A Design Example for a Circular Concrete Tank
PCA Design Method

CVEN 4830/4434
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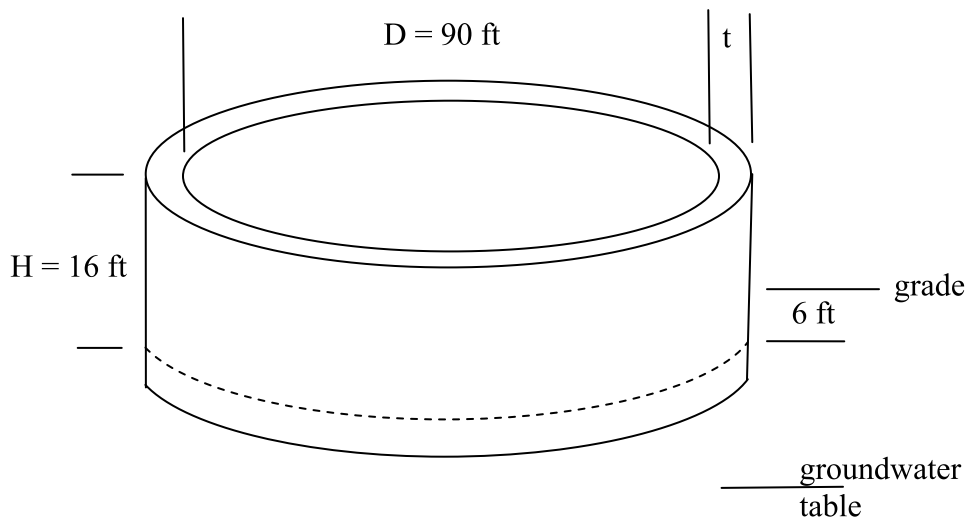
Circular Tank Example



fluid density inside tank = 65 pcf

f'_c = 4,000 psi f_y = 60,000 psi

soil bearing capacity = 2,400 psf

Walls above the groundwater table should be designed using a lateral earth pressure equivalent to that developed by a fluid weighing 40 pcf, below the groundwater table use 90 pcf.

Tensile Hoop Forces

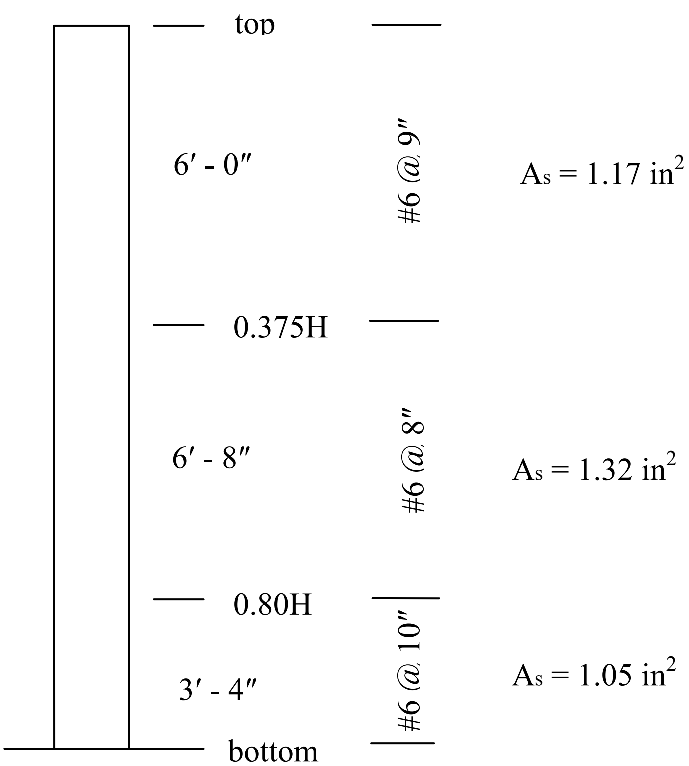
Assume a wall thickness t = 12″

$w_u = 1.65 \times 1.7 \times 65$ pcf = 183 pcf   H = 16 ft   R = 45 ft $H^2/Dt = 2.844 \approx 3.0$

From PCA-C Appendix:

	coef from table A-1	coef from table A-5	larger coef	hoop tensile force $T_u = coef \times w_u \times H \times R$	$A_s = T_u/0.9f_y$
top	0.134	0.074	0.134	17,656 lb	0.33 in^2
0.1H	0.203	0.179	0.203	26,748 lb	0.50 in^2
0.2H	0.267	0.281	0.281	37,025 lb	0.69 in^2
0.3H	0.322	0.375	0.375	49,410 lb	0.92 in^2
0.4H	0.357	0.449	0.449	59,161 lb	1.10 in^2
0.5H	0.362	0.506	0.506	66,671 lb	1.24 in^2
0.6H	0.330	0.519	0.519	68,384 lb	1.27 in^2
0.7H	0.262	0.479	0.479	63,114 lb	1.17 in^2
0.8H	0.157	0.375	0.375	49,410 lb	0.92 in^2
0.9H	0.052	0.210	0.210	27,670 lb	0.52 in^2
bottom	0.000	0.000			

Bars will be installed on both faces as illustrated below:



Note that the above solution is not unique

Hoop Tension in Concrete

$C = 0.0003$
$A_g = 12'' \times 12'' = 144 \text{ in}^2$
$A_s = 1.32 \text{ in}^2$
$E_c = 57{,}000\sqrt{f'_c} = 57{,}000\sqrt{4{,}000 \text{ psi}} = 3{,}605{,}000 \text{ psi}$
$E_s = 29{,}000{,}000 \text{ psi}$
$n = E_s/E_c \approx 8$
$T = 68{,}384 \text{ lb}/(1.65)(1.7) = 24{,}380 \text{ lb}$

$$f_c = \frac{C \cdot E_s \cdot A_s + T}{A_g + n \cdot A_s} = \frac{(0.0003)(29{,}000{,}000 \text{ psi})(1.32 \text{ in}^2) + 24{,}380 \text{ lb}}{144 \text{ in}^2 + 8(1.32 \text{ in}^2)} = 233 \text{ psi}$$

$f_c/f'_c = 0.058 = 5.8\%$ seems reasonable

Compression Hoop Forces

The wall thickness t = 1′

Conservatively consider the soil to be saturated

$w_u = 1.3 \times 1.7 \times 90$ pcf = 199 pcf H = 6 ft R = 45 ft $H^2/Dt = 0.4$

From PCA-C Appendix:

	coef from table A-1	coef from table A-5	larger coef	hoop comp. force $C_u = \text{coef} \times w_u \times H \times R$
top	0.149	0.474	0.474	25,469 lb
0.1H	0.134	0.440	0.440	23,642 lb
0.2H	0.120	0.395	0.395	21,224 lb
0.3H	0.101	0.352	0.352	18,913 lb
0.4H	0.082	0.308	0.308	16,549 lb
0.5H	0.066	0.264	0.264	14,185 lb
0.6H	0.049	0.215	0.215	11,552 lb
0.7H	0.029	0.165	0.165	8,866 lb
0.8H	0.014	0.111	0.111	5,965 lb
0.9H	0.004	0.057	0.057	3,063 lb
bottom	0.000	0.000		

$$\varphi P_n = 0.55(0.75)f'_c A_g = 0.55(0.75)(4{,}000\text{ psi})(12\text{ in})(12\text{ in}) = 237{,}600\text{ lb}$$

Vertical Moment and Shear

The wall thickness t = 12″

It is safe to assume that the internal fluid pressure will cause moments greater than the external soil pressure, even if the soil is saturated. The internal fluid pressure scenario will be used for the flexure design, and reinforcing will be the same on both faces. (For final calculations this should be verified)

$w_u = 1.3 \times 1.7 \times 65$ pcf = 144 pcf   H = 16 ft   R = 45 ft $H^2/Dt = 2.844 \approx 3.0$

From PCA-C Appendix Table A-2:

	coef	$M_u = \text{coef} \times w_u \times H^3$
top	0.0	
0.1H	0.0006	354 lb-ft/ft
0.2H	0.0024	1,416 lb-ft/ft
0.3H	0.0047	2,773 lb-ft/ft
0.4H	0.0071	4,188 lb-ft/ft
0.5H	0.0090	5,309 lb-ft/ft
0.6H	0.0097	5,722 lb-ft/ft
0.7H	0.0077	4,542 lb-ft/ft
0.8H	0.0012	708 lb-ft/ft
0.9H	-0.0119	-7,019 lb-ft/ft
bottom	-0.0333	-19,642 lb-ft/ft

For shear at the base of the wall, table A-12 gives a shear coefficient 0.262 for $H^2/Dt = 3.0$.

$w_u = 1.0 \times 1.7 \times 65$ pcf = 111 pcf

$V_u = \text{coef} \times w_u \times H^2 = 7{,}445$ lb/ft

Lower 3′ of wall:

The wall is 12″ thick wall with 2″ clear concrete cover, try #6 bars @ 6″ for the lower 3 ft of wall.

$b_w = 12''$ $\quad$ $d = 12'' - 2'' - \text{bar dia}/2 = 9.625''$ $\quad$ $A_g = 144 \text{ in}^2$

$f'_c = 4{,}000$ psi $\quad$ $f_y = 60{,}000$ psi $\quad$ $A_s = 0.88 \text{ in}^2$

$$c = \frac{A_s \cdot f_y}{\beta_1 \cdot b_w \cdot (0.85 \cdot f'_c)} = 1.522 \text{ in}$$

$$\phi M_n = (0.9) \cdot A_s \cdot f_y \cdot \left(d - \frac{\beta_1 \cdot c}{2}\right) = 426{,}632 \text{ lb-in/ft} = 35{,}553 \text{ lb-ft/ft}$$

$M_u = 19{,}642$ lb-ft/ft $\quad$ $\phi M_n = 35{,}553$ lb-ft/ft

minimum flexural steel

ACI 350-06 § 10.5.1 $\quad \dfrac{200 \cdot b_w \cdot d}{f_y} \le A_s \quad \Rightarrow A_{s.min} = 0.385 \text{ in}^2$ [flexure steel $A_s = 0.88 \text{ in}^2$]

ACI 350-06 § 10.5.1 $\quad \dfrac{3 \cdot \sqrt{f'_c}}{f_y} \cdot b_w \cdot d \le A_s \quad \Rightarrow A_{s.min} = 0.366 \text{ in}^2$ [flexure steel $A_s = 0.88 \text{ in}^2$]

minimum vertical wall steel

ACI 350-06 § 14.3.2 $\quad 0.003 \times A_g \le A_s \quad \Rightarrow A_{s.min} = 0.432 \text{ in}^2$ [total steel $A_s = 1.76 \text{ in}^2$]

minimum steel for temperature and shrinkage

ACI 350-06 § 14.3.2 $\quad 0.005 \times A_g \le A_s \quad \Rightarrow A_{s.min} = 0.72 \text{ in}^2$ [total steel $A_s = 1.76 \text{ in}^2$]

maximum flexural steel

ACI 318 § 10.3.3 $\quad A_{s,max} = \dfrac{(0.0019125) \cdot \beta_1 \cdot b_w \cdot d \cdot f'_c}{\left(0.003 + \dfrac{f_y}{E_s}\right) \cdot f_y} = 2.469 \text{ in}^2$ [flexure steel $A_s = 0.88 \text{ in}^2$]

Upper 13′ of wall:

The wall is 12″ thick wall with 2″ clear concrete cover, try #6 bars @ 12″ for the wall above the bottom 3 ft.

b_w = 12″   d = 12″ - 2″ - bar dia/2 = 9.625″ A_g = 144 in^2

f'_c = 4,000 psi f_y = 60,000 psi A_s = 0.44 in^2

$$c = \frac{A_s \cdot f_y}{\beta_1 \cdot b_w \cdot (0.85 \cdot f'_c)} = 0.761 \text{ in}$$

$$\phi M_n = (0.9) \cdot A_s \cdot f_y \cdot \left(d - \frac{\beta_1 \cdot c}{2}\right) = 221{,}003 \text{ lb-in/ft} = 18{,}417 \text{ lb-ft/ft}$$

M_u = 7,019 lb-ft/ft ϕM_n = 18,417 lb-ft/ft

minimum flexural steel

ACI 350-06 § 10.5.1 $\frac{200 \cdot b_w \cdot d}{f_y} \le A_s$ ⇒ $A_{s.min}$ = 0.385 in^2 [flexure steel A_s = 0.44 in^2]

ACI 350-06 § 10.5.1 $\frac{3 \cdot \sqrt{f'_c}}{f_y} \cdot b_w \cdot d \le A_s$ ⇒ $A_{s.min}$ = 0.366 in^2 [flexure steel A_s = 0.44 in^2]

minimum vertical wall steel

ACI 350-06 § 14.3.2 $0.003 \times A_g \le A_s$ ⇒ $A_{s.min}$ = 0.432 in^2 [total steel A_s = 0.88 in^2]

minimum steel for temperature and shrinkage

ACI 350-06 § 14.3.2 $0.005 \times A_g \le A_s$ ⇒ $A_{s.min}$ = 0.72 in^2 [total steel A_s = 0.88 in^2]

maximum flexural steel

ACI 318 § 10.3.3 $A_{s,max} = \frac{(0.0019125) \cdot \beta_1 \cdot b_w \cdot d \cdot f'_c}{\left(0.003 + \frac{f_y}{E_s}\right) \cdot f_y}$ = 2.469 in^2 [flexure steel A_s = 0.44 in^2]

Shear

$V_u = 7{,}445$ lb/ft

$$V_c = 2\sqrt{f'_c}\,b_w d$$

$$\phi V_n = \phi V_c = 2\varphi\sqrt{f'_c}\,b_w d = 2(0.75)\sqrt{4{,}000\text{ psi}}(12\text{ in})(9.6875\text{ in}) = 11{,}028\text{ lb/ft}$$

$V_u = 7{,}445$ lb/ft $\qquad$ $\phi V_n = 11{,}028$ lb/ft

Flexure Crack Width

Lower 3′:

M = (19,642 lb-ft/ft)/(1.3)(1.7) = 8,888 lb-ft/ft [unfactored moment]

A_s = 0.88 in^2 [flexure reinforcing]

ρ = steel ratio = $A_s/b_w d$ = (0.88 in^2) / (12″)(9.625″) = 0.00762

n = $E_s/E_c \approx 8$

$$k = \sqrt{2 \cdot \rho \cdot n + (\rho \cdot n)^2} - \rho \cdot n = 0.293$$

J = 1 – k/3 = 0.902

$$f_s = \frac{M}{A_s jd} = \frac{(8{,}888 \text{ lb - ft/ft})}{(0.88 \text{ in}^2)(0.902)(9.625 \text{ in})} = 1{,}164 \text{ psi}$$

d_c = 2.375″

A = 2(2.375″)(12″) = 57 in^2

$$z = \frac{f_s \sqrt{d_c A}}{1000} = \frac{(1{,}164 \text{ psi})\sqrt{(2.375 \text{ in})(57 \text{ in}^2)}}{1000} = 13.5 \qquad [\text{Limit } z \le 95]$$

Upper 13′:

M = (7,019 lb-ft/ft)/(1.3)(1.7) = 3,177 lb-ft/ft [unfactored moment]

A_s = 0.44 in^2 [flexure reinforcing]

ρ = steel ratio = $A_s/b_w d$ = (0.44 in^2) / (12″)(9.625″) = 0.00381

n = $E_s/E_c \approx 8$

$$k = \sqrt{2 \cdot \rho \cdot n + (\rho \cdot n)^2} - \rho \cdot n = 0.218$$

J = 1 – k/3 = 0.927

$$f_s = \frac{M}{A_s jd} = \frac{(7{,}019 \text{ lb-ft/ft})}{(0.44 \text{ in}^2)(0.927)(9.625 \text{ in})} = 1{,}788 \text{ psi}$$

d_c = 2.375″

A = 2(2.375″)(12″) = 57 in^2

$$z = \frac{f_s \sqrt{d_c A}}{1000} = \frac{(1{,}788 \text{ psi})\sqrt{(2.375 \text{ in})(57 \text{ in}^2)}}{1000} = 20.8 \qquad [\text{Limit } z \le 95]$$

Slab Design

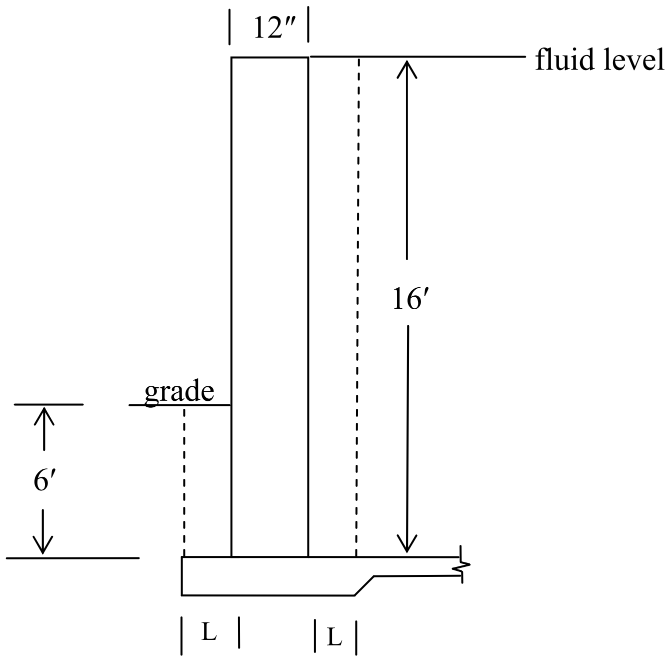
Let the slab thickness to be 6″ with one layer of reinforcing.

#5 @ 10″ each way → $A_s = 0.372 \text{ in}^2$

$A_g = (6'')(12'') = 72 \text{ in}^2$

minimum steel for temperature and shrinkage
ACI 350-06 § 14.3.2 $0.005 \times A_g \le A_s$ ⇒ $A_{s.min} = 0.36 \text{ in}^2$ [total steel $A_s = 0.372 \text{ in}^2$]

Footing Design



Design a 1′ wide strip of wall.

Assume the soil density to be 130 pcf and try L = 2′, a footing thickness of 12″, #5 bars @ 10″ to match the slab reinforcing.

Unfactored Load:

wall weight = (16′)(1′)(150 pcf) = 2,400 lb

fluid weight = (16′)(2′)(65 pcf) = 2080 lb

soil weight = (6′)(2′)(130 pcf) = 1,560 lb

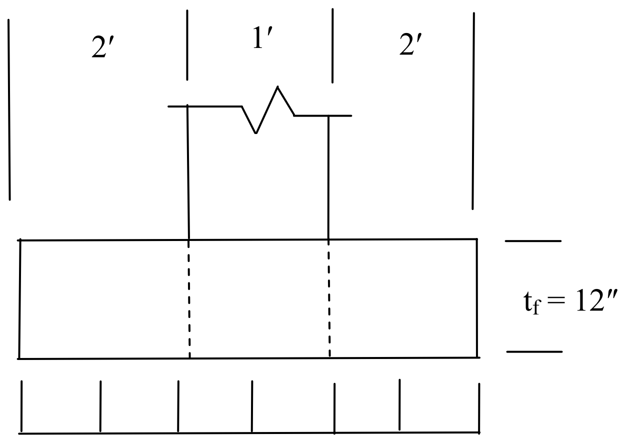
total unfactored weight = 6,040 lb

bearing pressure on soil = 6,040 lb/5 ft^2 = 1,208 lb/ft^2

Note: assuming a 6″ slab thickness, the soil pressure below the slab due to the fluid and the slab will be approximately 1,115 psf. This is very close to the bearing pressure below the footing, which is highly desirable to reduce differential settlement.

Footing Design:

wall weight = 1.3(1.4)(16′)(1′)(150 pcf) = 4,368 lb



w_u = 4,368 lb/5 ft^2 = 874 lb/ft

$M_u = w_u \cdot 2' \cdot 1'$ = 1,748 lb-ft

This is a very low moment that will likely be easily resisted by the minimum flexural steel required. Use #5 @ 10" to match the slab reinforcing.

3″ clear concrete cover → d = 12″ - 3″ - bar dia/2 = 8.6875″ $\qquad A_s = 0.372$ in^2

minimum flexural steel

ACI 350-06 § 10.5.1 $\dfrac{200 \cdot b_w \cdot d}{f_y} \le A_s \quad \Rightarrow A_{s.min} = 0.3475$ in^2 [flexure steel $A_s = 0.372$ in^2]

ACI 350-06 § 10.5.1 $\dfrac{3 \cdot \sqrt{f'_c}}{f_y} \cdot b_w \cdot d \le A_s \quad \Rightarrow A_{s.min} = 0.3297$ in^2 [flexure steel $A_s = 0.372$ in^2]

maximum flexural steel

ACI 318 § 10.3.3 $A_{s,max} = \dfrac{(0.0019125) \cdot \beta_1 \cdot b_w \cdot d \cdot f'_c}{\left(0.003 + \dfrac{f_y}{E_s}\right) \cdot f_y} = 2.228$ in^2 [flexure steel $A_s = 0.372$ in^2]

$$c = \frac{A_s \cdot f_y}{\beta_1 \cdot b_w \cdot (0.85 \cdot f'_c)} = 0.644 \text{ in}$$

$$\phi M_n = (0.9) \cdot A_s \cdot f_y \cdot \left(d - \frac{\beta_1 \cdot c}{2}\right) = 169{,}020 \text{ lb-in/ft} = 14{,}085 \text{ lb-ft/ft}$$

M_u = 1,748 lb-ft/ft $\qquad \phi M_n$ = 14,085 lb-ft/ft

Shear:

$V_u = 1.0(1.4)(874 \text{ lb/ft})(2') = 2{,}448 \text{ lb/ft}$

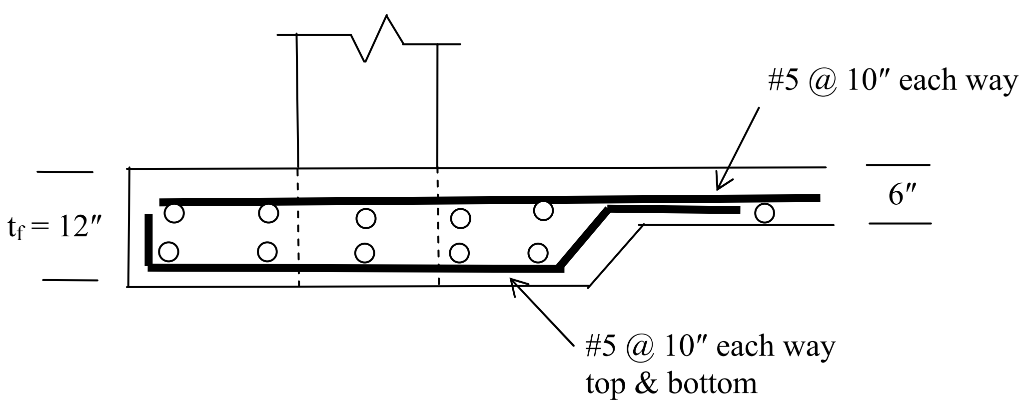
$V_c = 2\sqrt{f'_c}b_w d = 13{,}186 \text{ lb/ft}$

design shear strength = $\phi V_n = 0.75V_c = 9{,}890 \text{ lb/ft}$

$V_u = 2{,}448 \text{ lb/ft}$ $\phi V_n = 9{,}890 \text{ lb/ft}$

Minimum Footing Reinforcing:

The minimum reinforcing for temperature and shrinkage will include the slab reinforcing that extends into the footing.



$A_g = (12'')(12'') = 144 \text{ in}^2$

total $A_s = 2(0.31 \text{ in}^2)(12''/10'') = 0.744 \text{ in}^2$

minimum steel for temperature and shrinkage
ACI 350-06 § 14.3.2 $0.005 \times A_g \le A_s$ $\Rightarrow$ $A_{s.min} = 0.72 \text{ in}^2$