Appendix D Example 30 – Short Poured-In-Place Concrete Piles

Refer to *Falsework Manual,* Section 5-6, *Short Poured-In-Place Concrete Piles* and the sample calculations in Appendix D Example 29 – *Short Poured-In-Place Concrete Piles*. This example demonstrates how to perform a complete analysis for a short poured-in-place concrete pile.

Given Information

A contractor proposes to use an 18-inch diameter poured in place concrete pile as an anchorage for his falsework cable bracing. Prior to being used for bracing the falsework this pile will be used as an anchorage for the column reinforcing cage and form.

This anchor pile will be subjected to three short term loads in the same direction.

Soil (cohesionless):
Internal Friction ϕ = 35°
Unit weight $\gamma_s = 110$ pcf

Concrete:
Unit weight γ_c = 145 pcf
Compressive strength f'_c = 3250 psi

Bar Reinforcing Steel:
2-#5 Grade 60 bars, full length each side of centerline

Pile Dimensions:
a = 2 in.
b = 6 ¼ in
d = 18 in = 1.5 ft
e = 1.2 ft
θ = 35°
L = 12 ft (lower 2 ft submerged)

Design load:
P = 8000 lbs

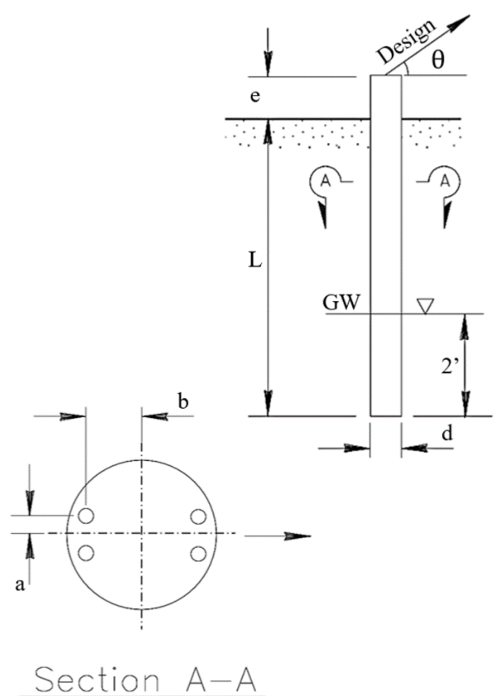

Figure D-30-1. Laterally Loaded Reinforced Concrete Pile

Determine the Adequacy of the Pile

Check for adequate reinforcement clearance

Find distance from center of pile to center of bar (use geometry) and bar radius

9 in – $\{\sqrt{(6.25)^2 + (2)^2} + \frac{0.625}{2}\}$ = 2.13 in > 2 in minimum clearance **OK**

Calculate load components

H_{DESIGN} = Design cos 35° = (8000) (cos (35°)) = 6553 lbs

V_{DESIGN} = Design sin 35° = (8000) (sin (35°)) = 4589 lbs

Calculate Factor of Safety

FS = 2.0 + (x-1) (0.25) = 2.0 + (3-1) (0.25) (5-6.03-1)

= 2.5 for lateral soil loading

Check Uplift Capacity

$S = \beta\sigma_z$

where:

z = 12 ft (z_{dry} = 10 ft; Z_{wet} = 2 ft)

σ_z = 10(110) + 2(110 – 62.4) = 1195 psf

$\beta = 1.5 - 0.315\, z^{1/2}$ = 1.5 – 0.315$(12)^{1/2}$ = 0.41 > 0.25 **OK**

$S = 0.41\ (1{,}195) = 490\ \text{psf} \quad 4{,}000\ \text{psf} \quad \text{OK}$

Net pile shearing resistance $Rs = \pi dzS = \pi(1.5)(12)(490) = 27{,}709\ \text{lbs}$

Pile weight W = $\pi\left(\frac{d}{2}\right)^2 L_p\, \gamma_c = \pi\left(\frac{1.5}{2}\right)^2 (12 + 1.2)(145)$ = 3382 lbs

Ultimate load capacity R = 27,709 + 3382 = 31,091 lbs

Working load V = $\frac{31{,}091}{2.5}$ = 12,436 lbs > 4,589 lbs **OK**

Check lateral capacity

4d = (4) (1.5) = 6.0 ≤ 12 ft (Meets minimum embedment length requirements)

$$K_p = \tan^2\left(45° + \frac{\phi}{2}\right) = \tan^2\left(45° + \frac{35°}{2}\right) = 3.69$$

L/d = 12/1.5 = 8.0 ≤ 20 (meets short pile criteria) e/d = 1.2/1.5 = 0.8

From Figure 5-23, $\frac{H_{ULT}}{K_p \gamma_s d^3} \approx 16$

Find effective unit weight of soil by using weighted average to account for variable soil layers:

$$\gamma_2 = \frac{(10)(110) + (2)(110 - 62.4)}{12} = 99.6 \text{ pcf}$$

$H_{ULT} = 16 \times K_p \gamma_s d^3 = 16 \times (3.69)(99.6)(1.5)^3 = 19{,}846$ lbs

Working Load Value for $H_{ULT} = \frac{19{,}846}{2.5} = 7938$ lbs > 6533 lbs **<u>OK</u>**

$$(f_g)^2 = \frac{H_{ULT}}{1.5\,\gamma_s\, dK_p} \qquad (5\text{-}6.03A\text{-}1)$$

$$f_g = \left(\frac{H_{ULT}}{1.5\,\gamma_s\, dK_p}\right)^{\frac{1}{2}} = \left(\frac{19{,}846}{1.5\,(99.6)(1.5)(3.69)}\right)^{\frac{1}{2}} = 4.90 \text{ ft}$$

$$M_{ULT} = H_{ULT}\left(e + \frac{2f_g}{3}\right)$$

$$= (19{,}846)\left(1.2 + \frac{(2)(4.90)}{3}\right) = 88{,}645 \text{ ft-lb}$$

Working Load Value for $M_{ULT} = \frac{88{,}645}{2.5} = 35{,}458$ ft-lb

Pile Adequacy

Pile capacity is to be based on design loads. The lateral force H_{DESIGN} may be substituted for H_{ULT} and M_{DESIGN} for M_{ULT} in the critical soil equations.

V_{DESIGN} = 4,589 lbs

H_{DESIGN} = 6,553 lbs

$$(f_g)^2 = \frac{H_{ULT}}{1.5\,\gamma_s\, dK_p} \qquad (5\text{-}6.03A\text{-}1)$$

$$f_g = \left(\frac{H_{ULT}}{1.5\,\gamma_s\, dK_p}\right)^{\frac{1}{2}} = \left(\frac{6553}{1.5\,(99.6)(1.5)(3.69)}\right)^{\frac{1}{2}} = 2.82 \text{ ft}$$

$$M_{DESIGN} = M_{ULT} = H_{ULT}\left(e + \frac{2f_g}{3}\right) \qquad (5\text{-}6.03A\text{-}2)$$

$$M_{DESIGN} = 6553\left(1.2 + \frac{(2)(2.82)}{3}\right) = 20{,}183 \text{ ft-lb}$$

Depth to plane of zero shear of pile $\approx \frac{M_{DESIGN}}{H_{DESIGN}} \approx \frac{20{,}183}{6{,}553} = 3.08$ ft

Concrete Stress

Pile weight = $\pi\left(\frac{d}{2}\right)^2 (3.08 + e)\,\gamma_c = \pi\left(\frac{1.5}{2}\right)^2 (3.08+1.2)(145) = 1096$ lbs

V' = 4589 – 1096 = 3493 lbs

$$I_g = \frac{\pi d^4}{64} = \frac{\pi(18)^4}{64} = 5153 \text{ in}^4$$

$$A_g = \frac{\pi d^2}{4} = \frac{\pi(18)^2}{4} = 254.5 \text{ in}^2$$

$$f_c = \frac{Md}{2I_g} - \frac{V'}{A_g} \leq \frac{f'c}{2} \qquad (5\text{-}6.04\text{-}1)$$

$$f_c = \frac{(20{,}183)(12)(1.5)(12)}{(2)(5153)} - \frac{3493}{254.5} = 409 \text{ psi} < 1625 \text{ psi} = \frac{f'_c}{2} \qquad \underline{\textbf{OK}}$$

$$V_u = 2\sqrt{f'_c} = 2\sqrt{3250} = 114 \text{ psi}$$

$$v_u = \frac{v}{0.5bd} \approx \frac{v}{0.5A} \approx \frac{6{,}553}{(0.5)(254.5)} \approx 51 \text{ psi}$$

51 psi < 114 psi allowable **OK**

Bar Reinforcing Stress

A_s = 0.31 in^2 d_{bar} = 0.625 in

d_s = 2b = (2)(6.25) = 12.50 in

ΣA_s = 4(0.31 in^2) = 1.24 in^2

$$f_s = \frac{M}{A_s d_s} + \frac{V'}{\Sigma A_s} = \frac{(20{,}183)(12)}{(2 \text{ x } 0.31)(12.50)} + \frac{3493}{1.24} = 34{,}068 \text{ psi}$$

$F_s \leq 0.70\ F_y$ = 0.7 (60,000) = 42,000 psi (5-6.05-4)

34,068 psi < 42,000 psi allowable **OK**

This pile is capable of resisting the applied loads. The pile is satisfactory for use as designed by the contractor.