Drilled Shaft Design

14.528 Drilled Deep Foundations
Spring 2014
Homework No. 3

March 19, 2014

Table 1. Representative Soil Profile and Properties for Drilled Shaft Design

Stratum	Depth (ft)		Thickness	N_{corr}	γ	ϕ'	S_u/q_u	f_s (ksf)		q_p (ksf)	
	Top	Bottom	(ft)	(bpf)	(pcf)	(deg.)	(ksf)	Ultimate	Mobilized	Ultimate	Mobilized
Granular fill	0	22	22	32	125	35	N/A	N/A	N/A	N/A	N/A
Organic Silt	22	27	5	N/A	100	0	0.5	N/A	N/A	N/A	N/A
Clay - BBC	27	44	17	13	120	0	2.7	0.9	**0.6**	N/A	N/A
Glacial Deposits	44	54	10	44	135	39	N/A	3.0	**1.8**	N/A	N/A
Weathered Rock	54	75	22	>100	145	44	N/A	4.9	**2.9**	N/A	N/A
Fractured Rock	75	114	39	N/A	150	N/A	535	21.9	**13.1**	300.0	**30.0**
Competent Rock	114	142	28	N/A	172	N/A	1300	N/A	N/A	N/A	N/A

Table 2. Summary of Required Drilled Shaft Design Lengths, Allowable Capacity, and Estimated Settlement

Design Load (kips)	Total Length Req'd (ft)	Req'd Depth in FR (ft)	Ultimate Capacity (kips)				Allowable Capacity, Q_{all} (kips)	Estimated Settlement (in)
			Skin Resistance, Q_{skin}	Tip Resistance, Q_{tip}	Boyant Shaft Weight, W'	Total Capacity, Q_{ult}		
3850	100	25	9161	1155	368	9948	**3980**	**0.29**
2700	90	15	6280	1155	335	7100	**2840**	**0.23**

Homework #3: Drilled Shaft Design

Given:
- 7 ft diameter drilled shafts (2)
- P = 2700 & 3850 Kips
- Subsurface profile & soil profile attached
- $f_c' = 4000$ psi
- A_s = 1% total section area (f_y = 36ksi, E = 29,000 ksi)
- Casing extended to top of clay layer
- FS = 2.5 & Max settlement = 0.5 inches

Required:
1. Require DS lengths for 2 axial loads
2. Estimated shaft settlement
3. Structural capacity of drilled shafts

Solution:

(1) i) side resistance:

⇒ Clay (S_u = 2700 psf): use recomendations in FHWA IF-99-025

↳ $f_{max} = \alpha S_u$ where $\alpha = 0.55$ for $S_u/P_a \leq 1.50$

$S_u/P_a = (2.7\text{ksf})/(2.12\text{ksf}) = 1.27 \leq 1.50$

↳ use $\alpha = 0.55$ (Note: S_u in N.C. Clay $= 0.22\sigma_v' \approx 630$psf)

↳ use ave S_u to account of NC bottom layer: $f_{max} = 0.55\left(\frac{2700+630\text{psf}}{2}\right) = 915$psf

⇒ Glacial Deposits ($\phi' = 39°$, $N_{1,60} = 44$ bpf)

↳ $f_{max} = \beta\sigma_v'$ where $\beta = 2.0 - 0.15(z_i)^{0.75}$ in gravelly sands
z_i = depth to center of layer (m)

$\sigma_v' = (125\text{pcf})(22\text{ft}) + (110\text{pcf})(5\text{ft}) + (120\text{pcf})(17\text{ft}) + (135\text{pcf})(10\text{ft}/2) - (62.4\text{pcf})(49\text{ft}-13\text{ft})$

$\sigma_v' = 3770$ psf

$\beta = 2.0 - 0.15(49\text{ft}/3.28\text{m/ft})^{0.75} = 0.86$

↳ assume upper limit = $K\tan\delta$ where $K = 1$ & $\delta = \phi'$

$\beta_{max} = (1.0)\tan 39° = 0.81$

↳ $f_{max} = \beta \sigma_v' = (0.81)(3770\,psf) = 3055\,psf$

⟹ Weathered Rock ($\phi' = 44°$, $N_{1,60} \geq 100\,bpf$)

↳ $f_{max} = \sigma_v' K_o \tan\phi'$ for cohesionless IGM

where $K_o \cong (1-\sin\phi)\left[\frac{0.2 P_a N_{1,60}}{\sigma_v'}\right]^{(\sin\phi')}$

σ_v' @ center (64.5 ft) $= (125\,pcf)(22\,ft) + (110\,pcf)(5\,ft) + (120\,pcf)(17\,ft) + (135\,pcf)(10\,ft) + (145\,pcf)(22\,ft/2) - (62.4\,pcf)(64.5\,ft - 13\,ft)$

$\sigma_v' = 5070\,psf$

$K_o \cong (1-\sin 44)\left[\frac{0.2(2.12\,ksf)(100\,bpf)}{(5.07\,ksf)}\right]^{(\sin 44°)} = 1.34$

↳ assume $K_{o,max} = 1.0$ for design

↳ $f_{max} = \sigma_v' K_o \tan\phi' = (5070\,psf)(1.0)\tan(44°) = 4895\,psf$

⟹ Fractured Rock ($q_u = 535\,ksf$, RQD = 28%)

↳ $f_{max} = 0.65 P_a [q_u/P_a]^{0.5} \leq 0.65 P_a [f_c'/P_a]^{0.5}$

↳ $f_{max} = 0.65(2.12\,ksf)\left[\frac{535\,ksf}{2.12\,ksf}\right]^{0.5} = 21.9\,ksf$

↳ $f_c' = 4000\,psi = 576\,ksf > 535\,ksf$ use $f_{max} = 21.9\,ksf$

① ii) Tip resistence

↳ assume end bearing in Fractured Rock

↳ evaluate two methods outlined in FHWA-IF-99-025

↳ lower bound solution: $q = [s^{0.5} + (ms^{0.5} + s)^{0.5}]\,q_u$

where m+s are related to rock quality + based on rock quality

↳ assume the following: gouge filled open joints: 0.05 to 0.20 in
water under moderate pressure
joints spaced 2 in to 1 ft

↳ given $q_u = 535\,ksf$, $RQD_{ave} = 28\%$ & using AASHTO 2007 (table 10.4.6.4-1)

↳ $RMR \cong 32$

↳ using Table 11.3 in FHWA-IF-99-05 to find m & s
↳ $m = 0.05$ $s = 10^{-5}$ for Argillaceous rock of poor quality

$$q = \left[(10^{-5})^{0.5} + \left(0.05 \times \sqrt{10^{-5}} + 10^{-5}\right)^{0.5}\right](535\ ksf) = 8.63\ ksf$$

↳ result is low & is considered the lower bound solution

↳ Evaluate alternative methods for end bearing in rock
↳ equation 11.8 in FHWA-IF-99-025 is applicable if:
a) Rock description B - argillaceous rocks
b) $B > 12$ in
c) $0.05 < S_v/B < 2.0$
d) $0 < t_d/S_v < 0.02$

where S_v is the average vertical spacing btwn joints
t_d = average thickness of joints

↳ assuming $S_v \cong 6$ inches & $t_d \cong 0.125$ inches

$S_v/B = 6''/84'' = 0.07$ ✓ good
$t_d/S_v = 0.125''/6'' = 0.02$ ✓ good

↳ $q_{max} = 3 K_{sp} \theta q_u$
↳ $K_{sp} = \dfrac{3 + S_v/B}{10\sqrt{1 + 300\, t_d/S_v}} = \dfrac{3 + (6''/84'')}{10\sqrt{1 + 300 \frac{0.125''}{6''}}} = 0.114$

$\theta = 1 + 0.4\left(\dfrac{D_s}{B}\right)$ where D_s = depth in F. Rock

↳ assume D_s = 10 to 20 ft
↳ $\theta = 1 + 0.4\left[\dfrac{10\ ft}{7\ ft}\right] = 1.57$ $\theta = 1 + 0.4\left[\dfrac{20\ ft}{7\ ft}\right] = 2.14$

$q_1 = 3(0.114)(1.57)(535\ ksf) = 285\ ksf$
$q_2 = 3(0.114)(2.14)(535\ ksf) = 390\ ksf$

↳ this method assumes horizontal joints and therefore may be considered the upper bound.
↳ use $q_{ult} = 300$ ksf and evaluate mobilized tip resistance based on estimated shaft settlement.
↳ use methods described in Appendix C of FHWA-IF-99-025 to evaluate mobilized tip & shaft resistance

$\delta_s = k\frac{Q_T L}{AE}$ where $A = \frac{\pi B^2}{4} = \frac{\pi(84in)^2}{4} = 5542\ in^2$

↳ $A_s = 0.01(5542 in^2) = 55.42 in^2$

$A_c = 5542 in^2 - 55.42 in^2 = 5486.58 in^2$

$E_c = 57\sqrt{f_c'} = 57\sqrt{4000psi} = 3605 ksi$

$E_s = 29000 ksi$

↳ $E_{comp} = \frac{(3605 ksi)(5486 in^2) + (29000ksi)(55 in^2)}{(5542 in^2)}$

↳ $E_{comp} = 3860 ksi$

$L = 90ft = 1080 in$ (assumed)

$K = 0.5$ (friction only) to 1.0 (end bearing only)

↳ use 0.67 for preliminary evaluation

↳ $\delta_s = \frac{(0.67)(3850 kips)(1080 in)}{(5542 in^2)(3860 ksi)} = 0.13 in$

for flexible foundation, settlement along shaft (w_s) & at the base (w_b) will vary

↳ estimate total settlement (w_T) using Paikowsky et al (2008)

↳ determine $w_s = w_T - 0.5\delta_s$ & $w_B = w_T - \delta_s$

↳ $w_T \cong \frac{(0.15 + B/120)}{6} + \frac{2}{3}\frac{PL}{AE}$

$w_T = \frac{(0.15 + 84''/120)}{6} + \frac{2}{3}\frac{(3850 kips)(1080 in)}{(5542 in^2)(3860 ksi)} = 0.27 in$

↳ $w_s = 0.27 - 0.5(0.13 in) = 0.205 in$

↳ $w_B = 0.27 - (0.13 in) = 0.14 in$

- Estimate % side load transfer: $\frac{\text{settlement}}{\text{diameter}} = \frac{0.205 in}{84 in} = 0.2\%$

↳ using trendline in Figure C.3 → $\frac{\text{side load transfer}}{\text{ult. side load}} = 60\%$

- Estimate % End bearing: $\frac{\text{Base settlement}}{\text{diameter}} = \frac{0.14 in}{84 in} = 0.2\%$

↳ using trendline in Figure C.4 → $\frac{\text{End Bearing}}{\text{ult. End bearing}} = 10\%$

↳ Assume 60% side friction (from prev. calculated $f_{s,max}$) & 10% end bearing.

Strata	Thickness (ft)	unit skin friction (f_s) (ultimate) (mobilized)	unit tip resistance (q_t) (ultimate) (mobilized)
Clay (27 to 44 ft)	17	0.9 ksf × 0.6 = 0.6 ksf	N/A
G. Deposits (44 to 54 ft)	10	3.0 ksf × 0.6 = 1.8 ksf	N/A
W. Rock (54 to 75 ft)	21	4.9 ksf × 0.6 = 2.9 ksf	N/A
F. Rock (75 to 114 ft)	39	21.9 ksf × 0.6 = 13.1 ksf	300 ksf × 0.1 = 30 ksf

① iii) Length required to resist 2700 kips (F.S. = 2.5)
↳ Check 90 ft (15 ft in F. Rock)

$Q_{all} = [Q_s + Q_t - W']/2.5$

↳ $W' = [\gamma_{conc}(G.W.T.) + (L - GWT)\gamma'_{conc}] \times (\text{Area})$

↳ $[(150\,pcf)(13ft) + (90ft - 13ft)(150\,pcf - 62.4\,pcf)]\left[\frac{\pi(7ft)^2}{4}\right]$

↳ $W' = 335$ kips

↳ $Q_s = \pi D(\Sigma f_{si} \times D_i)$

↳ $\pi(7ft)[(17ft)(0.6ksf) + (10ft)(1.8ksf) + (21ft)(2.9ksf) + (15ft)(13.1ksf)]$

↳ $Q_s = 6280$ kips

↳ $Q_t = \frac{\pi D^2}{4}[q_t] = \frac{\pi(7ft)^2}{4}(30ksf) = 1155$ kips

↳ $Q_{all} = [6280k + 1155k - 335k]/2.5$

↳ $Q_{all} = 2840$ kips ≥ 2700 kips ✓ good

* Follow same approach for length required to resist 3850 kips

↳ Check L = 100 ft (25 ft in F. Rock)

↳ $Q_s = 9161$ kips $Q_t = 1155$ kips $W' = 368$ kips

↳ $Q_{all} = (9161k + 1155k - 368k)/2.5$

↳ $Q_{all} = 3980$ kips ≥ 3850 kips ✓ Good

② Estimate settlement @ top of shaft

P = 2700 kips   L = 90 ft = 1080 in   A = 5542 in²   E = 3860 ksi

↳ $S = \frac{x}{6} + \frac{2}{3}\frac{PL}{AE}$ where $x = 0.15 + \frac{B}{120} = 0.15 + \frac{84\text{in}}{120} = 0.85"$

↳ $S = \frac{0.85"}{6} + \frac{2}{3}\frac{(2700k)(1080in)}{(5542in^2)(3860ksi)}$

↳ $S = 0.23$ inches ≤ 0.50 in   ✓ good

P = 3850 kips   L = 100 ft = 1200 in

↳ $S = \frac{0.85"}{6} + \frac{2}{3}\frac{(3850\text{ kips})(1200\text{ in})}{(5542in^2)(3860ksi)}$

↳ $S = 0.29$ inches ≤ 0.5 in   ✓ Good

③ Evaluate structural capacity of shaft

↳ $f_c' = 4000$ psi $f_y = 36{,}000$ psi

↳ $\sigma_y = \frac{A_c F_c' + A_s f_y}{A_t}$ where $A_t = 5542 in^2$; $A_c = 5486 in^2$; $A_s = 55.42 in^2$

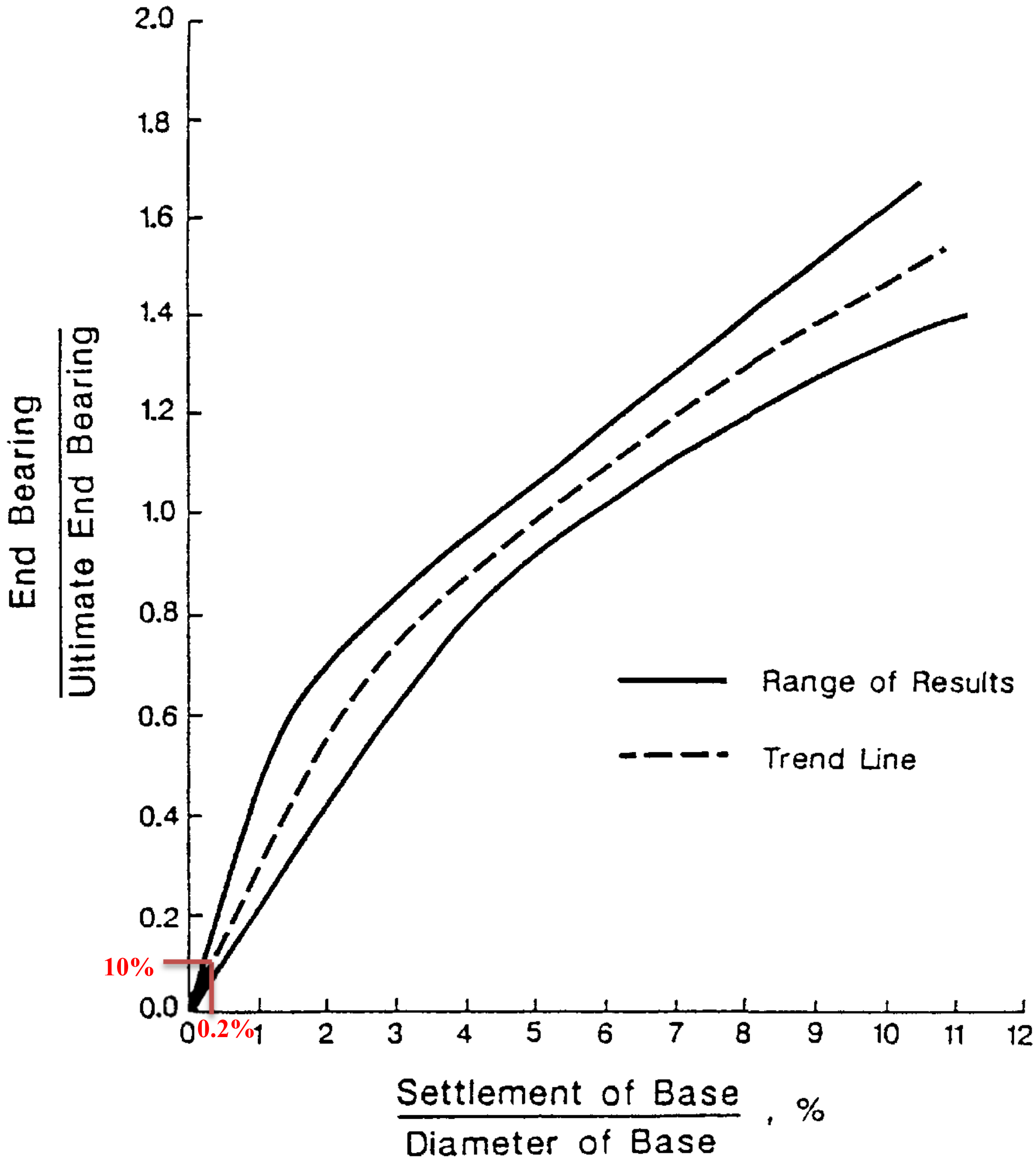
↳ $\sigma_y = \frac{(5486in^2)(4000psi) + (55.42in^2)(36{,}000psi)}{(5542in^2)}$

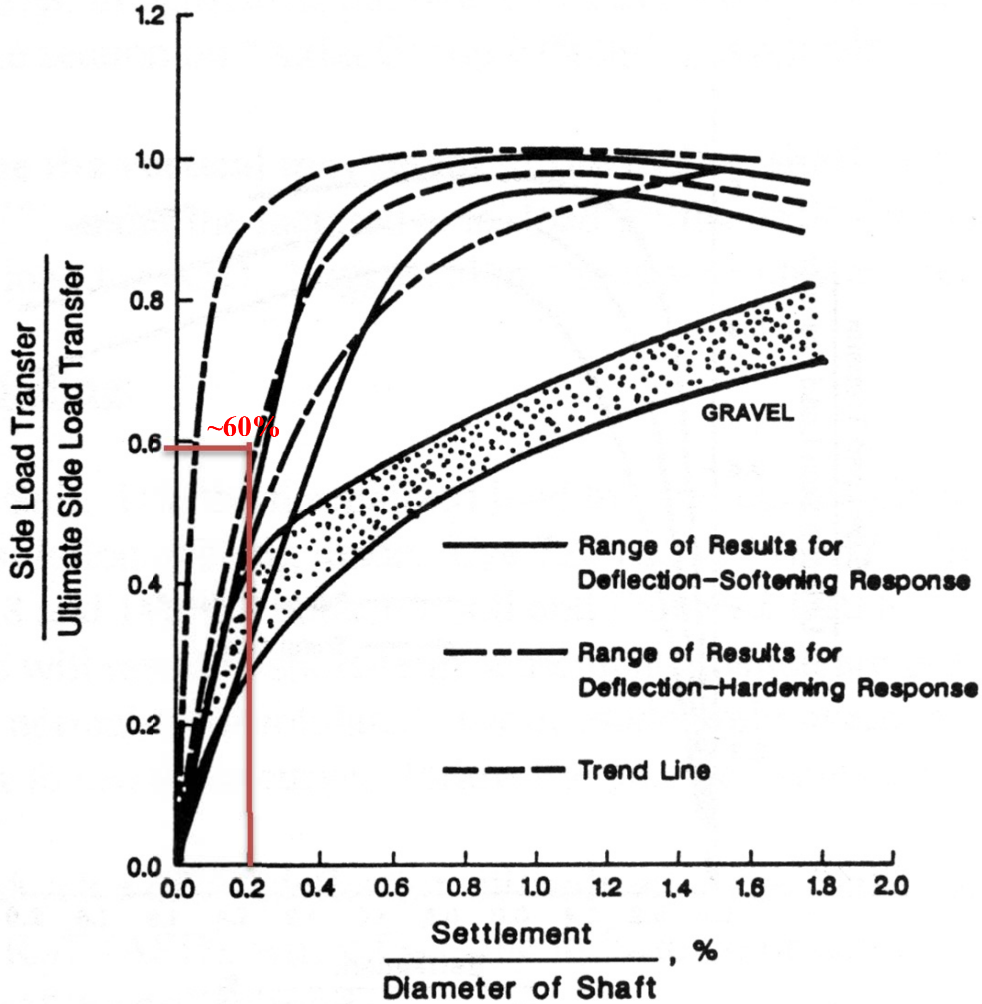
↳ $\sigma_y = 4320$ psi

→ $P_{au} = \sigma_y A_t / FS$ where FS = 2.5

↳ $P_{au} = (4320psi)(5542in^2)/(2.5)$

↳ $P_{au} = 9575$ kips ≥ 3850 kips   ✓ good.


End Bearing / Ultimate End Bearing
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
10%
0.2%
0
1
2
3
4
5
6
7
8
9
10
11
12
Range of Results
Trend Line
Settlement of Base / Diameter of Base , %



Side Load Transfer / Ultimate Side Load Transfer
1.2
1.0
0.8
0.6
0.4
0.2
0.0
~60%
GRAVEL
Range of Results for Deflection–Softening Response
Range of Results for Deflection–Hardening Response
Trend Line
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Settlement / Diameter of Shaft, %