtoe} = 0.90\, d_e = 0.90 \times 14.75 = 13.28$ in

or d_{vtoe} = 0.72 h = 0.72 x 18.00 = 12.96 in

Check Heel for Shear

The critical shear section for the heel of the footing is located at the back face of the wall. The heel of the footing is assumed to carry its self weight and the rectangular soil block above it. This neglects the benefit of any upward soil pressure below the footing (conservative).

$V_u = (\gamma_{EV} w_{ss} h_{ss} + \gamma_{EV} w_{sb} h_{sb} + \gamma_{DC} w_c h_{heel} + \gamma_{LL} w_{ss} h_{LL} + \gamma_{WA} w_w h_{water}) \times w_{heel}$

V_u = (1.35 x 0.130 x 7.82 + 1.35 x 0.068 x 0.00 + 1.25 x 0.150 x 1.50 +
1.75 x 0.130 x 3.70 + 1.00 x 0.062 x 0.00) x 6.00 = 14.97 k/ft

[5.8.3.3] Using β = 2.00 and assuming bars in the top mat as above:

[5.8.3.4] $\phi V_c = \phi\, 0.0316\, \beta\, (f_c')^{0.5}\, b_v\, d_v$
[5.8.2.9]

ϕV_c = 0.90 x 0.0316 x 2.00 x (4.0)$^{0.50}$ x 12.00 x 14.92 = 20.37 k

20.37 k > 14.97 k OK

Check Toe for Shear

The peak bearing stress is 2.42 ksf for the Extreme IIb load case.

The critical section for the toe of the footing is at dv from the front face of the wall. For a quick simplified check, try applying the peak bearing stress over the entire length of the toe (conservative).

$V_u = \sigma_V w_{toe}$ = 2.42 x 2.00 = 4.84 k/ft

[5.8.3.3] Using β = 2.00 and assuming bars in the bottom mat as above:

[5.8.3.4] $\phi V_c = \phi\, 0.0316\, \beta\, (f_c')^{0.5}\, b_v\, d_v$
[5.8.2.9]

ϕV_c = 0.90 x 0.0316 x 2.00 x (4.0)$^{0.50}$ x 12.00 x 14.60 = 19.93 k

19.93 k > 4.84 k OK

Design Footing Reinforcement [5.13.3.4]

Each mat of reinforcement is checked to ensure that it has adequate capacity and that the maximum and minimum reinforcement checks are satisfied. The critical section for flexure in the footing is at the face of the wall.

Top Transverse Reinforcement

From the shear check of the heel, V_u = 14.97 k/ft

$M_u = V_u \times (w_{heel} / 2)$ = 14.97 x (6.00 / 2) = 44.92 k-ft

Set up the equation to solve for the required steel area:

$$M_u = \phi M_n = \phi A_s f_y (d_s - a/2) = \phi A_s f_y \left(d_s - \frac{A_s f_y}{1.7 f_c' b} \right)$$

$$M_u = 0.90 \times A_s \times 60 \left(ds - \frac{As \times 60}{1.7 \times 4.0 \times 12} \right) \times \left(\frac{1}{12} \right)$$

$$3.309\ A_s^2 - 4.50\ d_s A_s + M_u = 0$$

For the reinforcing steel assumed for the heel, d_s = 15.50 in

Substituting and solving for A_s, it is found that required A_s = 0.66 in^2/ft

Try: #8 bars at 12.00 in c/c for the top transverse bars in the heel As = 0.79 in^2/ft

Check Minimum Reinforcement [5.7.3.3.2]

Determine the cracking moment:

$f_r = 0.24\ (f_c')^{0.5}$ = 0.24 x (4.0)$^{0.50}$ = 0.48 ksi

$I_g = (1/12)\ b\ h^3$ = 0.0833 x 12.00 x (18.00)^3 = 5832.0 in^4

$y_t = (1/2)\ h$ = 0.5000 x 18.00 = 9.00 in

$M_{CR} = \gamma_1\ \gamma_3\ f_r\ I_g\ /\ y_t$ = 1.60 x 0.67 x 0.48 x 5832.0 / (9.00 x 12.00) = 27.79 k-ft

The capacity of the section must be greater than or equal to the smaller of:

M_{CR} = 27.79 = 27.79 k-ft GOVERNS
$(4/3)\ M_u$ = 1.33 x 44.92 = 59.89 k-ft

The capacity of the top mat of reinforcement is:

$M_r = \phi\ A_s\ f_y\ (d_s - a/2)$

For the reinforcing steel used, d_s = 18.00 - 2.00 - (1.000 / 2) = 15.50 in

$$M_r = 0.90 \times 0.79 \times 60 \times \left(15.50 - \frac{0.79 \times 60}{1.7 \times 4.0 \times 12} \right) \times \left(\frac{1}{12} \right) = 53.04 \text{ k-ft}$$

53.04 k-ft > 27.79 k-ft OK

Check minimum reinforcement for temperature and shrinkage (5.10.8) A_s = 0.79 in^2/ft > 0.17 in^2/ft OK

Use #8 bars at 12.00 in c/c for top transverse reinforcement in the footing.

Bottom Transverse Reinforcement

From the shear check of the toe, V_u = 4.84 k/ft

$M_u = V_u \times (w_{toe}\ /\ 2)$ = 4.84 x (2.00 / 2) = 4.84 k-ft

Set up the equation to solve for the required steel area and again use:

3.309 A_s^2 - 4.50 $d_s\ A_s + M_u = 0$

For the reinforcing steel assumed for the heel, d_s = 14.75 in

Substituting and solving for A_s, it is found that required A_s = 0.07 in^2/ft

Try: #4 bars at 12.00 in c/c for the bottom transverse bars in the toe As = 0.20 in^2/ft

Check Minimum Reinforcement [5.7.3.3.2]

Determine the cracking moment:

$f_r = 0.24\ (f_c')^{0.5}$ = 0.24 x (4.0)$^{0.50}$ = 0.48 ksi

$I_g = (1/12)\ b\ h^3$ = 0.0833 x 12.00 x (18.00)^3 = 5832.0 in^4

$y_t = (1/2)\ h$ = 0.5000 x 18.00 = 9.00 in

$M_{CR} = \gamma_1\ \gamma_3\ f_r\ I_g\ /\ y_t$ = 1.60 x 0.67 x 0.48 x 5832.0 / (9.00 x 12.00) = 27.79 k-ft

The capacity of the section must be greater than or equal to the smaller of:

M_{CR} = 27.79 = 27.79 k-ft
$(4/3)\ M_u$ = 1.33 x 4.84 = 6.45 k-ft GOVERNS

The capacity of the bottom mat of reinforcement is:

$M_r = \phi\ A_s\ f_y\ (d_s - a/2)$

For the reinforcing steel used, d_s = 18.00 - 3.00 - (0.500 / 2) = 14.75 in

$M_r = 0.90 \times 0.20 \times 60 \times \left(14.75 - \frac{0.20 \times 60}{1.7 \times 4.0 \times 12} \right) \times \left(\frac{1}{12} \right) = 13.14$ k-ft

13.14 k-ft > 6.45 k-ft OK

Check minimum reinforcement for temperature and shrinkage (5.10.8) A_s = 0.20 in^2/ft > 0.17 in^2/ft OK

Use #4 bars at 12.00 in c/c for bottom transverse reinforcement in the footing.

Longitudinal Reinforcement [5.10.8]

Provide longitudinal reinforcement in the footing based on shrinkage and temperature requirements.

$h_{max} = \max(h_{heel}, h_{toe})$ = 18.00 in

$\text{Min. } A_s = \frac{1.30\ w_{foot}\ h_{max}}{2\ (w_{foot} + h_{max})\ f_y} = \frac{1.30 \times 114.00 \times 18.00}{2 \times (114.00 + 18.00) \times 60} = 0.17$ in^2/ft

The maximum spacing of reinforcement is:

$h_{min} = \min(h_{heel}, h_{toe})$ = 18.00 in

Max. spacing = 3 h_{min} = 54.00 in
or Max. spacing = 18 in = 18.00 in
or Max. spacing = 12 in (for walls and footings ≥ 18 in thick) = 12.00 in GOVERNS

Try: #4 bars at 12.00 in c/c for the top and bottom longitudinal bars A_s = 0.20 in^2/ft > 0.17 in^2/ft OK

0.11 < A_s = 0.20 in^2/ft < 0.60 OK

spacing = 12.00 in = 12.00 in OK

Use #4 bars at 12.00 in c/c for top and bottom longitudinal reinforcement in the footing.

Determine Loads for Wall Stem Design

The loads on the stem at the top of the footing are determined to arrive at the design forces for the wall.

Saturated Earth Pressure:

$P_{EH(S)} = (1/2)\ w_{ss}\ K_a\ h_{ss}^2$ = 0.5 x 0.130 x 0.280 x (7.82)^2 = 1.11 k

$M_{EH(S)} = P_{EH(S)} \times [\ (1/3)\ h_{ss} + h_{sb}\ ]$ = 1.11 x [0.333 x 7.82 + 0.00] = 2.90 k-ft

Buoyant Earth Pressure:

$P_{EH(B)} = K_a\ h_{sb}\ [w_{ss}\ h_{ss} + (1/2)\ w_{sb}\ h_{sb}]$

$P_{EH(B)}$ = 0.280 x 0.00 x [0.130 x 7.82 + 0.5 x 0.068 x 0.00] = 0.00 k

$y_B = [\ h_{sb}\ (\ w_{ss}\ h_{ss} + (1/3)\ w_{sb}\ h_{sb}\ )\ ]\ /\ (2\ w_{ss}\ h_{ss} + w_{sb}\ h_{sb}\ )$

y_B = [0.00 x (0.130 x 7.82 + 0.333 x 0.068 x 0.00)]
/ (2.0 x 0.130 x 7.82 + 0.068 x 0.00) = 0.00 ft

$M_{EH(B)} = P_{EH(B)} \times y_B$ = 0.00 x 0.00 = 0.00 k-ft

Water Pressure:

$P_{WA} = (1/2)\ w_w\ h_{water}^2$ = 0.5 x 0.062 x (0.00)^2 = 0.00 k

$M_{WA} = P_{WA} \times (1/3)\ h_{water}$ = 0.00 x 0.333 x 0.00 = 0.00 k-ft

Live Load Surcharge:

$P_{LS} = w_{ss}\ K_a\ h_{LL}\ h_s$ = 0.130 x 0.280 x 3.70 x 7.82 = 1.05 k

$M_{LS} = P_{LS} \times (1/2)\, h_s$ = 1.05 x 0.500 x 7.82 = 4.12 k-ft

Collision Load at Top of Parapet:

Use a Live Load of 1929 lbs/ft applied at h_r = 3.5 ft above the top of the wall.

P_{CT} = 1.93 k

$M_{CT} = P_{CT} \times (h_w + h_r)$ = 1.93 x (7.82 + 3.50) = 21.83 k-ft

Using the Strength I load combination, the factored design forces for the wall stem are:

$H_u = 1.50\,(P_{EH(S)} + P_{EH(B)}) + 1.00\,P_{WA} + 1.75\,(P_{LS} + P_{LL})$

H_u = 1.50 x (1.11 + 0.00) + 1.00 x 0.00 + 1.75 x (1.05 + 0.00) = 3.51 k

$M_u = 1.50\,(M_{EH(S)} + M_{EH(B)}) + 1.00\,M_{WA} + 1.75\,(M_{LS} + M_{LL})$

M_u = 1.50 x (2.90 + 0.00) + 1.00 x 0.00 + 1.75 x (4.12 + 0.00) = 11.56 k-ft

The Extreme Event II design forces for the wall stem are:

$H_{ext} = 1.50\,(P_{EH(S)} + P_{EH(B)}) + 1.00\,P_{CT} + 0.50\,(P_{LS} + P_{LL})$

H_{ext} = 1.50 x (1.11 + 0.00) + 1.00 x 1.93 + 0.50 x (1.05 + 0.00) = 4.12 k

$M_{ext} = 1.50\,(M_{EH(S)} + M_{EH(B)}) + 1.00\,M_{CT} + 0.50\,(M_{LS} + M_{LL})$

M_{ext} = 1.50 x (2.90 + 0.00) + 1.00 x 21.83 + 0.50 x (4.12 + 0.00) = 28.24 k-ft

The service design forces for the wall stem are:

$H_{serv} = 1.00\,(P_{EH(S)} + P_{EH(B)}) + 1.00\,P_{WA} + 1.00\,(P_{LS} + P_{LL})$

H_{serv} = 1.00 x (1.11 + 0.00) + 1.00 x 0.00 + 1.00 x (1.05 + 0.00) = 2.17 k

$M_{serv} = 1.00\,(M_{EH(S)} + M_{EH(B)}) + 1.00\,M_{WA} + 1.00\,(M_{LS} + M_{LL})$

M_{serv} = 1.00 x (2.90 + 0.00) + 1.00 x 0.00 + 1.00 x (4.12 + 0.00) = 7.02 k-ft

Wall Stem Design - Investigate Shear

Shear typically does not govern the design of retaining walls. If shear does become an issue, the thickness of the stem should be increased such that transverse reinforcement is not required.

Ignoring the benefits of the shear key and axial compression, the shear capacity of the stem can be shown to be greater than that required.

$V_n = V_c + V_s + V_p$ [5.8.3.3-1]

Recognizing that V_s and V_p are zero

$V_n = V_c$

$V_c = 0.0316\, \beta\, (f_c')^{0.5}\, b_v\, d_v$ [5.8.3.3-3]

The maximum effective shear depth is:

For 2.0 inch clear cover and #6 bars, d_s = 18.00 - 2.00 - (0.750 / 2) = 15.63 in

$d_v = 0.9\, d_e = 0.9\, d_s$ = 0.90 x 15.63 = 14.06 in <u>GOVERNS</u>
$d_v = 0.72\, h = 0.72\, t_{wb}$ = 0.72 x 18.00 = 12.96 in

Follow the General Procedure using the provisions of Appendix B5, as per Section [5.8.3.4.2]:

β = 1.825634 ε_s= 0.00221 s_{xe}= 11.42945 Max Aggregate Size = 1 in (BDM S5.10.3.1.1)

ϕV_c = 0.90 x 0.0316 x 1.83 x (4.0)$^{0.50}$ x 12.00 x 14.06 = 17.52 k

17.52 k > 4.12 k OK

Wall Stem Design - Investigate Strength Limit State

Determine the area of back-face flexural reinforcement necessary to satisfy the design moment:

$M_u =$ 28.24 k-ft

Again use the equation:

$3.309 \; A_s^2 - 4.50 \; d_s A_s + M_u = 0$

Substituting and solving for A_s, it is found that required $A_s =$ 0.41 in^2/ft

Try: #6 bars at 12.00 in c/c $A_s =$ 0.44 in^2/ft > 0.41 in^2/ft OK

Wall Stem Design - Investigate Service Limit State [5.7.3.4]

Check the crack control equations to ensure that the primary flexural reinforcement is well distributed. The service load bending moment is:

$M_{serv} =$ 7.02 k-ft

Check the modulus of rupture for concrete:

$\phi f_r = \phi \, 0.24 \, (f_c')^{0.5} = 0.80 \times 0.24 \times (4.0)^{0.50} = 0.384$ ksi

$S = \dfrac{b \, t_{wb}^2}{6} = \dfrac{12.00 \times (18.00)^2}{6} = 648.00 \; in^3$

$f_{act} = \dfrac{M_{serv}}{S} = \dfrac{7.02 \times 12}{648.00} = 0.130$ ksi < 0.384 ksi OK

If modulus of rupture check is "NG", then check the spacing of the reinforcement. First, determine the modular ratio:

$E_c = 33{,}000 \, w_c^{1.5} (f_c')^{0.5} = 33{,}000 \times (0.150)^{1.5} \times (4.00)^{0.5} = 3{,}834$ ksi

$n = E_s/E_c = 29{,}000 \; / \; 3{,}834 = 7.56$, use n = 8.00

For 2.0 inch clear cover and #6 bars, $d_s = 18.00 - 2.00 - (0.750 \; / \; 2) = 15.63$ in

Determine the location of the neutral axis:

$0.5 \, b \, x^2 = n \, A_s (d_s - x)$

$0.5 \, (12) \, x^2 = 8 \times 0.44 \times (15.63 - x)$

solving, x = 2.75 in

Check the spacing of the reinforcement to control cracking:

$y = d_s - x = 15.63 - 2.75 = 12.88$ in

$I_{cr} = \dfrac{b \, x^3}{3} + n \, A_s (d_s - x)^2$

$I_{cr} = \dfrac{12.00 \times (2.75)^3}{3} + 8 \times 0.44 \times (15.63 - 2.75)^2 = 666.68 \; in^4$

$\gamma_e =$ 1.00

For 2.0 inch clear cover and #6 bars, $d_c = 2.00 + (0.750 \; / \; 2) = 2.38$ in

$\beta_s = 1.0 + \dfrac{d_c}{0.7 (t_{wb} - d_c)} = 1.0 + \dfrac{2.38}{0.7 \times (18.00 - 2.38)} = 1.22$

$f_{ss} = n \dfrac{M_{serv} \, y}{I_{cr}} = 8 \times \dfrac{7.02 \times 12.88 \times 12.00}{666.68} = 13.01$ ksi

$s_{max} = \dfrac{700\,\gamma_e}{\beta_s f_{ss}} - 2.0\,d_c = \dfrac{700 \times 1.00}{1.22 \times 13.01} - 2.0 \times 2.38 = 39.44$ in

$s_{max} =$ 39.44 in > 12.00 in OK

Wall Stem Design - Check Reinforcement Limits

Check Minimum Reinforcement [5.7.3.3.2]

Determine the cracking moment:

$f_r = 0.24\,(f_c')^{0.5}$ = 0.24 x (4.0)$^{0.50}$ = 0.48 ksi

$I_g = (1/12)\,b\,h^3$ = 0.0833 x 12.00 x (18.00)^3 = 5832.0 in^4

$y_t = (1/2)\,h$ = 0.5000 x 18.00 = 9.00 in

$M_{CR} = \gamma_1\,\gamma_3\,f_r\,I_g\,/\,y_t$ = 0.67 x 1.6 x 0.48 x 5832.0 / (9.00 x 12.00) = 27.79 k-ft

The capacity of the section must be greater than or equal to the smaller of:

M_{CR}	=			27.79	=	27.79	k-ft	GOVERNS
(4/3) M_u	=	1.33	x	28.24	=	37.66	k-ft	

The capacity of the reinforcement is:

$M_r = \phi\,A_s\,f_y\,(d_s - a/2)$

$M_r = 0.90 \times 0.44 \times 60 \times \left(15.63 - \dfrac{0.44 \times 60}{1.7 \times 4.0 \times 12}\right) \times \left(\dfrac{1}{12}\right) = 30.30$ k-ft

30.30 k-ft > 27.79 k-ft OK

Check minimum reinforcement for temperature and shrinkage (5.10.8) A_s = 0.44 in^2/ft > 0.16 in^2/ft OK

Use #6 bars at 12.00 in c/c for wall stem back face vertical reinforcing.

Wall Stem Design - Shrinkage and Temperature Reinforcement [5.10.8]

A minimum amount of reinforcement should be placed near each face of concrete elements to limit the size of cracks associated with concrete shrinkage and temperature changes.

$h_{max} = \max(t_{wt}, t_{wb})$ = 18.00 in

h_w = 7.82 ft = 93.84 in

$\text{Min. } A_s = \dfrac{1.30\,h_w\,h_{max}}{2\,(h_w + h_{max})\,f_y} = \dfrac{1.30 \times 93.84 \times 18.00}{2 \times (93.84 + 18.00) \times 60} = 0.16$ in^2/ft

The maximum spacing of reinforcement is:

$h_{min} = \min(t_{wt}, t_{wb})$ = 18.00 in

Max. spacing = 3 h_{min}	=	54.00	in	
or Max. spacing = 18 in	=	18.00	in	
or Max. spacing = 12 in (for walls and footings ≥ 18 in thick)	=	12.00	in	GOVERNS

Try: #4 bars at 12.00 in c/c

A_s = 0.20 in^2/ft > 0.16 in^2/ft OK

0.11 < A_s = 0.20 in^2/ft < 0.60 OK

spacing = 12.00 in = 12.00 in OK

Use #4 bars at 12.00 in c/c for wall stem front face reinforcing and back face horizontal reinforcing.

Dimensions and Weights for Concrete Design

Footing width, w_{foot} =	9.50	ft	=	114.00	in	Concrete weight, w_c =	0.150	kcf	
						Water weight, w_w =	0.062	kcf	
Footing heel width, w_{heel} =	6.00	ft				Saturated soil weight, w_{ss} =	0.130	kcf	
Footing heel height, h_{heel} =	1.50	ft	=	18.00	in	Buoyant soil weight, w_{sb} =	0.068	kcf	
Footing toe width, w_{toe} =	2.00	ft				Height of wall, h_w =	8.85	ft	top of heel to top of wall
Footing toe height, h_{toe} =	1.50	ft	=	18.00	in	Height of water, h_{water} =	0.00	ft	top of heel to water line
Wall width at top, t_{wt} =	1.50	ft	=	18.00	in	Height of soil, h_s =	8.85	ft	top of heel to ground line
Wall width at base, t_{wb} =	1.50	ft	=	18.00	in	Height of satur. soil, h_{ss} =	8.85	ft	height of satur. soil above top of heel
						Height of buoy. soil, h_{sb} =	0.00	ft	height of buoy. soil above top of heel
Concrete strength, f_c' =	4.00	ksi							
Rebar strength, f_y =	60.00	ksi				Active pressure coeff., K_a =	0.280		
Steel mod. of elast., E_s =	29,000	ksi				LL surcharge soil ht., h_{LL} =	3.45	ft	

Optional Collision Loading for Barrier on Top of Wall

Per ODOT comments for I-70/71, use the transverse loading of 54 kips for a TL-4 test level railing [AASHTO, Table A13.2-1] distributed over the retaining wall's joint spacing.

Collision Loading =	y	(y or n)
Joint Spacing =	28.00	ft
Barrier Height =	3.50	ft

Design Summary

Summary of Design Status

Design Item	Footing Heel	Footing Toe	Wall Stem
Shear	OK	OK	OK
Minimum Reinforcement	OK	OK	OK
Shrinkage & Temperature	OK	OK	OK
Crack Control	N/A	N/A	OK

For calculations related to each design item, see below.

Reinforcing Steel Summary

Footing:	Top transverse:	#8 bars at 12.00 in c/c
	Bottom transverse:	#4 bars at 12.00 in c/c
	Longitudinal:	#4 bars at 12.00 in c/c
Wall Stem:	Back face vertical:	#7 bars at 12.00 in c/c
	Front face vertical:	#4 bars at 12.00 in c/c
	Horizontal:	#4 bars at 12.00 in c/c

Design Footing for Shear [5.13.3.6]

Design footings to have adequate shear capacity without transverse reinforcement.

Determine d_v

Assume: #8 bars at 12.00 in c/c for the top transverse bars in the heel A_s = 0.79 in^2/ft 2.0 in cover

#4 bars at 12.00 in c/c for the bottom transverse bars in the toe A_s = 0.20 in^2/ft 3.0 in cover

For the heel:

$d_{sheel} = h - cover - d_{bar}/2 = 18.00 - 2.00 - (1.000 / 2) = 15.50$ in

$a_{heel} = \frac{A_s f_y}{0.85 f_c' b} = \frac{0.79 \times 60}{0.85 \times 4.0 \times 12} = 1.16$ in

[5.8.2.9] $d_{vheel} = d_{sheel} - a/2 = 15.50 - (1.16 / 2) = 14.92$ in GOVERNS

or $d_{vheel} = 0.90 d_e = 0.90 \times 15.50 = 13.95$ in

or $d_{vheel} = 0.72 h = 0.72 \times 18.00 = 12.96$ in

For the toe:

$d_{stoe} = h - cover - d_{bar}/2 = 18.00 - 3.00 - (0.500 / 2) = 14.75$ in

$a_{toe} = \frac{A_s f_y}{0.85 f_c' b} = \frac{0.20 \times 60}{0.85 \times 4.0 \times 12} = 0.29$ in

[5.8.2.9] $d_{vtoe} = d_{stoe} - a/2 = 14.75 - (0.29 / 2) = 14.60$ in GOVERNS

or $d_{vtoe} = 0.90 d_e = 0.90 \times 14.75 = 13.28$ in

or d_{vtoe} = 0.72 h = 0.72 x 18.00 = 12.96 in

Check Heel for Shear

The critical shear section for the heel of the footing is located at the back face of the wall. The heel of the footing is assumed to carry its self weight and the rectangular soil block above it. This neglects the benefit of any upward soil pressure below the footing (conservative).

$V_u = (\gamma_{EV} w_{ss} h_{ss} + \gamma_{EV} w_{sb} h_{sb} + \gamma_{DC} w_c h_{heel} + \gamma_{LL} w_{ss} h_{LL} + \gamma_{WA} w_w h_{water}) \times w_{heel}$

V_u = (1.35 x 0.130 x 8.85 + 1.35 x 0.068 x 0.00 + 1.25 x 0.150 x 1.50 + 1.75 x 0.130 x 3.45 + 1.00 x 0.062 x 0.00) x 6.00 = 15.72 k/ft

[5.8.3.3] Using β = 2.00 and assuming bars in the top mat as above:

[5.8.3.4] $\phi V_c = \phi\, 0.0316\, \beta\, (f_c')^{0.5}\, b_v\, d_v$

[5.8.2.9]

ϕV_c = 0.90 x 0.0316 x 2.00 x (4.0)$^{0.50}$ x 12.00 x 14.92 = 20.37 k

20.37 k > 15.72 k OK

Check Toe for Shear

The peak bearing stress is 2.62 ksf for the Extreme IIb load case.

The critical section for the toe of the footing is at dv from the front face of the wall. For a quick simplified check, try applying the peak bearing stress over the entire length of the toe (conservative).

$V_u = \sigma_V w_{toe}$ = 2.62 x 2.00 = 5.24 k/ft

[5.8.3.3] Using β = 2.00 and assuming bars in the bottom mat as above:

[5.8.3.4] $\phi V_c = \phi\, 0.0316\, \beta\, (f_c')^{0.5}\, b_v\, d_v$

[5.8.2.9]

ϕV_c = 0.90 x 0.0316 x 2.00 x (4.0)$^{0.50}$ x 12.00 x 14.60 = 19.93 k

19.93 k > 5.24 k OK

Design Footing Reinforcement [5.13.3.4]

Each mat of reinforcement is checked to ensure that it has adequate capacity and that the maximum and minimum reinforcement checks are satisfied. The critical section for flexure in the footing is at the face of the wall.

Top Transverse Reinforcement

From the shear check of the heel, V_u = 15.72 k/ft

$M_u = V_u \times (w_{heel} / 2)$ = 15.72 x (6.00 / 2) = 47.15 k-ft

Set up the equation to solve for the required steel area:

$$M_u = \phi M_n = \phi A_s f_y (d_s - a/2) = \phi A_s f_y \left(d_s - \frac{A_s f_y}{1.7 f_c' b} \right)$$

$$M_u = 0.90 \times A_s \times 60 \left(ds - \frac{As \times 60}{1.7 \times 4.0 \times 12} \right) \times \left(\frac{1}{12} \right)$$

3.309 A_s^2 - 4.50 $d_s A_s + M_u = 0$

For the reinforcing steel assumed for the heel, d_s = 15.50 in

Substituting and solving for A_s, it is found that required A_s = 0.70 in^2/ft

Try: #8 bars at 12.00 in c/c for the top transverse bars in the heel As = 0.79 in^2/ft

Check Minimum Reinforcement [5.7.3.3.2]

Determine the cracking moment:

$f_r = 0.24\,(f_c')^{0.5}$ = 0.24 x (4.0)$^{0.50}$ = 0.48 ksi

$I_g = (1/12)\,b\,h^3$ = 0.0833 x 12.00 x (18.00)^3 = 5832.0 in^4

$y_t = (1/2)\,h$ = 0.5000 x 18.00 = 9.00 in

$M_{CR} = \gamma_1\,\gamma_3\,f_r\,I_g / y_t$ = 1.60 x 0.67 x 0.48 x 5832.0 / (9.00 x 12.00) = 27.79 k-ft

The capacity of the section must be greater than or equal to the smaller of:

M_{CR} = 27.79 = 27.79 k-ft GOVERNS
(4/3) M_u = 1.33 x 47.15 = 62.86 k-ft

The capacity of the top mat of reinforcement is:

$M_r = \phi\,A_s\,f_y\,(d_s - a/2)$

For the reinforcing steel used, d_s = 18.00 - 2.00 - (1.000 / 2) = 15.50 in

M_r = 0.90 x 0.79 x 60 x (15.50 - $\frac{0.79 \times 60}{1.7 \times 4.0 \times 12}$) x ($\frac{1}{12}$) = 53.04 k-ft

53.04 k-ft > 27.79 k-ft OK

Check minimum reinforcement for temperature and shrinkage (5.10.8) A_s = 0.79 in^2/ft > 0.17 in^2/ft OK

Use #8 bars at 12.00 in c/c for top transverse reinforcement in the footing.

Bottom Transverse Reinforcement

From the shear check of the toe, V_u = 5.24 k/ft

$M_u = V_u \times (w_{toe} / 2)$ = 5.24 x (2.00 / 2) = 5.24 k-ft

Set up the equation to solve for the required steel area and again use:

3.309 A_s^2 - 4.50 $d_s\,A_s + M_u = 0$

For the reinforcing steel assumed for the heel, d_s = 14.75 in

Substituting and solving for A_s, it is found that required A_s = 0.08 in^2/ft

Try: #4 bars at 12.00 in c/c for the bottom transverse bars in the toe As = 0.20 in^2/ft

Check Minimum Reinforcement [5.7.3.3.2]

Determine the cracking moment:

$f_r = 0.24\,(f_c')^{0.5}$ = 0.24 x (4.0)$^{0.50}$ = 0.48 ksi

$I_g = (1/12)\,b\,h^3$ = 0.0833 x 12.00 x (18.00)^3 = 5832.0 in^4

$y_t = (1/2)\,h$ = 0.5000 x 18.00 = 9.00 in

$M_{CR} = \gamma_1\,\gamma_3\,f_r\,I_g / y_t$ = 1.60 x 0.67 x 0.48 x 5832.0 / (9.00 x 12.00) = 27.79 k-ft

The capacity of the section must be greater than or equal to the smaller of:

M_{CR} = 27.79 = 27.79 k-ft
(4/3) M_u = 1.33 x 5.24 = 6.99 k-ft GOVERNS

The capacity of the bottom mat of reinforcement is:

$M_r = \phi\,A_s\,f_y\,(d_s - a/2)$

For the reinforcing steel used, d_s = 18.00 - 3.00 - (0.500 / 2) = 14.75 in

$$M_r = 0.90 \times 0.20 \times 60 \times \left(14.75 - \frac{0.20 \times 60}{1.7 \times 4.0 \times 12}\right) \times \left(\frac{1}{12}\right) = 13.14 \text{ k-ft}$$

13.14 k-ft > 6.99 k-ft OK

Check minimum reinforcement for temperature and shrinkage (5.10.8) A_s = 0.20 in^2/ft > 0.17 in^2/ft OK

Use #4 bars at 12.00 in c/c for bottom transverse reinforcement in the footing.

Longitudinal Reinforcement [5.10.8]

Provide longitudinal reinforcement in the footing based on shrinkage and temperature requirements.

$h_{max} = \max(h_{heel}, h_{toe})$ = 18.00 in

$$\text{Min. } A_s = \frac{1.30\, w_{foot}\, h_{max}}{2\,(w_{foot} + h_{max})\, f_y} = \frac{1.30 \times 114.00 \times 18.00}{2 \times (114.00 + 18.00) \times 60} = 0.17 \text{ in}^2\text{/ft}$$

The maximum spacing of reinforcement is:

$h_{min} = \min(h_{heel}, h_{toe})$ = 18.00 in

Max. spacing = 3 h_{min} = 54.00 in
or Max. spacing = 18 in = 18.00 in
or Max. spacing = 12 in (for walls and footings ≥ 18 in thick) = 12.00 in GOVERNS

Try: #4 bars at 12.00 in c/c for the top and bottom longitudinal bars A_s = 0.20 in^2/ft > 0.17 in^2/ft OK

0.11 < A_s = 0.20 in^2/ft < 0.60 OK

spacing = 12.00 in = 12.00 in OK

Use #4 bars at 12.00 in c/c for top and bottom longitudinal reinforcement in the footing.

Determine Loads for Wall Stem Design

The loads on the stem at the top of the footing are determined to arrive at the design forces for the wall.

Saturated Earth Pressure:

$P_{EH(S)} = (1/2)\, w_{ss}\, K_a\, h_{ss}^2$ = 0.5 x 0.130 x 0.280 x (8.85)^2 = 1.43 k

$M_{EH(S)} = P_{EH(S)} \times [\,(1/3)\, h_{ss} + h_{sb}\,]$ = 1.43 x [0.333 x 8.85 + 0.00] = 4.21 k-ft

Buoyant Earth Pressure:

$P_{EH(B)} = K_a\, h_{sb}\, [w_{ss}\, h_{ss} + (1/2)\, w_{sb}\, h_{sb}]$

$P_{EH(B)}$ = 0.280 x 0.00 x [0.130 x 8.85 + 0.5 x 0.068 x 0.00] = 0.00 k

$y_B = [\, h_{sb}\, (\, w_{ss}\, h_{ss} + (1/3)\, w_{sb}\, h_{sb}\,)\,] \,/\, (2\, w_{ss}\, h_{ss} + w_{sb}\, h_{sb}\,)$

y_B = [0.00 x (0.130 x 8.85 + 0.333 x 0.068 x 0.00)]
/ (2.0 x 0.130 x 8.85 + 0.068 x 0.00) = 0.00 ft

$M_{EH(B)} = P_{EH(B)} \times y_B$ = 0.00 x 0.00 = 0.00 k-ft

Water Pressure:

$P_{WA} = (1/2)\, w_w\, h_{water}^2$ = 0.5 x 0.062 x (0.00)^2 = 0.00 k

$M_{WA} = P_{WA} \times (1/3)\, h_{water}$ = 0.00 x 0.333 x 0.00 = 0.00 k-ft

Live Load Surcharge:

$P_{LS} = w_{ss}\, K_a\, h_{LL}\, h_s$ = 0.130 x 0.280 x 3.45 x 8.85 = 1.11 k

$M_{LS} = P_{LS}$ x (1/2) h_s = 1.11 x 0.500 x 8.85 = 4.92 k-ft

Collision Load at Top of Parapet:

Use a Live Load of 1929 lbs/ft applied at h_r = 3.5 ft above the top of the wall.

P_{CT} = 1.93 k

$M_{CT} = P_{CT}$ x ($h_w + h_r$) = 1.93 x (8.85 + 3.50) = 23.82 k-ft

Using the Strength I load combination, the factored design forces for the wall stem are:

$H_u = 1.50 (P_{EH(S)} + P_{EH(B)}) + 1.00\ P_{WA} + 1.75 (P_{LS} + P_{LL})$

H_u = 1.50 x (1.43 + 0.00) + 1.00 x 0.00 + 1.75 x (1.11 + 0.00) = 4.08 k

$M_u = 1.50 (M_{EH(S)} + M_{EH(B)}) + 1.00\ M_{WA} + 1.75 (M_{LS} + M_{LL})$

M_u = 1.50 x (4.21 + 0.00) + 1.00 x 0.00 + 1.75 x (4.92 + 0.00) = 14.91 k-ft

The Extreme Event II design forces for the wall stem are:

$H_{ext} = 1.50 (P_{EH(S)} + P_{EH(B)}) + 1.00\ P_{CT} + 0.50 (P_{LS} + P_{LL})$

H_{ext} = 1.50 x (1.43 + 0.00) + 1.00 x 1.93 + 0.50 x (1.11 + 0.00) = 4.62 k

$M_{ext} = 1.50 (M_{EH(S)} + M_{EH(B)}) + 1.00\ M_{CT} + 0.50 (M_{LS} + M_{LL})$

M_{ext} = 1.50 x (4.21 + 0.00) + 1.00 x 23.82 + 0.50 x (4.92 + 0.00) = 32.58 k-ft

The service design forces for the wall stem are:

$H_{serv} = 1.00 (P_{EH(S)} + P_{EH(B)}) + 1.00\ P_{WA} + 1.00 (P_{LS} + P_{LL})$

H_{serv} = 1.00 x (1.43 + 0.00) + 1.00 x 0.00 + 1.00 x (1.11 + 0.00) = 2.54 k

$M_{serv} = 1.00 (M_{EH(S)} + M_{EH(B)}) + 1.00\ M_{WA} + 1.00 (M_{LS} + M_{LL})$

M_{serv} = 1.00 x (4.21 + 0.00) + 1.00 x 0.00 + 1.00 x (4.92 + 0.00) = 9.12 k-ft

Wall Stem Design - Investigate Shear

Shear typically does not govern the design of retaining walls. If shear does become an issue, the thickness of the stem should be increased such that transverse reinforcement is not required.

Ignoring the benefits of the shear key and axial compression, the shear capacity of the stem can be shown to be greater than that required.

$V_n = V_c + V_s + V_p$ [5.8.3.3-1]

Recognizing that V_s and V_p are zero

$V_n = V_c$

$V_c = 0.0316\ \beta\ (f_c')^{0.5}\ b_v\ d_v$ [5.8.3.3-3]

The maximum effective shear depth is:

For 2.0 inch clear cover and #7 bars, d_s = 18.00 - 2.00 - (0.875 / 2) = 15.56 in

$d_v = 0.9\ d_e = 0.9\ d_s$ = 0.90 x 15.56 = 14.01 in <u>GOVERNS</u>
$d_v = 0.72\ h = 0.72\ t_{wb}$ = 0.72 x 18.00 = 12.96 in

Follow the General Procedure using the provisions of Appendix B5, as per Section [5.8.3.4.2]:

β = 2.020466 ε_s= 0.00187 s_{xe}= 11.42945 Max Aggregate Size = 1 in (BDM S5.10.3.1.1)

$\phi\ V_c$ = 0.90 x 0.0316 x 2.02 x (4.0)$^{0.50}$ x 12.00 x 14.01 = 19.32 k

19.32 k > 4.62 k OK

Wall Stem Design - Investigate Strength Limit State

Determine the area of back-face flexural reinforcement necessary to satisfy the design moment:

$M_u =$ 32.58 k-ft

Again use the equation:

$3.309 \; A_s^2 - 4.50 \; d_s A_s + M_u = 0$

Substituting and solving for A_s, it is found that required $A_s =$ 0.48 in^2/ft

Try: #7 bars at 12.00 in c/c $A_s =$ 0.60 in^2/ft > 0.48 in^2/ft OK

Wall Stem Design - Investigate Service Limit State [5.7.3.4]

Check the crack control equations to ensure that the primary flexural reinforcement is well distributed. The service load bending moment is:

$M_{serv} =$ 9.12 k-ft

Check the modulus of rupture for concrete:

$\phi f_r = \phi \, 0.24 \, (f_c')^{0.5} = 0.80 \times 0.24 \times (4.0)^{0.50} = 0.384$ ksi

$S = \frac{b \, t_{wb}^2}{6} = \frac{12.00 \times (18.00)^2}{6} = 648.00 \; in^3$

$f_{act} = \frac{M_{serv}}{S} = \frac{9.12 \times 12}{648.00} = 0.169$ ksi < 0.384 ksi OK

If modulus of rupture check is "NG", then check the spacing of the reinforcement. First, determine the modular ratio:

$E_c = 33{,}000 \, w_c^{1.5} (f_c')^{0.5} = 33{,}000 \times (0.150)^{1.5} \times (4.00)^{0.5} = 3{,}834$ ksi

$n = E_s/E_c = 29{,}000 \;/\; 3{,}834 = 7.56$, use n = 8.00

For 2.0 inch clear cover and #7 bars, $d_s = 18.00 - 2.00 - (0.875 \;/\; 2) = 15.56$ in

Determine the location of the neutral axis:

$0.5 \, b \, x^2 = n \, A_s (d_s - x)$

$0.5 \, (12) \, x^2 = 8 \times 0.60 \times (15.56 - x)$

solving, x = 3.15 in

Check the spacing of the reinforcement to control cracking:

$y = d_s - x = 15.56 - 3.15 = 12.41$ in

$I_{cr} = \frac{b \, x^3}{3} + n \, A_s (d_s - x)^2$

$I_{cr} = \frac{12.00 \times (3.15)^3}{3} + 8 \times 0.60 \times (15.56 - 3.15)^2 = 864.56 \; in^4$

$\gamma_e =$ 1.00

For 2.0 inch clear cover and #7 bars, $d_c = 2.00 + (0.875 \;/\; 2) = 2.44$ in

$\beta_s = 1.0 + \frac{d_c}{0.7 \, (t_{wb} - d_c)} = 1.0 + \frac{2.44}{0.7 \times (18.00 - 2.44)} = 1.22$

$f_{ss} = n \frac{M_{serv} \, y}{I_{cr}} = 8 \times \frac{9.12 \times 12.41 \times 12.00}{864.56} = 12.57$ ksi

$s_{max} = \dfrac{700\ \gamma_e}{\beta_s\ f_{ss}} - 2.0\ d_c = \dfrac{700 \times 1.00}{1.22 \times 12.57} - 2.0 \times 2.44 = 40.62$ in

$s_{max} =$ 40.62 in > 12.00 in OK

Wall Stem Design - Check Reinforcement Limits

Check Minimum Reinforcement [5.7.3.3.2]

Determine the cracking moment:

$f_r = 0.24\ (f_c')^{0.5} = 0.24 \times (4.0)^{0.50} = 0.48$ ksi

$I_g = (1/12)\ b\ h^3 = 0.0833 \times 12.00 \times (18.00)^3 = 5832.0$ in^4

$y_t = (1/2)\ h = 0.5000 \times 18.00 = 9.00$ in

$M_{CR} = \gamma_1\ \gamma_3\ f_r\ I_g\ /\ y_t = 0.67 \times 1.6 \times 0.48 \times 5832.0\ /\ (9.00 \times 12.00) = 27.79$ k-ft

The capacity of the section must be greater than or equal to the smaller of:

M_{CR}	=		27.79	=	27.79 k-ft	GOVERNS
$(4/3)\ M_u$	=	1.33 x	32.58	=	43.45 k-ft	

The capacity of the reinforcement is:

$M_r = \phi\ A_s\ f_y\ (d_s - a/2)$

$M_r = 0.90 \times 0.60 \times 60 \times \left(15.56 - \dfrac{0.60 \times 60}{1.7 \times 4.0 \times 12}\right) \times \left(\dfrac{1}{12}\right) = 40.83$ k-ft

40.83 k-ft > 27.79 k-ft OK

Check minimum reinforcement for temperature and shrinkage (5.10.8) $A_s =$ 0.60 in^2/ft > 0.17 in^2/ft OK

Use #7 bars at 12.00 in c/c for wall stem back face vertical reinforcing.

Wall Stem Design - Shrinkage and Temperature Reinforcement [5.10.8]

A minimum amount of reinforcement should be placed near each face of concrete elements to limit the size of cracks associated with concrete shrinkage and temperature changes.

$h_{max} = \max(t_{wt}, t_{wb}) = 18.00$ in

$h_w =$ 8.85 ft = 106.20 in

$\text{Min. } A_s = \dfrac{1.30\ h_w\ h_{max}}{2\ (h_w + h_{max})\ f_y} = \dfrac{1.30 \times 106.20 \times 18.00}{2 \times (106.20 + 18.00) \times 60} = 0.17$ in^2/ft

The maximum spacing of reinforcement is:

$h_{min} = \min(t_{wt}, t_{wb}) = 18.00$ in

Max. spacing = 3 h_{min}	=	54.00	in	
or Max. spacing = 18 in	=	18.00	in	
or Max. spacing = 12 in (for walls and footings ≥ 18 in thick)	=	12.00	in	GOVERNS

Try: #4 bars at 12.00 in c/c

$A_s =$ 0.20 in^2/ft > 0.17 in^2/ft OK

0.11 < $A_s =$ 0.20 in^2/ft < 0.60 OK

spacing = 12.00 in = 12.00 in OK

Use #4 bars at 12.00 in c/c for wall stem front face reinforcing and back face horizontal reinforcing.

Dimensions and Weights for Concrete Design

Footing width, w_{foot} =	9.50	ft	= 114.00	in	Concrete weight, w_c =	0.150	kcf		
					Water weight, w_w =	0.062	kcf		
Footing heel width, w_{heel} =	6.00	ft			Saturated soil weight, w_{ss} =	0.130	kcf		
Footing heel height, h_{heel} =	1.50	ft	= 18.00	in	Buoyant soil weight, w_{sb} =	0.068	kcf		
Footing toe width, w_{toe} =	2.00	ft			Height of wall, h_w =	10.89	ft	top of heel to top of wall	
Footing toe height, h_{toe} =	1.50	ft	= 18.00	in	Height of water, h_{water} =	0.00	ft	top of heel to water line	
Wall width at top, t_{wt} =	1.50	ft	= 18.00	in	Height of soil, h_s =	10.89	ft	top of heel to ground line	
Wall width at base, t_{wb} =	1.50	ft	= 18.00	in	Height of satur. soil, h_{ss} =	10.89	ft	height of satur. soil above top of heel	
					Height of buoy. soil, h_{sb} =	0.00	ft	height of buoy. soil above top of heel	
Concrete strength, f_c' =	4.00	ksi							
Rebar strength, f_y =	60.00	ksi			Active pressure coeff., K_a =	0.280			
Steel mod. of elast., E_s =	29,000	ksi			LL surcharge soil ht., h_{LL} =	3.14	ft		

Optional Collision Loading for Barrier on Top of Wall

Per ODOT comments for I-70/71, use the transverse loading of 54 kips for a TL-4 test level railing [AASHTO, Table A13.2-1] distributed over the retaining wall's joint spacing.

Collision Loading = y (y or n)
Joint Spacing = 28.00 ft
Barrier Height = 3.50 ft

Design Summary

Summary of Design Status

Design Item	Footing Heel	Footing Toe	Wall Stem
Shear	OK	OK	OK
Minimum Reinforcement	OK	OK	OK
Shrinkage & Temperature	OK	OK	OK
Crack Control	N/A	N/A	OK

For calculations related to each design item, see below.

Reinforcing Steel Summary

Footing:	Top transverse:	#8 bars at 12.00 in c/c
	Bottom transverse:	#4 bars at 12.00 in c/c
	Longitudinal:	#4 bars at 12.00 in c/c
Wall Stem:	Back face vertical:	#8 bars at 12.00 in c/c
	Front face vertical:	#4 bars at 12.00 in c/c
	Horizontal:	#4 bars at 12.00 in c/c

Design Footing for Shear [5.13.3.6]

Design footings to have adequate shear capacity without transverse reinforcement.

Determine d_v

Assume: #8 bars at 12.00 in c/c for the top transverse bars in the heel A_s = 0.79 in^2/ft 2.0 in cover
#4 bars at 12.00 in c/c for the bottom transverse bars in the toe A_s = 0.20 in^2/ft 3.0 in cover

For the heel:

$d_{sheel} = h - cover - d_{bar}/2$ = 18.00 - 2.00 - (1.000 / 2) = 15.50 in

$a_{heel} = \frac{A_s f_y}{0.85 f_c' b} = \frac{0.79 \times 60}{0.85 \times 4.0 \times 12}$ = 1.16 in

[5.8.2.9] $d_{vheel} = d_{sheel} - a/2$ = 15.50 - (1.16 / 2) = 14.92 in GOVERNS
or $d_{vheel} = 0.90\ d_e$ = 0.90 x 15.50 = 13.95 in
or $d_{vheel} = 0.72\ h$ = 0.72 x 18.00 = 12.96 in

For the toe:

$d_{stoe} = h - cover - d_{bar}/2$ = 18.00 - 3.00 - (0.500 / 2) = 14.75 in

$a_{toe} = \frac{A_s f_y}{0.85 f_c' b} = \frac{0.20 \times 60}{0.85 \times 4.0 \times 12}$ = 0.29 in

[5.8.2.9] $d_{vtoe} = d_{stoe} - a/2$ = 14.75 - (0.29 / 2) = 14.60 in GOVERNS
or $d_{vtoe} = 0.90\ d_e$ = 0.90 x 14.75 = 13.28 in

or d_{vtoe} = 0.72 h = 0.72 x 18.00 = 12.96 in

Check Heel for Shear

The critical shear section for the heel of the footing is located at the back face of the wall. The heel of the footing is assumed to carry its self weight and the rectangular soil block above it. This neglects the benefit of any upward soil pressure below the footing (conservative).

$V_u = (\gamma_{EV} w_{ss} h_{ss} + \gamma_{EV} w_{sb} h_{sb} + \gamma_{DC} w_c h_{heel} + \gamma_{LL} w_{ss} h_{LL} + \gamma_{WA} w_w h_{water}) \times w_{heel}$

V_u = (1.35 x 0.130 x 10.89 + 1.35 x 0.068 x 0.00 + 1.25 x 0.150 x 1.50 + 1.75 x 0.130 x 3.14 + 1.00 x 0.062 x 0.00) x 6.00 = 17.44 k/ft

[5.8.3.3] Using β = 2.00 and assuming bars in the top mat as above:

[5.8.3.4] $\phi V_c = \phi\, 0.0316\, \beta\, (f_c')^{0.5}\, b_v\, d_v$

[5.8.2.9]

ϕV_c = 0.90 x 0.0316 x 2.00 x (4.0)$^{0.50}$ x 12.00 x 14.92 = 20.37 k

20.37 k > 17.44 k OK

Check Toe for Shear

The peak bearing stress is 3.13 ksf for the Extreme IIb load case.

The critical section for the toe of the footing is at dv from the front face of the wall. For a quick simplified check, try applying the peak bearing stress over the entire length of the toe (conservative).

$V_u = \sigma_V w_{toe}$ = 3.13 x 2.00 = 6.26 k/ft

[5.8.3.3] Using β = 2.00 and assuming bars in the bottom mat as above:

[5.8.3.4] $\phi V_c = \phi\, 0.0316\, \beta\, (f_c')^{0.5}\, b_v\, d_v$

[5.8.2.9]

ϕV_c = 0.90 x 0.0316 x 2.00 x (4.0)$^{0.50}$ x 12.00 x 14.60 = 19.93 k

19.93 k > 6.26 k OK

Design Footing Reinforcement [5.13.3.4]

Each mat of reinforcement is checked to ensure that it has adequate capacity and that the maximum and minimum reinforcement checks are satisfied. The critical section for flexure in the footing is at the face of the wall.

Top Transverse Reinforcement

From the shear check of the heel, V_u = 17.44 k/ft

$M_u = V_u \times (w_{heel} / 2)$ = 17.44 x (6.00 / 2) = 52.32 k-ft

Set up the equation to solve for the required steel area:

$$M_u = \phi M_n = \phi A_s f_y (d_s - a/2) = \phi A_s f_y \left(d_s - \frac{A_s f_y}{1.7 f_c' b} \right)$$

$$M_u = 0.90 \times A_s \times 60 \left(ds - \frac{As \times 60}{1.7 \times 4.0 \times 12} \right) \times \left(\frac{1}{12} \right)$$

$$3.309\ A_s^2 - 4.50\ d_s A_s + M_u = 0$$

For the reinforcing steel assumed for the heel, d_s = 15.50 in

Substituting and solving for A_s, it is found that required A_s = 0.78 in^2/ft

Try: #8 bars at 12.00 in c/c for the top transverse bars in the heel As = 0.79 in^2/ft

Check Minimum Reinforcement [5.7.3.3.2]

Determine the cracking moment:

$f_r = 0.24\,(f_c')^{0.5}$ = 0.24 x (4.0)$^{0.50}$ = 0.48 ksi

$I_g = (1/12)\,b\,h^3$ = 0.0833 x 12.00 x (18.00)^3 = 5832.0 in^4

$y_t = (1/2)\,h$ = 0.5000 x 18.00 = 9.00 in

$M_{CR} = \gamma_1\,\gamma_3\,f_r\,I_g / y_t$ = 1.60 x 0.67 x 0.48 x 5832.0 / (9.00 x 12.00) = 27.79 k-ft

The capacity of the section must be greater than or equal to the smaller of:

M_{CR} = 27.79 = 27.79 k-ft GOVERNS
$(4/3)\,M_u$ = 1.33 x 52.32 = 69.76 k-ft

The capacity of the top mat of reinforcement is:

$M_r = \phi\,A_s\,f_y\,(d_s - a/2)$

For the reinforcing steel used, d_s = 18.00 - 2.00 - (1.000 / 2) = 15.50 in

$$M_r = 0.90 \times 0.79 \times 60 \times \left(15.50 - \frac{0.79 \times 60}{1.7 \times 4.0 \times 12}\right) \times \left(\frac{1}{12}\right) = 53.04 \text{ k-ft}$$

53.04 k-ft > 27.79 k-ft OK

Check minimum reinforcement for temperature and shrinkage (5.10.8) A_s = 0.79 in^2/ft > 0.17 in^2/ft OK

Use #8 bars at 12.00 in c/c for top transverse reinforcement in the footing.

Bottom Transverse Reinforcement

From the shear check of the toe, V_u = 6.26 k/ft

$M_u = V_u \times (w_{toe} / 2)$ = 6.26 x (2.00 / 2) = 6.26 k-ft

Set up the equation to solve for the required steel area and again use:

$3.309\,A_s^2 - 4.50\,d_s\,A_s + M_u = 0$

For the reinforcing steel assumed for the heel, d_s = 14.75 in

Substituting and solving for A_s, it is found that required A_s = 0.09 in^2/ft

Try: #4 bars at 12.00 in c/c for the bottom transverse bars in the toe As = 0.20 in^2/ft

Check Minimum Reinforcement [5.7.3.3.2]

Determine the cracking moment:

$f_r = 0.24\,(f_c')^{0.5}$ = 0.24 x (4.0)$^{0.50}$ = 0.48 ksi

$I_g = (1/12)\,b\,h^3$ = 0.0833 x 12.00 x (18.00)^3 = 5832.0 in^4

$y_t = (1/2)\,h$ = 0.5000 x 18.00 = 9.00 in

$M_{CR} = \gamma_1\,\gamma_3\,f_r\,I_g / y_t$ = 1.60 x 0.67 x 0.48 x 5832.0 / (9.00 x 12.00) = 27.79 k-ft

The capacity of the section must be greater than or equal to the smaller of:

M_{CR} = 27.79 = 27.79 k-ft
$(4/3)\,M_u$ = 1.33 x 6.26 = 8.35 k-ft GOVERNS

The capacity of the bottom mat of reinforcement is:

$M_r = \phi\,A_s\,f_y\,(d_s - a/2)$

For the reinforcing steel used, d_s = 18.00 - 3.00 - (0.500 / 2) = 14.75 in

$$M_r = 0.90 \times 0.20 \times 60 \times \left(14.75 - \frac{0.20 \times 60}{1.7 \times 4.0 \times 12} \right) \times \left(\frac{1}{12} \right) = 13.14 \text{ k-ft}$$

13.14 k-ft > 8.35 k-ft OK

Check minimum reinforcement for temperature and shrinkage (5.10.8) A_s = 0.20 in^2/ft > 0.17 in^2/ft OK

Use #4 bars at 12.00 in c/c for bottom transverse reinforcement in the footing.

Longitudinal Reinforcement [5.10.8]

Provide longitudinal reinforcement in the footing based on shrinkage and temperature requirements.

$h_{max} = max(h_{heel}, h_{toe})$ = 18.00 in

$$\text{Min. } A_s = \frac{1.30\, w_{foot}\, h_{max}}{2\,(w_{foot} + h_{max})\, f_y} = \frac{1.30 \times 114.00 \times 18.00}{2 \times (114.00 + 18.00) \times 60} = 0.17 \text{ in}^2/\text{ft}$$

The maximum spacing of reinforcement is:

$h_{min} = min(h_{heel}, h_{toe})$ = 18.00 in

Max. spacing = 3 h_{min} = 54.00 in
or Max. spacing = 18 in = 18.00 in
or Max. spacing = 12 in (for walls and footings ≥ 18 in thick) = 12.00 in GOVERNS

Try: #4 bars at 12.00 in c/c for the top and bottom longitudinal bars A_s = 0.20 in^2/ft > 0.17 in^2/ft OK

0.11 < A_s = 0.20 in^2/ft < 0.60 OK

spacing = 12.00 in = 12.00 in OK

Use #4 bars at 12.00 in c/c for top and bottom longitudinal reinforcement in the footing.

Determine Loads for Wall Stem Design

The loads on the stem at the top of the footing are determined to arrive at the design forces for the wall.

Saturated Earth Pressure:

$P_{EH(S)} = (1/2)\, w_{ss}\, K_a\, h_{ss}^2$ = 0.5 x 0.130 x 0.280 x (10.89)^2 = 2.16 k

$M_{EH(S)} = P_{EH(S)} \times [\,(1/3)\, h_{ss} + h_{sb}\,]$ = 2.16 x [0.333 x 10.89 + 0.00] = 7.83 k-ft

Buoyant Earth Pressure:

$P_{EH(B)} = K_a\, h_{sb}\, [w_{ss}\, h_{ss} + (1/2)\, w_{sb}\, h_{sb}]$

$P_{EH(B)}$ = 0.280 x 0.00 x [0.130 x 10.89 + 0.5 x 0.068 x 0.00] = 0.00 k

$y_B = [\, h_{sb}\, (\, w_{ss}\, h_{ss} + (1/3)\, w_{sb}\, h_{sb}\,)\,] / (2\, w_{ss}\, h_{ss} + w_{sb}\, h_{sb}\,)$

y_B = [0.00 x (0.130 x 10.89 + 0.333 x 0.068 x 0.00)]
/ (2.0 x 0.130 x 10.89 + 0.068 x 0.00) = 0.00 ft

$M_{EH(B)} = P_{EH(B)} \times y_B$ = 0.00 x 0.00 = 0.00 k-ft

Water Pressure:

$P_{WA} = (1/2)\, w_w\, h_{water}^2$ = 0.5 x 0.062 x (0.00)^2 = 0.00 k

$M_{WA} = P_{WA} \times (1/3)\, h_{water}$ = 0.00 x 0.333 x 0.00 = 0.00 k-ft

Live Load Surcharge:

$P_{LS} = w_{ss}\, K_a\, h_{LL}\, h_s$ = 0.130 x 0.280 x 3.14 x 10.89 = 1.24 k

$M_{LS} = P_{LS}$ x (1/2) h_s = 1.24 x 0.500 x 10.89 = 6.78 k-ft

Collision Load at Top of Parapet:

Use a Live Load of 1929 lbs/ft applied at h_r = 3.5 ft above the top of the wall.

P_{CT} = 1.93 k

$M_{CT} = P_{CT}$ x ($h_w + h_r$) = 1.93 x (10.89 + 3.50) = 27.75 k-ft

Using the Strength I load combination, the factored design forces for the wall stem are:

$H_u = 1.50\ (P_{EH(S)} + P_{EH(B)}) + 1.00\ P_{WA} + 1.75\ (P_{LS} + P_{LL})$

H_u = 1.50 x (2.16 + 0.00) + 1.00 x 0.00 + 1.75 x (1.24 + 0.00) = 5.42 k

$M_u = 1.50\ (M_{EH(S)} + M_{EH(B)}) + 1.00\ M_{WA} + 1.75\ (M_{LS} + M_{LL})$

M_u = 1.50 x (7.83 + 0.00) + 1.00 x 0.00 + 1.75 x (6.78 + 0.00) = 23.61 k-ft

The Extreme Event II design forces for the wall stem are:

$H_{ext} = 1.50\ (P_{EH(S)} + P_{EH(B)}) + 1.00\ P_{CT} + 0.50\ (P_{LS} + P_{LL})$

H_{ext} = 1.50 x (2.16 + 0.00) + 1.00 x 1.93 + 0.50 x (1.24 + 0.00) = 5.79 k

$M_{ext} = 1.50\ (M_{EH(S)} + M_{EH(B)}) + 1.00\ M_{CT} + 0.50\ (M_{LS} + M_{LL})$

M_{ext} = 1.50 x (7.83 + 0.00) + 1.00 x 27.75 + 0.50 x (6.78 + 0.00) = 42.89 k-ft

The service design forces for the wall stem are:

$H_{serv} = 1.00\ (P_{EH(S)} + P_{EH(B)}) + 1.00\ P_{WA} + 1.00\ (P_{LS} + P_{LL})$

H_{serv} = 1.00 x (2.16 + 0.00) + 1.00 x 0.00 + 1.00 x (1.24 + 0.00) = 3.40 k

$M_{serv} = 1.00\ (M_{EH(S)} + M_{EH(B)}) + 1.00\ M_{WA} + 1.00\ (M_{LS} + M_{LL})$

M_{serv} = 1.00 x (7.83 + 0.00) + 1.00 x 0.00 + 1.00 x (6.78 + 0.00) = 14.61 k-ft

Wall Stem Design - Investigate Shear

Shear typically does not govern the design of retaining walls. If shear does become an issue, the thickness of the stem should be increased such that transverse reinforcement is not required.

Ignoring the benefits of the shear key and axial compression, the shear capacity of the stem can be shown to be greater than that required.

$V_n = V_c + V_s + V_p$ [5.8.3.3-1]

Recognizing that V_s and V_p are zero

$V_n = V_c$

$V_c = 0.0316\ \beta\ (f_c')^{0.5}\ b_v\ d_v$ [5.8.3.3-3]

The maximum effective shear depth is:

For 2.0 inch clear cover and #8 bars, d_s = 18.00 - 2.00 - (1.000 / 2) = 15.50 in

$d_v = 0.9\ d_e = 0.9\ d_s$ = 0.90 x 15.50 = 13.95 in <u>GOVERNS</u>
$d_v = 0.72\ h = 0.72\ t_{wb}$ = 0.72 x 18.00 = 12.96 in

Follow the General Procedure using the provisions of Appendix B5, as per Section [5.8.3.4.2]:

β = 2.024824 ε_s= 0.00186 s_{xe}= 11.42945 Max Aggregate Size = 1 in (BDM S5.10.3.1.1)

$\phi\ V_c$ = 0.90 x 0.0316 x 2.02 x (4.0)$^{0.50}$ x 12.00 x 13.95 = 19.28 k

19.28 k > 5.79 k OK

Wall Stem Design - Investigate Strength Limit State

Determine the area of back-face flexural reinforcement necessary to satisfy the design moment:

M_u = 42.89 k-ft

Again use the equation:

$3.309\ A_s^2 - 4.50\ d_s A_s + M_u = 0$

Substituting and solving for A_s, it is found that required A_s = 0.63 in²/ft

Try: #8 bars at 12.00 in c/c A_s = 0.79 in²/ft > 0.63 in²/ft OK

Wall Stem Design - Investigate Service Limit State [5.7.3.4]

Check the crack control equations to ensure that the primary flexural reinforcement is well distributed. The service load bending moment is:

M_{serv} = 14.61 k-ft

Check the modulus of rupture for concrete:

$\phi f_r = \phi\ 0.24\ (f_c')^{0.5}$ = 0.80 x 0.24 x (4.0)$^{0.50}$ = 0.384 ksi

$S = \dfrac{b\ t_{wb}^2}{6} = \dfrac{12.00 \times (18.00)^2}{6}$ = 648.00 in³

$f_{act} = \dfrac{M_{serv}}{S} = \dfrac{14.61 \times 12}{648.00}$ = 0.271 ksi < 0.384 ksi OK

If modulus of rupture check is "NG", then check the spacing of the reinforcement. First, determine the modular ratio:

$E_c = 33{,}000\ w_c^{1.5}\ (f_c')^{0.5}$ = 33,000 x (0.150)$^{1.5}$ x (4.00)$^{0.5}$ = 3,834 ksi

$n = E_s/E_c$ = 29,000 / 3,834 = 7.56 , use n = 8.00

For 2.0 inch clear cover and #8 bars, d_s = 18.00 - 2.00 - (1.000 / 2) = 15.50 in

Determine the location of the neutral axis:

$0.5\ b\ x^2 = n\ A_s\ (d_s - x)$

$0.5\ (12)\ x^2$ = 8 x 0.79 x (15.50 - x)

solving, x = 3.55 in

Check the spacing of the reinforcement to control cracking:

$y = d_s - x$ = 15.50 - 3.55 = 11.95 in

$I_{cr} = \dfrac{b\ x^3}{3} + n\ A_s\ (d_s - x)^2$

$I_{cr} = \dfrac{12.00 \times (3.55)^3}{3}$ + 8 x 0.79 x (15.50 - 3.55)² = 1081.47 in⁴

γ_e = 1.00

For 2.0 inch clear cover and #8 bars, d_c = 2.00 + (1.000 / 2) = 2.50 in

$\beta_s = 1.0 + \dfrac{d_c}{0.7\ (t_{wb} - d_c)} = 1.0 + \dfrac{2.50}{0.7 \times (18.00 - 2.50)}$ = 1.23

$f_{ss} = n\ \dfrac{M_{serv}\ y}{I_{cr}} = 8 \times \dfrac{14.61 \times 11.95 \times 12.00}{1081.47}$ = 15.50 ksi

$$s_{max} = \frac{700\,\gamma_e}{\beta_s f_{ss}} - 2.0\,d_c = \frac{700 \times 1.00}{1.23 \times 15.50} - 2.0 \times 2.50 = 31.70 \text{ in}$$

s_{max} = 31.70 in > 12.00 in OK

Wall Stem Design - Check Reinforcement Limits

Check Minimum Reinforcement [5.7.3.3.2]

Determine the cracking moment:

$f_r = 0.24\,(f_c')^{0.5}$ = 0.24 x (4.0)$^{0.50}$ = 0.48 ksi

$I_g = (1/12)\,b\,h^3$ = 0.0833 x 12.00 x (18.00)^3 = 5832.0 in^4

$y_t = (1/2)\,h$ = 0.5000 x 18.00 = 9.00 in

$M_{CR} = \gamma_1\,\gamma_3\,f_r\,I_g / y_t$ = 0.67 x 1.6 x 0.48 x 5832.0 / (9.00 x 12.00) = 27.79 k-ft

The capacity of the section must be greater than or equal to the smaller of:

M_{CR} = 27.79 = 27.79 k-ft GOVERNS
(4/3) M_u = 1.33 x 42.89 = 57.19 k-ft

The capacity of the reinforcement is:

$M_r = \phi\,A_s\,f_y\,(d_s - a/2)$

$$M_r = 0.90 \times 0.79 \times 60 \times \left(15.50 - \frac{0.79 \times 60}{1.7 \times 4.0 \times 12}\right) \times \left(\frac{1}{12}\right) = 53.04 \text{ k-ft}$$

53.04 k-ft > 27.79 k-ft OK

Check minimum reinforcement for temperature and shrinkage (5.10.8) A_s = 0.79 in^2/ft > 0.17 in^2/ft OK

Use #8 bars at 12.00 in c/c for wall stem back face vertical reinforcing.

Wall Stem Design - Shrinkage and Temperature Reinforcement [5.10.8]

A minimum amount of reinforcement should be placed near each face of concrete elements to limit the size of cracks associated with concrete shrinkage and temperature changes.

$h_{max} = \max(t_{wt}, t_{wb})$ = 18.00 in

h_w = 10.89 ft = 130.68 in

$$\text{Min. } A_s = \frac{1.30\,h_w\,h_{max}}{2\,(h_w + h_{max})\,f_y} = \frac{1.30 \times 130.68 \times 18.00}{2 \times (130.68 + 18.00) \times 60} = 0.17 \text{ in}^2\text{/ft}$$

The maximum spacing of reinforcement is:

$h_{min} = \min(t_{wt}, t_{wb})$ = 18.00 in

Max. spacing = 3 h_{min} = 54.00 in
or Max. spacing = 18 in = 18.00 in
or Max. spacing = 12 in (for walls and footings ≥ 18 in thick) = 12.00 in GOVERNS

Try: #4 bars at 12.00 in c/c A_s = 0.20 in^2/ft > 0.17 in^2/ft OK

0.11 < A_s = 0.20 in^2/ft < 0.60 OK

spacing = 12.00 in = 12.00 in OK

Use #4 bars at 12.00 in c/c for wall stem front face reinforcing and back face horizontal reinforcing.